S.E. Exam Review: Bridge Design

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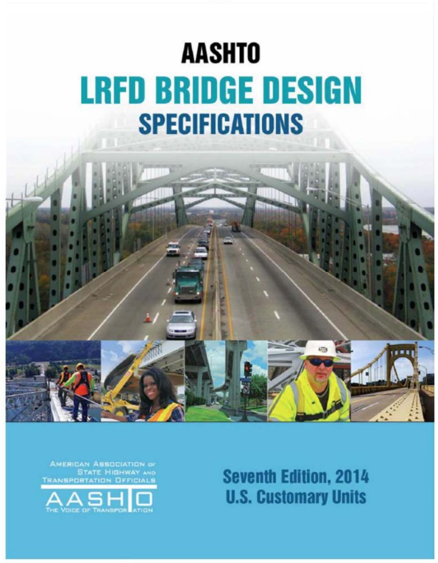
Basic Information

- AASHTO LRFD Bridge Design Specifications
  - 7th Edition
  - 2015, 2016, 2017 and 2018 interims available
  - Beware of other versions
- Strengths always in ksi

AASHTO Manual Overview

AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS
7th Edition
2014



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AASHTO Manual Overview

SECTION 3: LOADS AND LOAD FACTORS

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SECTION 3:
LOADS AND LOAD FACTORS

3.1—SCOPE

This Section specifies minimum requirements for loads and forces, the limits of their application, load factors, and load combinations used for the design of new bridges. The load provisions may also be applied to the structural evaluation of existing bridges.

Where multiple performance levels are provided, the selection of the design performance level is the responsibility of the Owner.

A minimum load factor is specified for force effects that may develop during construction. Additional requirements for construction of segmental concrete bridges are specified in Article 5.14.2.

C3.1

This Section includes, in addition to traditional loads, the force effects due to collisions, earthquakes, and settlement and distortion of the structure.

Vehicle and vessel collisions, earthquakes, and aeroelastic instability develop force effects that are dependent upon structural response. Therefore, such force effects cannot be determined without analysis and/or testing.

With the exception of segmental concrete bridges, construction loads are not provided, but the Designer should obtain pertinent information from prospective contractors.

3.2—DEFINITIONS

Active Earth Pressure—Lateral pressure resulting from the retention of the earth by a structure or component that is tending to move away from the soil mass.

Active Earth Wedge—Wedge of earth with a tendency to become mobile if not retained by a structure or component.

Aeroelastic Vibration—Periodic, elastic response of a structure to wind.

Apparent Earth Pressure—Lateral pressure distribution for anchored walls constructed from the top down.

Axle Unit—Single axle or tandem axle.

Berm—An earthwork used to redirect or slow down impinging vehicles or vessels and to stabilize fill, embankment, or soft ground and cut slopes.

Centrifugal Force—A lateral force resulting from a change in the direction of a vehicle's movement.

Damper—A device that transfers and reduces forces between superstructure elements and/or superstructure and substructure elements, while permitting thermal movements. The device provides damping by dissipating energy under seismic, braking or other dynamic loads.

Deep Draft Waterways—A navigable waterway used by merchant ships with loaded drafts of 14–60+ ft.

Design Lane—A notional traffic lane positioned transversely on the roadway.

Design Thermal Movement Range—The structure movement range resulting from the difference between the maximum design temperature and minimum design temperature as defined in Article 3.12.

Design Water Depth—Depth of water at mean high water.

Distortion—Change in structural geometry.

Dolphin—Protective object that may have its own fender system and that is usually circular in plan and structurally independent from the bridge.

Dynamic Load Allowance—An increase in the applied static force effects to account for the dynamic interaction between the bridge and moving vehicles.

Equivalent Fluid—A notional substance whose density is such that it would exert the same pressure as the soil it is seen to replace for computational purposes.

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Section 3: Loads and Load Factors **3-3**

3.3 – Notation

3.3.1 – General

A	= plan area of ice floe (ft^2); depth of temperature gradient (in.) (C3.9.2.3) (3.12.3)
AEP	= apparent earth pressure for anchored walls (ksf) (3.4.1)
AF	= annual frequency of bridge element collapse (number/yr.) (C3.14.4)
a	= length of uniform deceleration at breaking (ft); truncated distance (ft); average bow damage length (ft) (C3.6.4) (C3.9.5) (C3.14.9)
a_B	= bow damage length of standard hopper barge (ft) (3.14.11)
a_s	= bow damage length of ship (ft) (3.14.9)
A_S	= peak seismic ground acceleration coefficient modified by short-period site factor (3.10.4.2)
B	= notional slope of backfill (degrees) (3.11.5.8.1)
B'	= equivalent footing width (ft) (3.11.6.3)
B_e	= width of excavation (ft) (3.11.5.7.2b)
B_M	= beam (width) for barge, barge tows, and ship vessels (ft) (C3.14.5.2.3)
B_p	= width of bridge pier (ft) (3.14.5.3)
BR	= vehicular braking force; base rate of vessel aberrancy (3.3.2) (3.14.5.2.3)
b	= braking force coefficient; width of a discrete vertical wall element (ft) (C3.6.4) (3.11.5.6)
b_f	= width of applied load or footing (ft) (3.11.6.3)
C	= coefficient to compute centrifugal forces; constant for terrain conditions in relation to wind approach (3.6.3) (C3.8.1.1)
C_a	= coefficient for force due to crushing of ice (3.9.2.2)
C_D	= drag coefficient (s^2 lbs./ft^4) (3.7.3.1)
C_H	= hydrodynamic mass coefficient (3.14.7)
C_L	= lateral drag coefficient (C3.7.3.1)
C_n	= coefficient for nose inclination to compute F_b (3.9.2.2)
C_{sm}	= elastic seismic response coefficient for the mth mode of vibration (3.10.4.2)

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3.3.2 – Load and Load Design **3-8**

The following permanent and transient loads and forces shall be considered:

- Permanent Loads

CR	= force effects due to creep
DD	= downdrag force
DC	= dead load of structural components and nonstructural attachments
DW	= dead load of wearing surfaces and utilities
EH	= horizontal earth pressure load
EL	= miscellaneous locked-in force effects resulting from the construction process, including jacking apart of cantilevers in segmental construction
ES	= earth surcharge load
EV	= vertical pressure from dead load of earth fill
PS	= secondary forces from post-tensioning
SH	= force effects due to shrinkage

- Transient Loads

BL	= blast loading
BR	= vehicular braking force
CE	= vehicular centrifugal force
CT	= vehicular collision force
CV	= vessel collision force
EQ	= earthquake load
FR	= friction load
IC	= ice load
IM	= vehicular dynamic load allowance
LL	= vehicular live load
LS	= live load surcharge
PL	= pedestrian live load
SE	= force effect due to settlement
TG	= force effect due to temperature gradient
TU	= force effect due to uniform temperature
WA	= water load and stream pressure
WL	= wind on live load
WS	= wind load on structure

Basic Information

- Primary AASHTO Code Information
  - Chapter 6 – Steel Structures
- Outline for superstructure design steps given in Appendix C6.

Load Factors and Combinations

- Strength I
  - Load combination relating to the normal vehicular use without wind.
- Strength II
  - Combination relating to the use of the bridge by special design vehicles and permit vehicles
- Strength III
  - Combination relating to the bridge exposed to wind velocity exceeding 55 mph.
- Strength IV
  - Combination relating to very high dead to live load force effect ratios. Typically spans > 200'.
- Strength V
  - Combination relating to normal vehicular use of the bridge with wind of 55 mph velocity.

Load Factors and Combinations

- Service I
  - Combination relating to the normal operational use of the bridge with a 55 mph wind and all loads taken at their nominal values.
  - Used for deflections and settlement calculations
- Service II
  - Load combination intended to control yielding and permanent deformation of steel structures.
  - Design of slip critical bolted connections.
- Fatigue
  - **Fatigue** and fracture load combination relating to repetitive gravitational vehicular live load and dynamic responses under a single design truck

Load Factors and Combinations

Typical Strength Design Practice per AISC Manual

“For components not traditionally governed by wind force effects, the Strengths III and V Load Combinations should not govern. Unless Strengths II and IV as indicated above are needed, for a typical multi-girder highway overpass the Strength I Load Combination will generally be the only combination requiring design calculations.”

Factored Force Effect

General Equation (3.4.1):

$Q = \sum \eta_i y_i Q_i \leq \emptyset R$

η_i = load modifier per Article 1.3.2

Q_i = force effects

y_i = load factors (Table 1 and 2)

$\emptyset$ = resistance factor

R = nominal resistance

Resistance Factor

Given in AASHTO 6.5.4.2

6.5.4.2 Resistance Factors

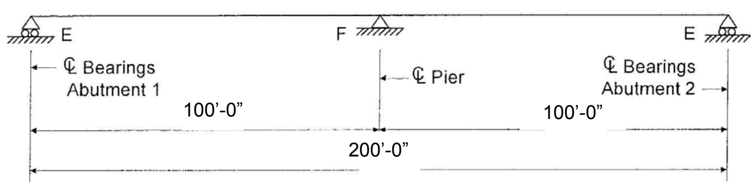
Resistance factors, ϕ, for the strength limit state shall be taken as follows:

- For flexure $\phi_f = 1.00$
- For shear $\phi_v = 1.00$
- For axial compression, steel only $\phi_c = 0.90$
- For axial compression, composite $\phi_c = 0.90$
- For tension, fracture in net section $\phi_u = 0.80$
- For tension, yielding in gross section $\phi_y = 0.95$
- For bearing on pins in reamed, drilled or bored holes and on milled surfaces $\phi_b = 1.00$
- For bolts bearing on material $\phi_{bb} = 0.80$
- For shear connectors $\phi_{sc} = 0.85$
- For A 325 and A 490 bolts in tension $\phi_t = 0.80$
- For A 307 bolts in tension $\phi_t = 0.80$
- For F 1554 bolts in tension $\phi_t = 0.80$
- For A 307 bolts in shear $\phi_s = 0.75$
- For F 1554 bolts in shear $\phi_s = 0.75$
- For A 325 and A 490 bolts in shear $\phi_s = 0.80$
- For block shear $\phi_{bs} = 0.80$
- For web crippling $\phi_w = 0.80$

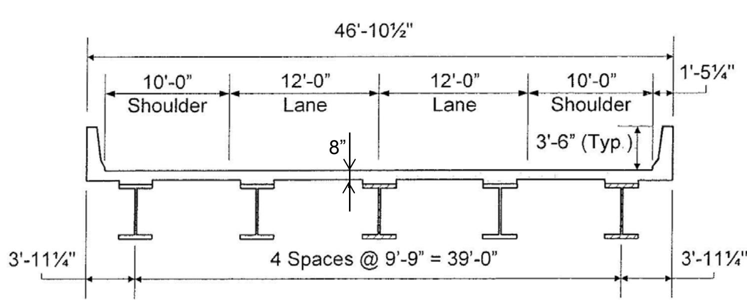
C6.5.4.2

Base metal ϕ as appropriate for resistance under consideration.

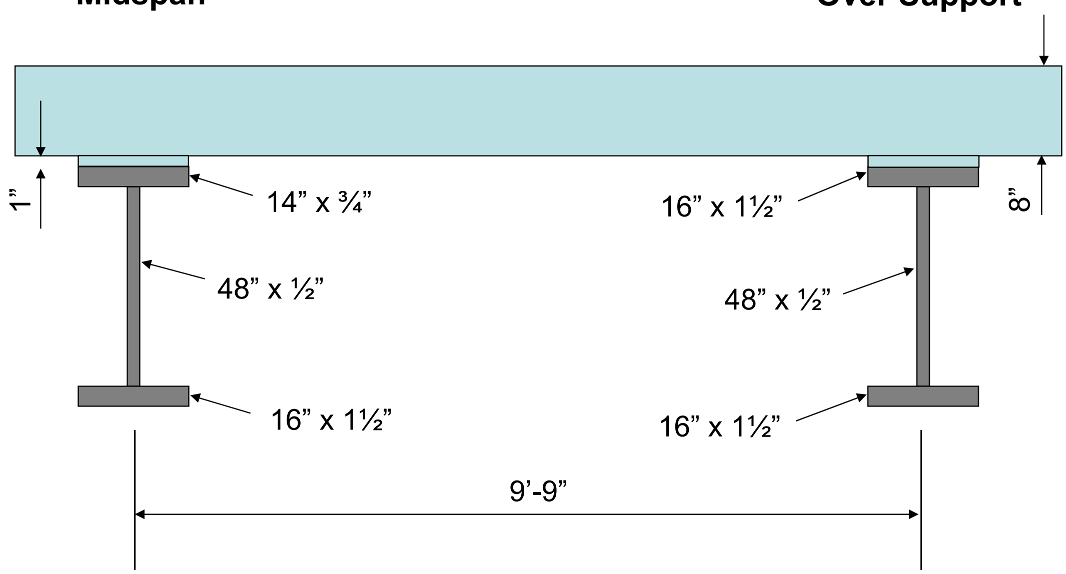
Typical Sections



Legend:
E = Expansion Bearings
F = Fixed Bearings



Typical Sections



Typical Sections

- Givens:
  - $Fy = 50$ ksi
    - $E_s = 29{,}000$ ksi AASHTO 6.4.1
  - $f'c = 4$ ksi
    - $E_c = 3{,}605$ ksi AASHTO 5.4.2.4
  - $\eta_D = \eta_R = \eta_I = 1.0$
  - Wearing Surface = 0.5"
  - Future Wearing Surface = 0.035 ksf
  - Analyze w/o longitudinal stiffners

Web Geometry

- Web Thickness
  - w/o longitudinal stiffeners
  - $D/t_w \leq 150$ Eq. 6.10.2.1.1-1
  - With longitudinal stiffeners
  - $D/t_w \leq 300$ Eq. 6.10.2.1.2-1

Web Geometry

- Web Thickness
  - $48"/0.5" = 96 \leq 150$ Therefore OK

Flange Geometry

- $b_f/2t_f \leq 12.0$ Eq. 6.10.2.2-1
  - Prevents the flange from distortion due to welding.
- $b_f \geq D/6.0$ Eq. 6.10.2.2-2
  - Flanges below this limit have less flexural and shear resistance than equations indicate.
- $t_f > 1.1t_w$ Eq. 6.10.2.2-3
  - Ensures that some restraint will be provided by the flanges against web shear buckling.

Flange Geometry

- $0.1 \leq I_{yc}/I_{yt} \leq 10$ Eq. 6.10.2.2-4
  - I_{yc} = moment of inertia of compression flange about the vertical axis in the plane of the web.
  - I_{yt} = moment of inertia of tension flange about the vertical axis in the plane of the web.

  - A section outside this limit acts more like a tee section than an I.

Flange Geometry

$b_f/2t_f = 14"/(2 \times 0.75") = 9.3 \leq 12.0$

$= 16"/(2 \times 1.5") = 5.3 \leq 12.0$

$b_f \geq D/6.0$

$14" \geq 48"/6.0 = 8"$

$t_f > 1.1t_w$

$0.75 > 1.1 \times 0.5 = 0.55"$

$1.50 > 1.1 \times 0.5 = 0.55"$

Therefore OK

Flange Geometry

- $0.1 \leq I_{yc}/I_{yt} \leq 10$ Eq. 6.10.2.2-4
  - For section over support $I_{yc}/I_{yt} = 1.0$, therefore OK
  - For midspan section
    - $I_{yc} = t_c \times b_c^3/12 = 0.75" (14")^3/12 = 171.5\ in^4$
    - $I_{yt} = 1.5" (16")^3/12 = 512.0\ in^4$
      - $I_{yc}/I_{yt} = 171.5/512.0 = 0.35$
      - $0.1 \leq 0.35 \leq 10$, therefore OK

Section Properties

Compute Section Properties over Support

Member	t (in)	w (in)	A (in^2)	d (in)	Ad	Ad^2	I_o	I	
Bott Flg	1.5	16	24	0.75	18	13.5	4.5	18	
Web	48	0.5	24	25.5	612	15606	4608	20,214	
Top Flg	1.5	16	24	50.25	1206	60601.5	4.5	60,606	
Total			72		1836			80,838	
							-x (Ad) =	(46,818)	
	x = Ad / A =		25.5	inch				34,020	in^4

D measured from bottom of member.

x = center of gravity measured from bottom chord.

Section Properties

Compute Noncomposite Section Properties at Midspan

Member	t (in)	w (in)	A (in^2)	d (in)	Ad	Ad^2	I_o	I	
Bott Flg	1.5	16	24	0.75	18	13.5	4.5	18	
Web	48	0.5	24	25.5	612	15606	4608	20,214	
Top Flg	0.75	14	10.5	49.875	523.6875	26118.91	0.492188	26,119	
Total			58.5		1153.688			46,351	
							-x (Ad) =	(22,752)	
	x = Ad / A =		19.72115	inch				23,599	in^4

D measured from bottom of member.

x = center of gravity measured from bottom chord.

Section Properties

Compute Composite Section Properties at Midspan

Member	t (in)	w (in)	A (in^2)	d (in)	Ad	Ad^2	I_o	I	
Bott Flg	1.5	16	24	0.75	18	13.5	4.5	18	
Web	48	0.5	24	25.5	612	15606	4608	20,214	
Top Flg	0.75	14	10.5	49.875	523.6875	26118.91	0.492188	26,119	
Slab	7.5	14.63	109.725	55	6034.875	331918.1	514.3359	332,432	
Total			168.225		7188.563			378,784	
							-x (Ad) =	(307,180)	
	x = Ad / A =		42.73	inch				71,603	in^4

Effective Slab Width $9.75' \times 12 = 117“$ AASHTO 4.6.2.6.1

$$w = 117"/n = 117/8.0 = 14.63“$$

Section Properties

Compute Composite Section Properties at Midspan

Member	t (in)	w (in)	A (in^2)	d (in)	Ad	Ad^2	I_o	I	
Bott Flg	1.5	16	24	0.75	18	13.5	4.5	18	
Web	48	0.5	24	25.5	612	15606	4608	20,214	
Top Flg	0.75	14	10.5	49.875	523.6875	26118.91	0.492188	26,119	
Slab	7.5	4.875	36.5625	55	2010.938	110601.6	171.3867	110,773	
Total			95.0625		3164.625			157,124	
							-x (Ad) =	(105,350)	
	x = Ad / A =		33.29	inch				51,774	in^4

For long term dead loads use 3*n* per AASHTO 6.10.1.1.1b

$$w = 117"/3n = 117/24.0 = 4.875“$$

Section Properties

Noncomposite at Midspan

$S_b = 23{,}599 \text{ in}^4 / 19.72" = 1{,}196.7 \text{ in}^3$
$S_t = 23{,}599 \text{ in}^4 / (50.25" - 19.72") = 773.0 \text{ in}^3$

Composite at Midspan ($n = 8$)

$S_b = 71{,}603 \text{ in}^4 / 42.73" = 1{,}675.7 \text{ in}^3$
$S_t = 71{,}603 \text{ in}^4 / (50.25" - 42.73") = 9{,}521.7 \text{ in}^3$

Composite at Midspan ($n = 24$)

$S_b = 51{,}774 \text{ in}^4 / 33.29" = 1{,}555.2 \text{ in}^3$
$S_t = 51{,}774 \text{ in}^4 / (50.25" - 33.29") = 3{,}052.7 \text{ in}^3$

Loads

Dead Loads

Deck $= 9.75' \times 0.67' \times 0.150 \text{ kcf} = 0.980$

Fillet $= 1.17' \times 0.08' \times 0.150 \text{ kcf} = 0.014$

Beam $(16{\times}1.5 + 48{\times}0.5 + 15{\times}1.13 \text{ ave})/144$

$\times 0.490 \text{ kcf} = 0.221$

Misc. Steel $= 10\% \times 0.221 = 0.022$

Noncomposite DC $= 1.237 \text{ klf}$

Loads

Rails $= 2 \times 0.570 \text{ klf}/5 \text{ girders} = 0.228 \text{ klf}$

Also medians, sidewalks, etc.

Composite DC $= 0.228 \text{ klf}$

$FWS = 0.035 \text{ ksf} \times 44'/5 \text{ girders} = 0.308 \text{ klf}$

Also other future dead loads.

Composite DW $= 0.305 \text{ klf}$

Loads

Live loads consist of HL-93 which is a combination of lane load and either truck or tandem loading. AASHTO 3.6.1.2

90% of two design trucks used for negative moments over supports.

Loads determined by linear analysis (or influence lines) which were described in Bridge Loads session.

This results in reactions/moments/shears per lane depending on influence lines used.

Loads

Distribution factors are then computed per AASHTO 4.6.2.2.

We will compute factors at midspan.

Live Load Distribution

Table 4.6.2.2.2b-1 Distribution of Live Loads Per Lane for *Moment in Interior Beams*.			
Type of Superstructure	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	One Design Lane 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: $\left(1.75 + \frac{S}{3.6}\right)\left(\frac{1}{L}\right)^{0.35}\left(\frac{1}{N_c}\right)^{0.45}$	$7.0 \leq S \leq 13.0$ $60 \leq L \leq 240$ $N_c \geq 3$

Live Load Distribution

Check the range of applicability

$3.5 \leq S \leq 16.0$

$S = 9.75$ ft OK

$4.5 \leq ts \leq 12.0$

$\text{ts} = 8.0$ in OK

$20 \leq L \leq 240$

$\text{L} = 100$ ft OK

$Nb \geq 4$

$\text{Nb} = 5$ OK

$10{,}000 \leq K_g \leq 7{,}000{,}000$

Live Load Distribution

Compute K_g at Midspan

$K_g = n\left(I + A \cdot e_g^2\right)$

$n = E_b/E_D = 29{,}000/3{,}605 = 8$

where:

I = moment of inertia of beam (in.4)

e_g = distance between the centers of gravity of the beam and deck (in.)

A = area of the beam (in.2)

$K_g = 8(23{,}559\ in^4 + 58.5\ in^2 \times (55.0'' - 42.73'')^2)$

$= 258{,}930\ in^4$ within range therefore OK

Live Load Distribution

One Design Lane Loaded:

$$0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$

$$= 0.06 + (9.75/14)^{0.4}(9.75/100)^{0.3}\left[258{,}930/\left(12(100)(8)\right)^3\right]^{0.1}$$

$$= 0.06 + (0.87)(0.50)(0.92) = 0.40$$

Therefore 1 beam carries 0.40 lanes of LL.

Do not convert units, already included in equations.

Live Load Distribution

Two or More Design Lanes Loaded:

$$0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$$

$$= 0.075 + (9.75/9.5)^{0.6}(9.75/100)^{0.2}[258{,}930/(12(100)(8)^3)]^{0.1}$$

$$= 0.075 + (1.02)(0.63)(0.92) = \mathbf{0.59}$$

Since 0.59 greater than 0.40, 0.59 governs for design

Unfactored/Undistributed Moments

Span	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
DC_{nc}	0	377	639	783	812	722	518	196	-242	-796	-1467
DC_{comp}	0	75	126	156	161	144	193	38	-39	-129	-238
DW	0	61	105	127	132	118	85	32	-48	-158	-292
+LL+IM	0	800	1356	1683	1827	1792	1599	1234	728	271	0
-LL+IM	0	-107	-212	-319	-425	-531	-638	-744	-914	-1183	-1910

All moments in ft-kips.

IM = 33% of LL for this case.

Strength I Moment at 0.4 Point

$$LL + IM = 1.75 \times 1{,}827 \times 0.59 = 1{,}890 \text{ ft} - \text{kip}$$

$$DC_{non} = 1.25 \times (812) = 1{,}015 \text{ ft} - \text{kip}$$

$$DC_{comp} = 1.25 \times (161) = 201 \text{ ft} - \text{kip}$$

$$DW = 1.5 \times 132 = 198 \text{ ft} - \text{kip}$$

$$Mu = 3{,}304 \text{ ft} - \text{kip}$$

Check Capacity per 6.10.1.1.1

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D6.2.2 Composite Sections in Positive Flexure

The yield moment of a composite section in positive flexure shall be taken as the sum of the moments applied separately to the steel and the short-term and long-term composite sections to cause nominal first yielding in either steel flange at the strength limit state. Flange lateral bending in all types of sections and web yielding in hybrid sections shall be disregarded in this calculation.

The yield moment of a composite section in positive flexure may be determined as follows:

- Calculate the moment M_{D1} caused by the factored permanent load applied before the concrete deck has hardened or is made composite. Apply this moment to the steel section.
- Calculate the moment M_{D2} caused by the remainder of the factored permanent load. Apply this moment to the long-term composite section.
- Calculate the additional moment M_{AD} that must be applied to the short-term composite section to cause nominal yielding in either steel flange.
- The yield moment is the sum of the total permanent load moment and the additional moment.

Check Capacity per 6.10.1.1.1

Symbolically, the procedure is:

1. Solve for M_{AD} from the equation:

$$F_{yf} = \frac{M_{D1}}{S_{NC}} + \frac{M_{D2}}{S_{LT}} + \frac{M_{AD}}{S_{ST}} \qquad \text{(D6.2.2-1)}$$

2. Then calculate:

$$M_y = M_{D1} + M_{D2} + M_{AD} \qquad \text{(D6.2.2-2)}$$

Where:

S_{NC} = noncomposite section modulus (in^3)

S_{ST} = short-term composite section modulus (in^3)

S_{LT} = long-term composite section modulus (in^3)

M_{D1}, M_{D2} & M_{AD} = moments due to the factored loads applied to the appropriate sections (kip-in)

M_y shall be taken as the lesser value calculated for the compression flange, M_{yc}, or the tension flange, M_{yt}.

Check Capacity per 6.10.1.1.1

Check Bottom Flange

$F_{yt} = 50\text{ ksi} = 1{,}015'\text{ k} \times 12/1{,}196.7\text{ in}^3 + (201'\text{ k} + 198'\text{ k}) \times 12/1{,}555.2\text{ in}^3 + M_{AD} \times 12/1{,}675.7\ \text{ in}^3$

$M_{AD} = (50\text{ ksi} - 10.18\text{ ksi} - 3.08\text{ ksi}) \times 1{,}675.7/12 = 5{,}129'\text{ k}$

$My = 1{,}015'\text{ k} + 201'\text{ k} + 198'\text{ k} + 5{,}129'\text{ k} = 6{,}543'\text{ k}$

Check Capacity per 6.10.1.1.1

Check Top Flange

$F_{yt} = 50\text{ ksi} = 1{,}015'\text{ k} \times 12/773.0\text{ in}^3 + (201'\text{ k} + 198'\text{ k}) \times 12/3{,}052.7\text{ in}^3 + M_{AD} \times 12/1{,}675.7\ \text{ in}^3$

$M_{AD} = (50\text{ ksi} - 15.76\text{ ksi} - 1.57\text{ ksi}) \times 9{,}521.7/12 = 25{,}923'\text{ k}$

$My = 1{,}015'\text{ k} + 201'\text{ k} + 198'\text{ k} + 25{,}923'\text{ k} = 27{,}136'\text{ k}$

Check Capacity per 6.10.1.1.1

$6{,}543 < 27{,}136$, therefore use $Fy = 6{,}543'\ \mathrm{k}$

$R = \Phi 6{,}543'\ \mathrm{k} = 1.0 \times 6{,}543'\ \mathrm{k} > 3{,}304'\ \mathrm{k}$

Therefore OK

Plastic Moment of Inertia

$Ps = 0.85 f_c' b_s t_s = 0.85 \times 4 \times 114 \times 7.5 = 2{,}907\ \mathrm{k}$

$Pc = F_{yc} b_c t_c = 50 \times 14" \times 0.75 = 525\ \mathrm{k}$

$Pw = F_{yw} b_w t_w = 50 \times 0.5 \times 48 = 1{,}200\ \mathrm{k}$

$Pr = F_{yt} b_t t_t = 50 \times 16 \times 1.5 = 1{,}200\ \mathrm{k}$

Ignore reinforcing and fillet, conservative

$Ps + Pc \geq Pw + Pt$

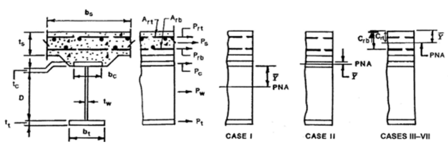
PNA in top flange

Plastic Moment of Inertia

6-290 AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS

Table D6.1-1 Calculation of $\overline{Y}$ and M_p for Sections in Positive Flexure.

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



Plastic Moment of Inertia

6-290 AASHTO LRFD Bridge Design Specifications

Table D6.1-1 Calculation of $\overline{Y}$ and M_P for Sections in Positive Flexure.			
Case	PNA	Condition	**$\overline{Y}$ and M_P**
I	In Web	$P_t + P_w \geq P_c + P_s + P_{rb} + P_{rt}$	$\overline{Y} = \left(\frac{D}{2}\right)\left[\frac{P_t - P_c - P_s - P_{rt} - P_{rb}}{P_w} + 1\right]$ $M_P = \frac{P_w}{2D}[\overline{Y}^2 + (D-\overline{Y})^2] + [P_s d_s + P_{rt} d_{rt} + P_{rb} d_{rb} + P_c d_c + P_t d_t]$
II	In Top Flange	$P_t + P_w + P_c \geq P_s + P_{rb} + P_{rt}$	$\overline{Y} = \left(\frac{t_c}{2}\right)\left[\frac{P_w + P_t - P_s - P_{rt} - P_{rb}}{P_c} + 1\right]$ $M_P = \frac{P_c}{2t_c}[\overline{Y}^2 + (t_c-\overline{Y})^2] + [P_s d_s + P_{rt} d_{rt} + P_{rb} d_{rb} + P_w d_w + P_t d_t]$

$$Y = (0.75/2)[(1{,}200 + 1{,}200 - 2{,}907)/525 + 1] = 0.01\text{“}$$

$$M_P = (525/2 \times 0.75)[0.01"^2 + (0.75" - 0.01")^2] +$$

$$2{,}907\text{ k} \times (0.01" + 7.5"/2) + 1{,}200\text{ k} \times (0.74" + 48"/2) +$$

$$1{,}200\text{ k} \times (0.74{+}48 + 1.5"/2) = 100{,}198/12 = 8{,}350\text{ ft} - \text{kip}$$

Check for Compact Section

$F_{yf} = 50\ ksi \leq 70\ ksi$ AASHTO 6.10.6.2.2

- The web satisfies the requirement of Article 6.10.2.1.1,

And:

- The section satisfies the web slenderness limit:

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

Where:

D_{cp} = depth of the web in compression at the plastic moment determined as specified in Article D6.3.2 (in.)

Check for Compact Section

6.10.2.1.1 checked on slide 15.

In positive moment area PNA in top flange, therefore $D_{cp} = 0$ and eq. 6.10.6.2.2-1 satisfied.

Check web at int. support per eq. 6.10.6.2.3-1. Symmetric Section.

$$D_{cp} = 48''/2 = 24''$$

$$2 \times 24''/0.5'' = 96$$

$$5.7 \times \text{sqrt}(29{,}000\ \text{ksi}/50\ \text{ksi}) = 137$$

Therefore neg. moment section is also compact.

Check for Compact Section

6.10.7 Flexural Resistance – Composite Sections in Positive Flexure

6.10.7.1 Compact Sections

6.10.7.1.1 General

At the strength limit state, the section shall satisfy:

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

Where:

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2

f_ℓ = flange lateral bending stress determined as specified in Article 6.10.1.6 (ksi)

M_n = nominal flexural resistance of the section determined as specified in Article 6.10.7.1.2 (kip-in)

M_u = bending moment about the major-axis of the cross-section determined as specified in Article 6.10.1.6 (kip-in)

M_{yt} = yield moment with respect to the tension flange determined as specified in Article D6.2 (kip-in)

S_{xt} = elastic section modulus about the major axis of the section to the tension flange taken as M_{yt}/F_{yt} (in^3)

Check for Compact Section

Check $D_p \leq 0.1 D_t$ 6.10.7.1.2

D_p = distance from the top of the concrete deck to the neutral axis of the composite section at the plastic moment (in)

D_t = total depth of the composite section (in)

$D_p = 7.5'' + 0.01'' = 7.51''$

$D_t = 1.5'' + 48'' + 0.75'' + 1'' + 7.5'' = 58.75''$

$0.1 \times 58.75'' = 5.87'' < 7.51''$

Therefore…

Check for Compact Section

$$M_n = M_p\left(1.07 - 0.7\frac{D_p}{D_t}\right) \qquad (6.10.7.1.2\text{-}2)$$

$$M_n = 8{,}350'\ \text{k}(1.07 - 0.7 \times 7.51''/58.75'') = 8{,}187'\ \text{k}$$

Since there’s no lateral bending for the straight girders, the left side of equation 6.10.7.1.1-1 simplifies to only the maximum moment.

$8{,}187'\ \text{k} > 3{,}304'\ \text{k}$ therefore OK

Chapter 6 Appendices

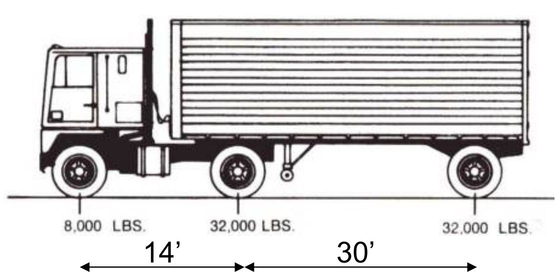
- App. A & B – Alternate design methods to increase allowable capacity.
- App. C – Quick Reference Outline of Basic Steel Superstructure Steps w/ Code Refs.
  - Includes Flow Charts
- App. D – Fundamental Calculations
  - Includes Plastic Moment

Steel Fatigue

- Similar to AISC Code
- Number of Cycles by calculation
- Illustrative Examples Figure 6.6.1.2.3-1
- Fatigue Category from Table 6.6.1.2.3-1
- Allowable Fatigue Thresholds Table 6.6.1.2.5-3

Steel Fatigue

- 3.6.1.4 Fatigue Load
  - Special Fatigue Truck is one design truck with a constant spacing of 30.0 ft. between the 32.0-kip axles with IM = 15%.
- No lane component



Steel Fatigue

SECTION 6: STEEL STRUCTURES 6-278.1

C6.4.3 Flowchart for LRFD Article 6.10.5

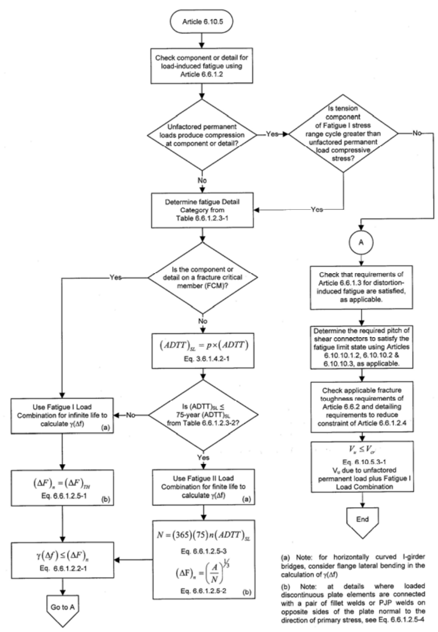


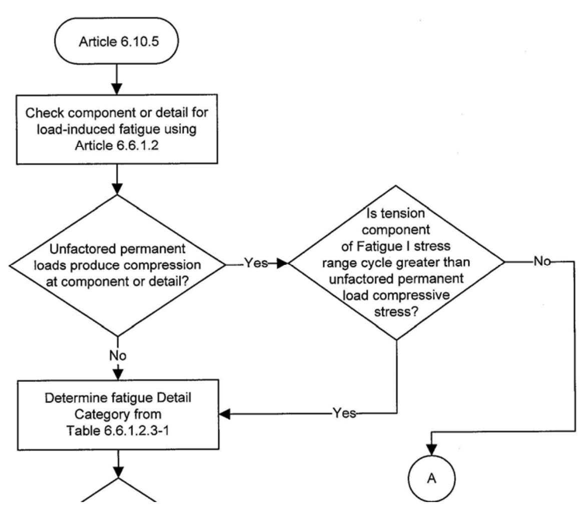
Figure C6.4.3-1 Flowchart for LRFD Article 6.10.5—Fatigue and Fracture Limit State.

ASCE | KNOWLEDGE & LEARNING

Steel Fatigue

SECTION 6: STEEL STRUCTURES 6-278.1

C6.4.3 Flowchart for LRFD Article 6.10.5



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Steel Fatigue

The frequency of the fatigue load shall be taken as the single-lane average daily truck traffic ($ADTT_{SL}$). This frequency shall be applied to all components of the bridge.

In the absence of better information, the single-lane average daily truck traffic shall be taken as:

$$ADTT_{SL} = p \times ADTT \qquad (3.6.1.4.2\text{-}1)$$

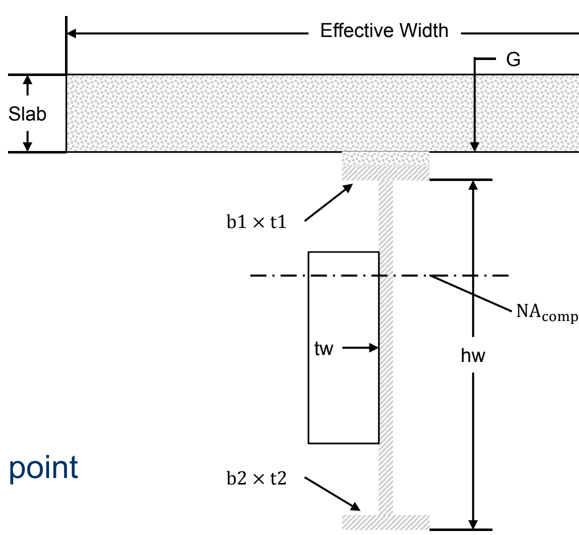
where:

$ADTT$ = the number of trucks per day in one direction averaged over the design life

Table 3.6.1.4.2-1 Fraction of Truck Traffic in a Single Lane, *p*.

Number of Lanes Available to Trucks	*p*
1	1.00
2	0.85
3 or more	0.80

Steel Fatigue



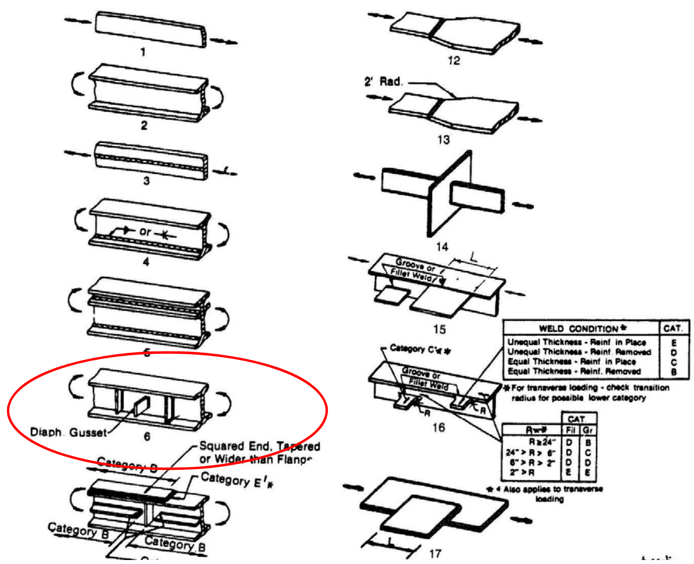
$f_f = M_r \times C / I$

M_r = moment range

c = dist. From NA to point considered

Steel Fatigue

Figure 6.6.1.2.3-1 Illustrative Examples



Steel Fatigue

From Figure 6.6.1.2.3-1 Detail Categories

General Condition	Situation	Detail Category	Illustrative Example, See Figure 6.6.1.2.3-1
Fillet-Welded Connections with Welds Normal to the Direction of Stress	Base metal: ▪ At details other than transverse stiffener-to-flange or transverse stiffener-to-web connections	Lesser of C or Eq. 6.6.1.2.5-3	14
	▪ At the toe of transverse stiffener-to-flange and transverse stiffener-to-web welds	C'	6
Fillet-Welded Connections with Welds Normal and/or Parallel to the Direction of the Stress	Shear stress on the weld throat Base Metal at end of weld	E	9

Steel Fatigue

Nominal Fatigue Resistance (6.6.1.2.5)

$(\Delta F)_n = (A/N)^{1/3}$

$$N = (365)(75)n(ADTT)_{SL}$$

where:

A = constant taken from Table 1 (ksi^3)

n = number of stress range cycles per truck passage taken from Table 2

$(ADTT)_{SL}$ = single-lane *ADTT* as specified in Article 3.6.1.4

$(\Delta F)_{TH}$ = constant-amplitude fatigue threshold taken from Table 3 (ksi)

Steel Fatigue

Table 6.6.1.2.5-1 Detail Category Constant, *A*.

Detail Category	Constant, *A* Times 10^8 (ksi3)
A	250.0
B	120.0
B’	61.0
C	44.0
C’	44.0
D	22.0
E	11.0
E’	3.9
M 164 (A 325) Bolts in Axial Tension	17.1
M 253 (A 490) Bolts in Axial Tension	31.5

Table 6.6.1.2.5-2 Cycles per Truck Passage, *n*.

Longitudinal Members	Span Length	
	> 40.0 ft	≤ 40.0 ft
Simple Span Girders	1.0	2.0
Continuous Girders		
1. Near interior support	1.5	2.0
2. Elsewhere	1.0	2.0
Cantilever Girders	5.0	
Trusses	1.0	
Transverse Members	Spacing	
	> 20.0 ft	≤ 20.0 ft
	1.0	2.0

Steel Fatigue

Table 6.6.1.2.5-3 Constant Amplitude Fatigue Thresholds	
Detail Category	Threshold (ksi)
A	24.0
B	16.0
B'	12.0
C	10.0
C'	12.0
D	7.0
E	4.5
E'	2.6
M 164 (A 325) Bolts in Axial Tension	31.0
M 253 (A 490) Bolts in Axial Tension	38.0

Problem 1

Find the nominal fatigue resistance for a gusset plate welded to a 120' simple span girder web.

$ADTT = 1{,}500 \text{ vpd}$

2 lanes available to trucks

Solution 1

$(ADTT)_{SL} = 0.85 \times 1{,}500 = 1{,}275 \text{ vpd}$

$N = (365)(75)(1.0)(1{,}275) = 34.9 \times 10^6$

$(\Delta F)_n = (A/N)^{1/3}$

$= (44.0 \times 10^8 / 34.9 \times 10^6)^{1/3}$

$= 5.01 \text{ ksi}$

Therefore, use 5.0 ksi allowable

Problem 2

Design Data:

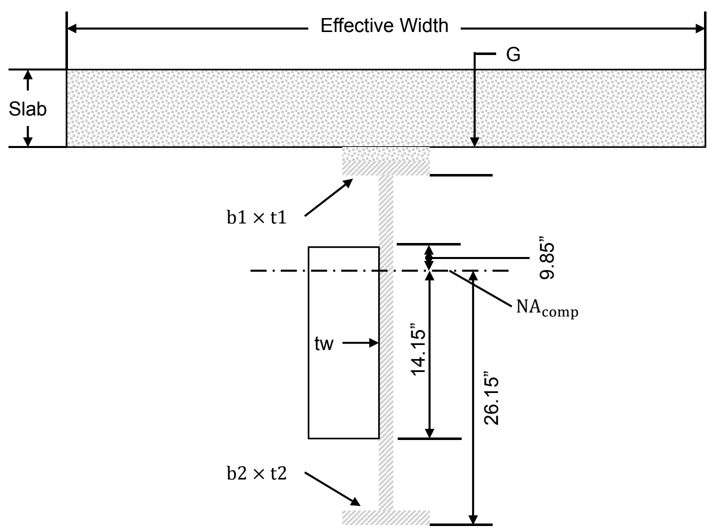
$I_c = 12{,}215 \text{ in}^4$

$C_b = 26.15“$

$M_{DL} = -93 \text{ ft} - \text{kip}$

$M_{LL+IM} = 307 \text{ ft} - \text{kip}$

$M_{LL+IM} = -149 \text{ ft} - \text{kip}$



What is the critical fatigue stress at the end of the gusset plate?

(A) 7.63 ksi
(B) 6.34 ksi
(C) 4.27 ksi
(D) 4.41 ksi

Solution 2

Problem 6

(B) Moment Range $= 307 - (-149) = 456\ \text{ft} - \text{k}$

$$f_f = M_r \times c/I$$

$$= 456\ \text{ft} - \text{k} \times 12"/\text{ft} \times 14.5"/12{,}215\ \text{in}^4$$

$$= 6.34\ \text{ksi}$$

Splice Design

Span	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
DC_{nc}	0	377	639	783	812	722	518	196	-242	-796	-1467
DC_{comp}	0	75	126	156	161	144	193	38	-39	-129	-238
DW	0	61	105	127	132	118	85	32	-48	-158	-292
+LL+IM	0	800	1356	1683	1827	1792	1599	1234	728	271	0
-LL+IM	0	-107	-212	-319	-425	-531	-638	-744	-914	-1183	-1910

Unfactored/Undistributed Moments

All moments in ft-kips.

$IM = 33\%$ of LL for this case.

Splice Design

Assume:

Noncomposite design

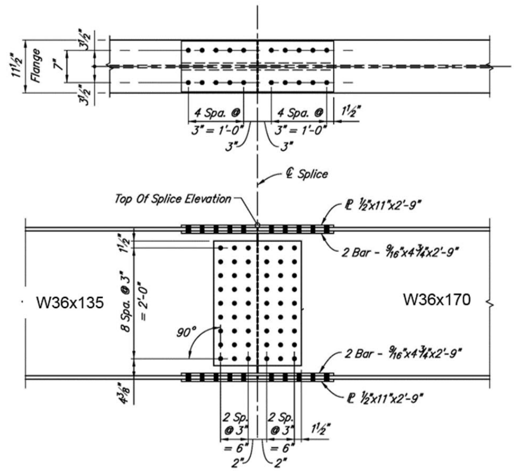
$Fy = 50\ \text{ksi}$

Resistance factor, $\Phi_f = 1.0$

Strength I design

Compute:

The minimum factored flexural resistance for the splice design.



Splice Design

Strength I Moment at 0.8 Point

$$LL + IM = 1.75 \times 914 \times 0.59 = 944\ \text{ft} - \text{kip}$$

$$DC_{non} = 1.25 \times (242) = 303\ \text{ft} - \text{kip}$$

$$DC_{comp} = 1.25 \times (39) = 49\ \text{ft} - \text{kip}$$

$$DW = 1.5 \times 48 = 72\ \text{ft} - \text{kip}$$

$$Mu = 1{,}368\ \text{ft} - \text{kip}$$

Splice Design

AASHTO 6.13.1

Except as specified otherwise, connections and splices for primary members shall be designed at the strength limit state for not less than the larger of:

- The average of the flexural moment-induced stress, shear, or axial force due to the factored loadings at the point of splice or connection and the factored flexural, shear, or axial resistance of the member or element at the same point, or
- 75 percent of the factored flexural, shear, or axial resistance of the member or element.

Splice Design

Use the smaller of the two sections.

W36 x 135 has $Sx = 439\ \text{in}^3$

$M_{rx} = \Phi_f \times Fy \times Sx$

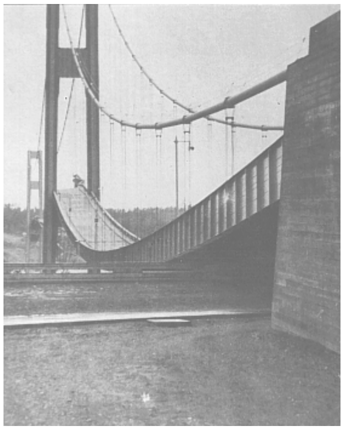
$$= 1.0 \times 50\ \text{ksi} \times 439\ \ \text{in}^3/12 = 1{,}829'\ \text{k}$$

Average $= (1{,}368'\ \text{k} + 1{,}829'\ \text{k})/2 = 1{,}599'\ \text{k}$

75% $M_{rx} = 0.75 \times 1{,}829'\ \text{k} = 1{,}372'\ \text{k}$

Therefore, design for 1,599’ k

Deflection



- AASHTO 2.5.2.6.2
  - Criteria optional except for orthotropic, metal decks or 3-sided box structures.
  - Deflection due to service live load plus impact shall not exceed 1/800 of the span (1/1000 with sidewalks).
  - When investigating the maximum absolute deflection for straight girder systems, all design lanes should be loaded, and all supporting components should be assumed to deflect equally;

Deflection

- AASHTO 2.5.2.6.2
  - For composite design, the stiffness of the design cross-section used to determine the deflection should include the entire width of the roadway and the structurally continuous portions of the railings, sidewalks, and median barriers;
  - The live load portion of Load Combination Service I should be used, including the dynamic load allowance, IM;

Deflection

- AASHTO 3.6.1.3.2
  - deflection should be taken as the larger of:
  - That resulting from the design truck alone, or
  - That resulting from 25 percent of the design truck taken together with the design lane load

Problem 3

A 120' long single span bridge is computed to have 1.51" of deflection using all the beams acting together. Does this meet AASHTO requirements for a bridge carrying only traffic?

For a bridge carrying pedestrians?

Deflection

Solution 3

Allowable deflection $= \text{Span}/800$

$(120' \times 12''/') / 800 = 1.80'' > 1.51$ OK

With Pedestrians $= Span/1{,}000$

$(120' \times 12''/') / 1{,}000 = 1.44'' < 1.51$ NG

Summary

- LRFD provisions similar to AISC and ACI
- Beware of other AASHTO versions.
- Statics are statics. Basic equations still work.
- Loads and Factors are specific to AASHTO.
- Examples available on FHWA website.

Questions

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Resources

- http://www.fhwa.dot.gov/bridge/steel/pubs/if12052/
  - Steel Design Examples
  - Based on 5th Edition with 2010 Interims
  - Loads and Load Combinations - Volume 7
  - Limit States - Volume 10
  - Design for Fatigue - Volume 12
  - Design Example: Three-span Continuous Straight I-Girder Bridge
  - Design Example: Two-span Continuous Straight I-Girder Bridge
  - Design Example: Two-span Continuous Straight Wide-Flange Beam Bridge

Resources

- http://www.fhwa.dot.gov/bridge/lrfd/examples.htm
  - Prestressed Concrete Girder Superstructure Example
  - Steel Girder Superstructure Example
  - Based on 2nd Edition and Interims through 2002
    - A number of sections have changed in the Code between 2002 and 2010 so be careful using this.
- http://www.aisc.org/contentNSBA.aspx?id=20244
  - National Steel Bridge Alliance Steel Beam and Girder Examples
  - Based on 3rd Edition and Interims through 2005

Load Combinations

Table 3.4.1-1

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	EQ (Use One of These at a Time)	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	-	-	1.00	0.50/1.20	-	-	-	-	-	-
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	γEQ	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	-	-	-	-	-	-	-	-	-	-	-

Load Combinations

Table 3.4.1-2

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