S.E. Exam Review: Bridge Loads

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Basic Information

- AASHTO LRFD Bridge Design Specifications
  - 7th Edition released in July 2014
    - 2015, 2016 and 2017 Revisions also available
  - Beware of other versions
- Strengths always in ksi

Basic Information

- Primary AASHTO Code Information
  - Chapter 2 – General Design
  - Chapter 3 – Loads
  - Chapter 9 – Decks

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
- 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.
- Service II
  - Load combination intended to control yielding of **steel** structures
- Service III
  - Load combination for longitudinal analysis relating to tension in **prestressed concrete** superstructures with the objective of crack control
- Service IV
  - Load combination relating only to tension in **prestressed concrete columns** with the objective of crack control.

Load Factors and Combinations

- Extreme Event I
  - Load combination including earthquake.
- Extreme Event II
  - Load combination relating to ice load, collision by vessels and vehicles, and certain hydraulic events with a reduced live load.
- Fatigue
  - **Fatigue** and fracture load combination relating to repetitive gravitational vehicular live load and dynamic responses under a single design truck

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

Factored Force Effect

- $0.95 \leq \eta \leq 1.05$
  - η_d relates to ductility
    - 1.05 = non, 1.00 normal, 0.95 exceptional
  - η_R relates to redundancy
    - 1.05 = non, 1.00 normal, 0.95 exceptional
  - η_I relates to importance
    - 1.05 = important, 1.00 normal, 0.95 less imp.
  - All factors multiplied to get η_i

AASHTO Loads

- Covered in Section 3
- Unit Weights (3.5.1-1)
  - Steel 0.490 kcf
  - Concrete 0.145 kcf
- Live Load (3.6.1)
  - Truck + Lane
  - Tandem + Lane
  - Combined loading is called HL93

Live Load

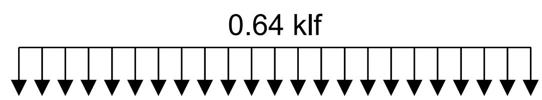
Live Load moment (shear) to a beam consists of the moment (shear), impact factor and distribution.

$$M_{tot} = M_{LL} \times (1 + IM) \times \text{Distr.}$$

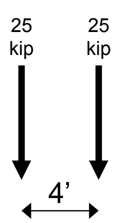
IM = Dynamic Load Allowance

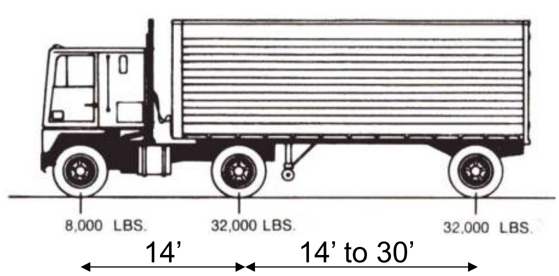
Live Loads

- One lane per 12’ width, no rounding.
  - Widths between 20’ and 24’ = 2 lanes.
- Apply Lane Load to contributory length only.
- Apply 90% of 2 Design Trucks at 50’ min. spacing and Lane Load for continuous spans for maximum moments.



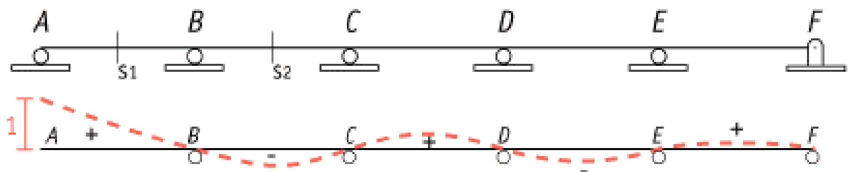
Live Loads



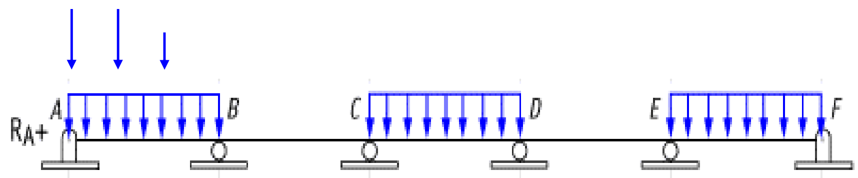


- Design Tandem is 2 axles at 25 k.
- Design Truck similar to old HS-20.
  - Rear axle can be increased to 30’ to obtain max. load.
- These used in conjunction with Lane Load unless otherwise specified.

Live Loads

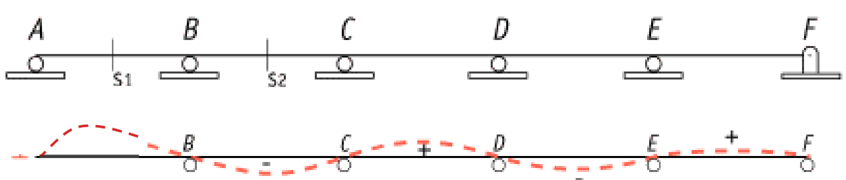


Influence Line for Reaction at Support A

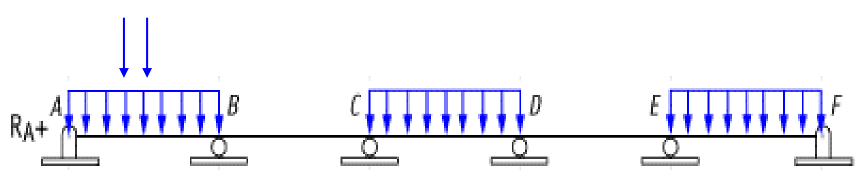


Lane Loading for Max. Reaction at Support A

Live Loads



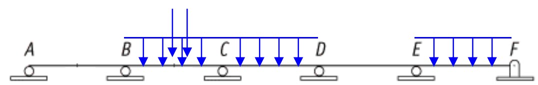
Influence Line for Moment at 0.4 Span A

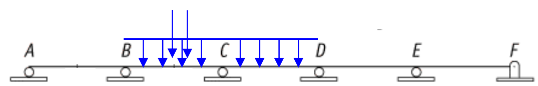


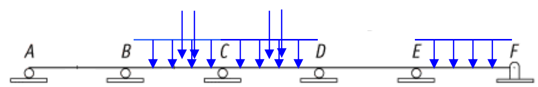
Lane Loading for Max. Moment at 0.4 Span A

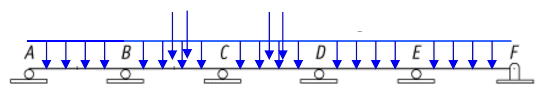
Problem 1

Which case gives the correct loading for maximum moment at support C assuming 150’ spans?

A. 

B. 

C. 

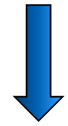
D. 

Solution

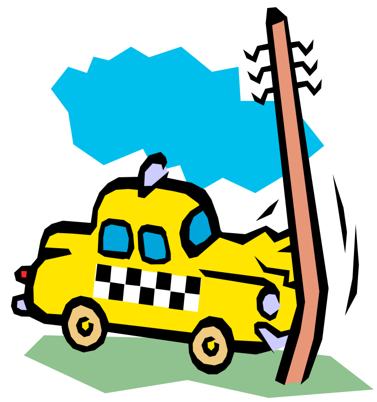
Problem 1

(C) To get the maximum negative moment you would load the adjacent spans and alternate spans further away which all have contributing areas on an influence diagram. AASHTO 3.6.1.3 says to add an additional truck at 50’ min. spacing for continuous spans.

Dynamic Load Allowance (Impact Load)



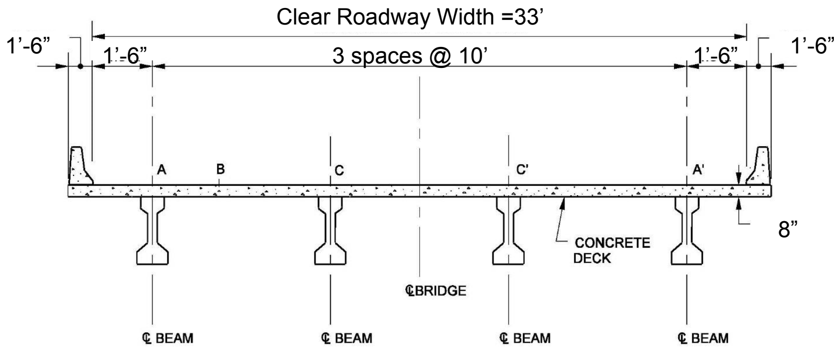
- Section 3.6.2
- Does not apply to:
  - Walls w/o vertical reactions from superstructure.
  - Foundation components entirely below ground.
  - Lane Load portion of HL-93



Dynamic Load Allowance (Impact Load)

Table 3.6.2.1-1 Dynamic Load Allowance, *IM*.	
Component	IM
Deck Joints – All Limit States	75%
All Other Components ▪ Fatigue and Fracture Limit State ▪ All Other Limit States	 15% 33%

Live Load Distribution



Problem 2: What is the maximum number of design lanes on this bridge?

(A) 0 (B) 1 (C) 2 (D) 3

Solution

Problem 2

(C) 33' clear roadway / 12' lane = 2.75.

Therefore the bridge has two full lanes.

Live Load Distribution

- Used to compute how much of the LL goes to a beam.
- 4.6.2.2 Beam-Slab Bridges
  - Dependent on type of beam
  - Different factor for interior and exterior beams
  - Different factor for moment or shear
  - Different factor for each region of bridge
  - Modifier for skewed structures
- Distribution is the percentage of lane load of live load + impact that is applied to one beam.

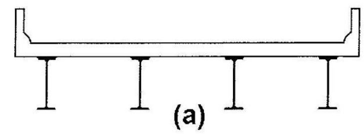
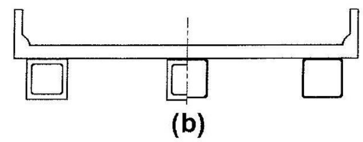
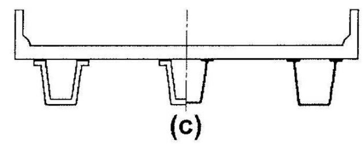
Live Load Distribution

Table C4.6.2.2.1-1 *L* for Use in Live Load Distribution Factor Equations

Force Effect	*L* (ft.)
Positive Moment	The length of the span for which moment is being calculated
Negative Moment – Near interior supports of continuous spans from point of contraflexure under a uniform load on all spans	The average length of the two adjacent spans
Negative Moment – Other than near interior supports of continuous spans	The length of the span for which moment is being calculated
Shear	The length of the span for which shear is being calculated
Exterior Reaction	The length of the exterior span
Interior Reaction of Continuous Span	The average length of the two adjacent spans

Live Load Distribution

Table 4.6.2.2.1-1 Common Deck Superstructures Covered in Articles 4.6.2.2.2 and 4.6.2.2.3.

SUPPORTING COMPONENTS	TYPE OF DECK	TYPICAL CROSS-SECTION
Steel Beam	Cast-in-place concrete slab, precast concrete slab, steel grid, glued/spiked panels, stressed wood	
Closed Steel or Precast Concrete Boxes	Cast-in-place concrete slab	
Open Steel or Precast Concrete Boxes	Cast-in-place concrete slab, precast concrete deck slab	

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 $N_h = 3$ or the lever rule	$N_h = 3$
Cast-in-Place Concrete Multicell Box	d	One Design Lane Loaded: $\left(1.75+\frac{S}{3.6}\right)\left(\frac{l}{L}\right)^{0.35}\left(\frac{l}{N_c}\right)^{0.45}$	$7.0 \leq S \leq 13.0$ $60 \leq L \leq 240$ $N_c \geq 3$

Live Load Distribution

Table 4.6.2.2.2d-1 Distribution of Live Loads Per Lane for ***Moment in Exterior Longitudinal*** 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	Lever Rule	Lever Rule	N/A
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	Lever Rule	$g = e g_{interior}$ $e = 0.77 + \frac{d_e}{9.1}$	$-1.0 \le d_e \le 5.5$
			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 Multicell Box	d	$g = \frac{W_e}{14}$	$g = \frac{W_e}{14}$	$W_e \le S$
		Or the provisions for a whole-width design specified in Article 4.6.2.2.1		

Live Load Distribution

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- and Double T-Sections	a, e, k and also 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 $\theta < 30°$ then $c_1 = 0.0$ If $\theta < 60°$ use $\theta = 60°$	$30° \le \theta \le 60°$ $3.5 \le S \le 16.0$ $20 \le L \le 240$ $N_b \ge 4$
Concrete Deck on Concrete Spread Box Beams, Cast-in-Place Multicell Box Concrete Box Beams and Double T-Sections used in Multibeam Decks	b, c, d, f, g	$1.05 - 0.25\tan\theta \le 1.0$ If $\theta > 60°$ use $\theta = 60°$	$0° \le \theta \le 60°$

Live Load Distribution

Compute the Live Load Distribution factor for an interior girder.

First, compute the longitudinal stiffness, Kg

$$Kg = n(I + A \cdot e_g^2)$$

$$n = E_B / E_D$$

where:

E_B = modulus of elasticity of beam mat'l (ksi)

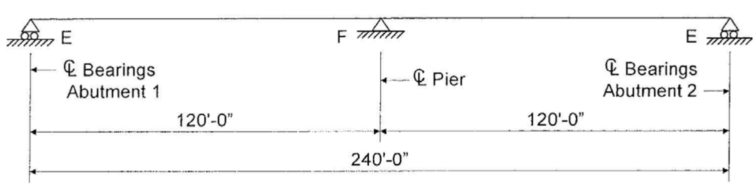
E_D = modulus of elasticity of deck mat'l (ksi)

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

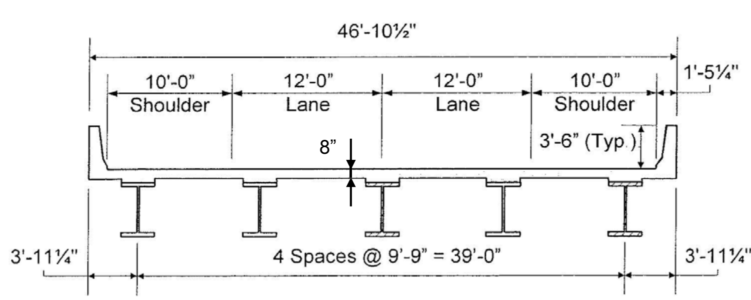
e_g = distance between the centers of gravity of the beam and deck (in)

A = area of the beam (in^2)

Live Load Distribution



Legend:
E = Expansion Bearings
F = Fixed Bearings



Live Load Distribution

Check the range of applicability

$3.5 \le S \le 16.0$

$S = 9.75 \text{ ft}$ OK

$4.5 \le ts \le 12.0$

$ts = 8.0 \text{ in}$ OK

$20 \le L \le 240$

$L = 120 \text{ ft}$ OK

$Nb \ge 4$

$Nb = 5$ OK

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

$K_g = 818{,}611 \text{ in}^4$ OK (based on section properties)

Live Load Distribution

One Design Lane Loaded:

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

Two or More Design Lanes Loaded:

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

Compute One Lane Distribution Factor

Live Load Distribution

One Design Lane Loaded:

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

$$= 0.06 + (9.75/14)^{0.4}(9.75/120)^{0.3}[818{,}611/(12(120)(8)^3)]^{0.1}$$

$$= 0.06 + (0.87)(0.47)(1.01) = 0.47$$

Therefore 1 beam carries 0.47 lanes of LL

Do not convert units, already included in equations.

Would then compute for 2 lanes and exterior girders…

Multiple Presence Factor

- Distribution was for applying live load to a girder.
- MPF is for calculating the live load's effect on substructure loadings.

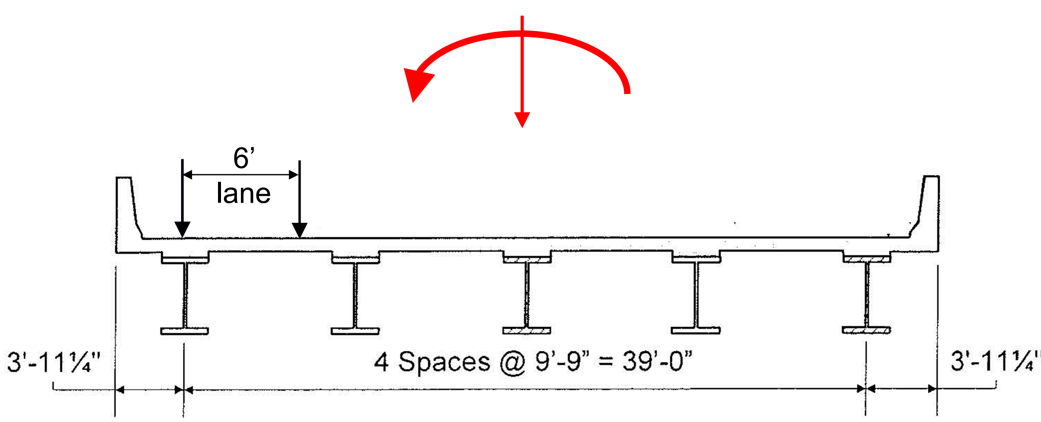
Multiple Presence Factor

- Probability based factor used to adjust LL for substructure design.
- Section 3.6.1.1.2
- Not used in conjunction with distribution factors. (Already in factor)
- Not applied to Fatigue Truck (always 1 truck)

Table 3.6.1.1.2-1 Multiple Presence Factors *m*.

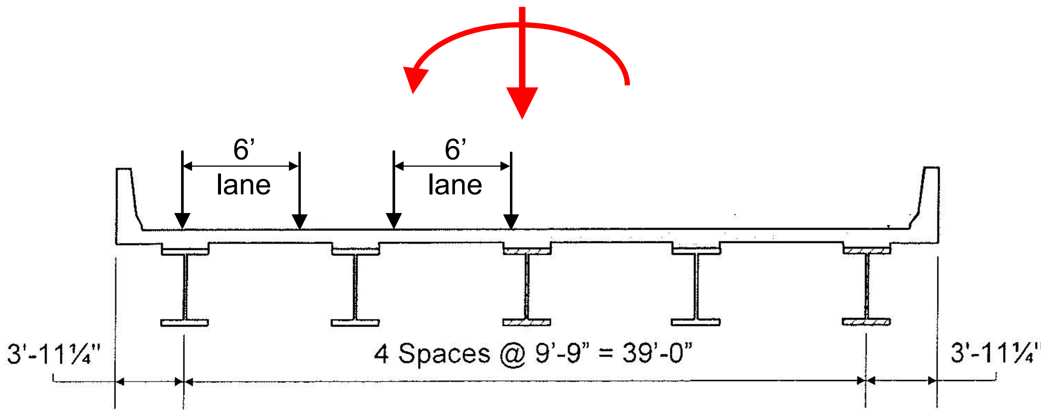
Number of Loaded Lanes	Multiple Presence Factors *m*
1	1.20
2	1.00
3	0.85
> 3	0.65

Multiple Presence Factor



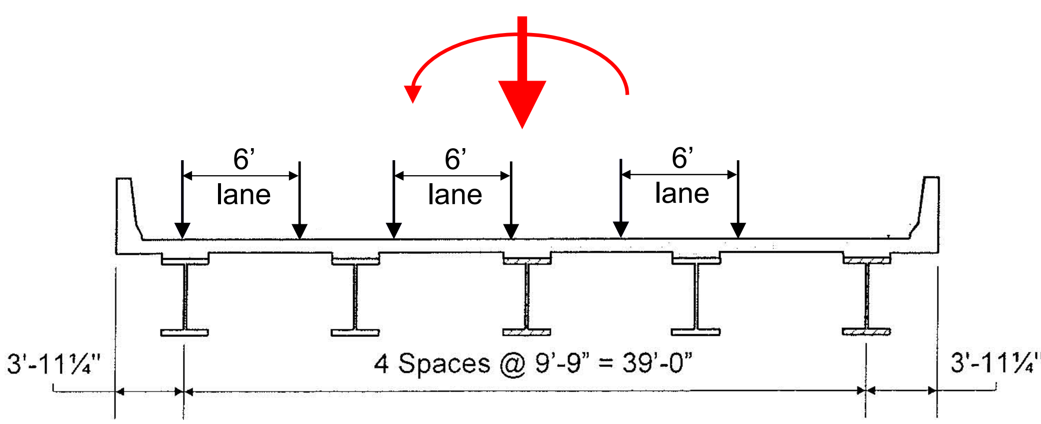
m = 1.2

Multiple Presence Factor



m = 1.0

Multiple Presence Factor



m = 0.85

Bridge Loads

Over 500 failures in the United States between 1989 and 2000.

Age from < 1 year (during construction) to 157 years. Average 52.5 years.

Most frequent causes:

- Flood and scour = 53%
- Bridge overload & lateral impact = 20%
- Other includes design, detailing, construction, material, and maintenance.

Pedestrian Loads

- AASHTO 3.6.1.6
- For highway bridges, Load = 0.075 ksf
  - Sidewalks > 2' wide
  - This load treated as 1 lane when using multi-presence factor per 3.6.1.1.2
- For pedestrian bridges, Load = 0.085 ksf
  - Add H5 truck if 7' < width < 10'
  - Add H10 truck if width > 10'
  - Only if bridge can be used for maint. vehicles.

Centrifugal Force (CE)

- AASHTO 3.6.3
- $C = f \times v^2/g \times R$
  - $f = 1.0$ for fatigue, 4/3 for all other combinations
  - $v =$ highway design speed (ft/sec)
  - $g = 32.2\ \text{ft/sec}$
  - $R =$ Radius of curvature of highway (ft)
  - Applied design truck weight 6’ above roadway

Centrifugal Force (CE)

Assume 50 mph, $R = 1000'$

Assume truck/tandem reaction = 50 k

$1.0\ \text{ft/sec} = 0.682\ \text{mph}$

$C = 4/3 \times (50/0.682)^2/(32.2 \times 1{,}000')$

$= 0.223$

$CE = 0.223 \times 50\ \text{k} = 11.2\ \text{k}$ per lane

Braking Force (BR)

- AASHTO 3.6.4
- Greater of:
  - 25% of truck/tandem axle loads
  - 5% of (truck/tandem + lane load)
  - Apply 6’ above roadway
  - All lanes simultaneously loaded if likely to become one-directional in future.

Braking Force (BR)

Given: truck/tandem reaction = 50 k

lane load reaction = 80 k

Greater of:

$$25\% \times 50\ \mathrm{k} = 12.5\ \mathrm{k}$$

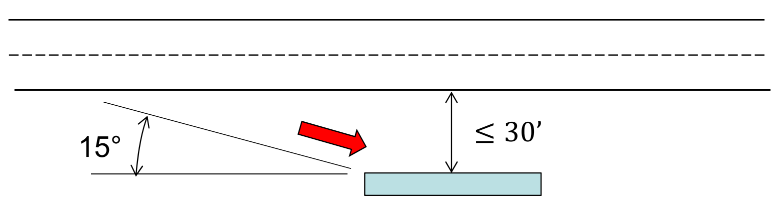
$$5\% * (50\ \mathrm{k} + 80\ \mathrm{k}) = 6.5\ \mathrm{k}$$

$BR = 12.5\ k$

Vehicular Collision Force (CT)

- AASHTO 3.6.5
- 600 k load applied 5' above ground.
- 0° to 15° with edge of pavement
  - Applied if:
    - < 30' to edge of roadway or
    - Per AREMA for RR impact
  - Not needed if:
    - Protected by embankment
    - Protected by 54" barrier within 10' of structure
    - Protected by 42" barrier more than 10' from struct.

Vehicular Collision Force (CT)



Equivalent to an 80 k Semi at 50 mph.

Water Loads (WA)

- AASHTO 3.7
  - Static Pressure applied similar to earth load but with 0.0624 kcf load
  - Buoyancy of 0.0624 kcf on submerged portions
    - This is an uplift force and reduces reactions

Water Loads (WA)

- Stream Pressure
  - Applied parallel to flow of stream.
  - $p = C_D \times V^2/1{,}000$
    - $p =$ stream pressure (ksf)
    - $C_D =$ drag coefficient
    - $V =$ design velocity of water (ft/sec)
- Longitudinal and Lateral equation the same but drag coefficients are different.

Water Loads (WA)

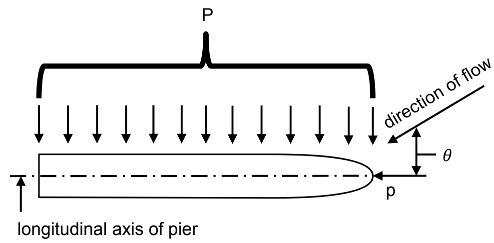


Table 3.7.3.1-1 Drag Coefficient.	
Type	C_D
Semicircular-nosed pier	0.7
Square-ended pier	1.4
Debris lodged against the pier	1.4
Wedged-nosed pier with nose angle 90° or less	0.8

Table 3.7.3.2-1 Lateral Drag Coefficient.	
Angle, θ, between direction of flow and longitudinal axis of the pier	C_L
$0°$	0.0
$5°$	0.5
$10°$	0.7
$20°$	0.9
$\geq 30°$	1.0

The lateral drag force shall be taken as the product of the lateral stream pressure and the surface exposed thereto.

Water Loads (WA)

Given: 3’ wide stem w/ semi-circular face

20’ high exposed surface

V = 6.0 ft/sec

$\theta = 0°$

$$p = C_D \times V^2/1{,}000 = 0.7 \times 6.0^2/1{,}000 = 0.025 \text{ ksf}$$

$$WA = 0.025 \times 3' \times 20' = 1.5 \text{ k}$$

Wind Load (WL and WS)

- AASHTO 3.8
- Pressure based on 100 mph wind.
- Apply to all exposed areas

Table 3.8.1.2.2-1 Base Wind Pressures, P_B, for Various Angles of Attack and $V_B = 100$ mph.

Skew Angle of Wind	Trusses, Columns and Arches		Girders	
	Lateral Load	Longitudinal Load	Lateral Load	Longitudinal Load
Degrees	ksf	ksf	ksf	ksf
0	0.075	0.000	0.050	0.000
15	0.070	0.012	0.044	0.006
30	0.065	0.028	0.041	0.012
45	0.047	0.041	0.033	0.016
60	0.024	0.050	0.017	0.019

Wind Load (WL and WS)

If structure above 30' above ground/water:

$$V_{DZ} = 2.5V_0\left(\frac{V_{30}}{V_B}\right) ln\left(\frac{Z}{Z_0}\right) \qquad (3.8.1.1\text{-}1)$$

Where:

V_{DZ} = design wind velocity at design elevation, Z (mph)

V_{30} = wind velocity at 30.0 ft. above low ground or above design water level (mph)

V_B = base wind velocity of 100 mph at 30.0 ft. height, yielding design pressures specified in Articles 3.8.1.2 and 3.8.2

Z = height of structure at which wind loads are being calculated as measured from low ground, or from water level, > 30.0 ft.

V_0 = friction velocity, a meteorological wind characteristic taken, as specified in Table 1, for various upwind surface characteristics (mph)

Wind Load (WL and WS)

Condition	Open Country	Suburban	City
V_0 (mph)	8.20	10.90	12.00
Z_0 (ft)	0.23	3.28	8.20

V_{30} may be established from:

- Fastest-mile-of-wind charts available in ASCE 7-88 for various recurrence intervals,
- Site-specific wind surveys, and
- In the absence of better criterion, the assumption that $V_{30} = V_B = 100\text{ mph}$

Wind Load (WL and WS)

- Wind on vehicles
  - 0.100 klf force applied normal to and 6' above deck.
- Wind on Superstructure
  - For girder bridges with individual spans < 125' with a max. height of 30' apply 0.100 ksf transverse and 0.040 ksf longitudinal simultaneously.

Wind Load (WL and WS)

- Vertical Wind Pressure (AASHTO 3.8.2)
  - Upward force of 0.020 ksf times deck width applied at windward deck quarter point.
  - Creates both upward force and overturning moment.

Earth Pressure (EH, ES & LS)

- AASHTO 3.11
- Walls with little to no movement designed for at-rest earth pressure.
- Must be “flexible” to use active pressures.

Table C3.11.1-1 Approximate Values of Relative Movements Required to Reach Active or Passive Earth Pressure Conditions (*Clough and Duncan 1991*).

Types of Backfill	Values of Δ/H	
	Active	Passive
Dense sand	0.001	0.01
Medium dense sand	0.002	0.02
Loose sand	0.004	0.04
Compacted silt	0.002	0.02
Compacted lean clay	0.010	0.05
Compacted fat clay	0.010	0.05

Earth Pressure (EH, ES & LS)

Use standard geotechnical equations to calculate loads.

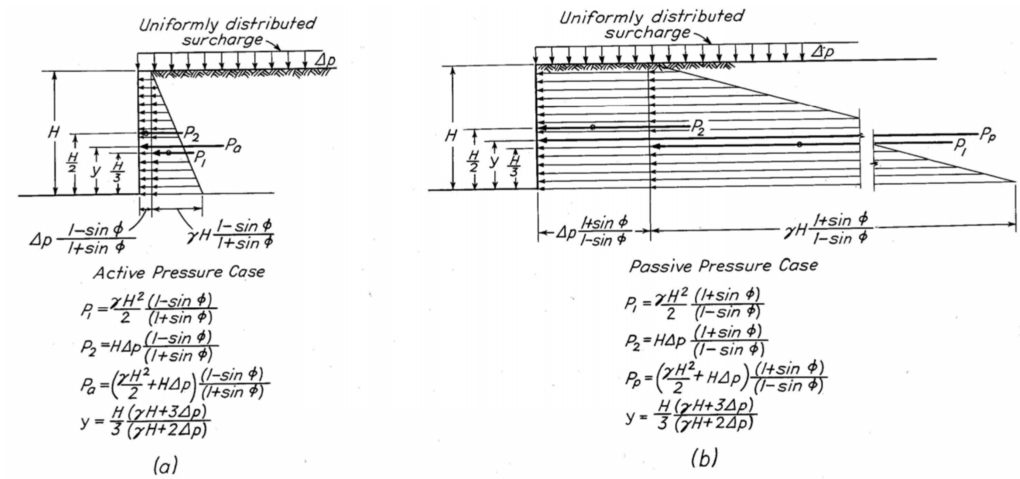


Fig. 7–22. Effect of uniformly distributed surcharge on lateral pressure, cohesionless soil.

Superimposed Deformations (TU, TG, SH, CR, SE & PS)

- AASHTO 3.12
- Temperature movement, TU
- Method A uses temp range from table

Table 3.12.2.1-1 Procedure A Temperature Ranges.			
Climate	Steel or Aluminum	Concrete	Wood
Moderate	0° to 120°F	10° to 80°F	10° to 75°F
Cold	−30° to 120°F	0° to 80°F	0° to 75°F

- If temp below 32° for 14 or more days then cold climate.

Superimposed Deformations (TU, TG, SH, CR, SE & PS)

- Method B uses temp range from contour maps Fig. 3.12.2.2-1 to 3.12.2.2-4.

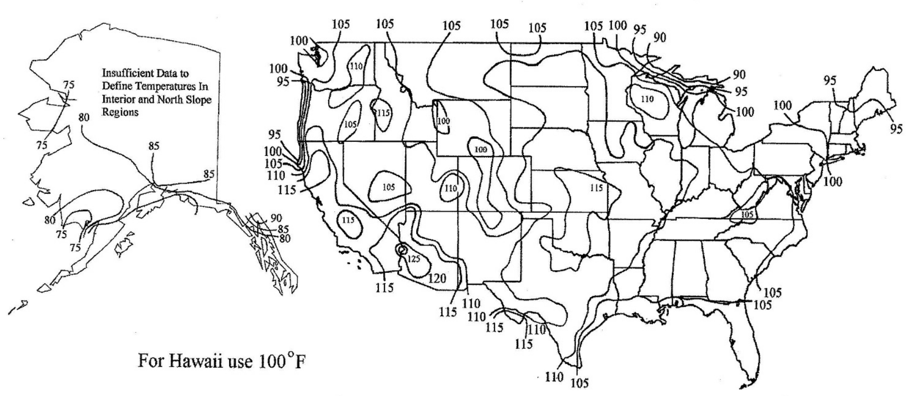


Figure 3.12.2.2-1 Contour Maps for $T_{MaxDesign}$ for Concrete Girder Bridges with Concrete Decks.

Superimposed Deformations (TU, TG, SH, CR, SE & PS)

- $\Delta = \alpha \Delta_t L$
  - α = coefficient of thermal expansion
    - Concrete is 6.0×10^{-6} (5.4.2.2)
    - Steel is 6.5×10^{-6}
  - L = expansion length (in)
  - Δ_t = temperature range (deg)

Superimposed Deformations (TU, TG, SH, CR, SE & PS)

- Shrinkage & Creep (3.12.4 refers to 5.4.2.3)
- Coefficient of shrinkage
  - 0.0002 after 28 days
  - 0.0005 after 1 year
- Creep covered in prestressed concrete session.

ASCE | KNOWLEDGE & LEARNING

Superimposed Deformations (TU & SH)

Given: Cold climate, method A

Concrete pour temp = 60°

Exp. Length = 120’

$\Delta_{SH} = 0.0005 \times 120' \times 12''/' = 0.72$ inch

$\Delta_{TU} = 6.0 \times 10^{-6} \times 60° \times 120' \times 12''/' = 0.52$ inch

ASCE | KNOWLEDGE & LEARNING

Superimposed Deformations (TU & SH)

The force on a column due to a thermal change in length of the superstructure is:

$$F = \frac{3EI\Delta}{(h)^3 \times (1{,}728)}$$

where:

$E =$ Modulus of Elasticity of column, ksi

$I =$ Moment of Inertia of column, in^4

$\Delta =$ Movement due to Temp or Shrinkage, in

$h =$ Column height, feet

$F =$ Force per column, kips

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	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	-	-	1.00	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	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 Construction 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 and Structural Plate Culverts with Deep Corrugations~~ ▪ Flexible Buried Structures • Metal Box Culverts and Structural Plate Culverts with Deep Corrugations • Thermoplastic culverts • All others	 1.00 1.35 1.30 1.35 ~~1.95~~ ~~1.50~~ 1.5 1.3 1.95	 N/A 1.00 0.90 0.90 ~~0.90~~ ~~0.90~~ 0.9 0.9 0.9
ES: Earth Surcharge	1.50	0.75

Problem 3

Assuming:

Service Dead Load Moment = 1,000 ft-kip

Service Live Load Moment = 750 ft-kip (400 ft-kip due to lane loading)

Live Load due to 1 lane loaded condition.

Water, Friction, Temp, Shrink. and Settlement all = 0

Compute the ultimate moments using the Strength I combination.

Solution

Problem No 3

$$IM = 33\%$$

$$LL + IM = 1.2[400 + (350 \times 1.33)] = 1{,}038.6\ \text{‘k}$$

$$\gamma_p = 1.25\ \text{max}, 0.90\ \text{min}$$

$$Mu_{max} = 1.25(1{,}000\ ‘k) + 1.75(1{,}038.6\ \text{‘k})$$

$$= 3{,}068\ ‘k$$

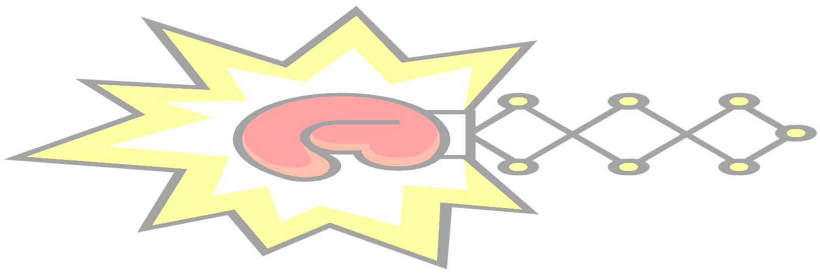
$$Mu_{min} = 0.90(1{,}000\ ‘k) + 1.75(1{,}038.6\ ‘k)$$

$$= 2{,}718\ ‘k$$

Are we there yet?

Deck Design

- Strip Method (4.6.2)
  - Structural simplification where deck is replaced by a set of continuous beams.
  - Beams assumed as unyielding supports.
  - A single line of wheels acts on this beam.



- Maximum width of strip is 144”

Deck Design

Table 4.6.2.1.3-1 Equivalent Strips.

Type of Deck	Direction of Primary Strip Relative to Traffic	Width of Primary Strip (in)
Concrete:		
• Cast-in-place	Overhang Either Parallel or Perpendicular	$45.0 + 10.0X$ $+M: 26.0 + 6.6S$ $-M: 48.0 + 3.0S$
• Cast-in-place with stay-in-place concrete formwork	Either Parallel or Perpendicular	$+M: 26.0 + 6.6S$ $-M: 48.0 + 3.0S$
• Precast, post-tensioned	Either Parallel or Perpendicular	$+M: 26.0 + 6.6S$ $-M: 48.0 + 3.0S$
Steel:		
• Open Grid	Main Bars	$1.25P + 4.0S_b$
• Filled or partially filled grid	Main Bars	Article 4.6.2.1.8 applies
• Unfilled, composite grids	Main Bars	Article 4.6.2.1.8 applies

Deck Design

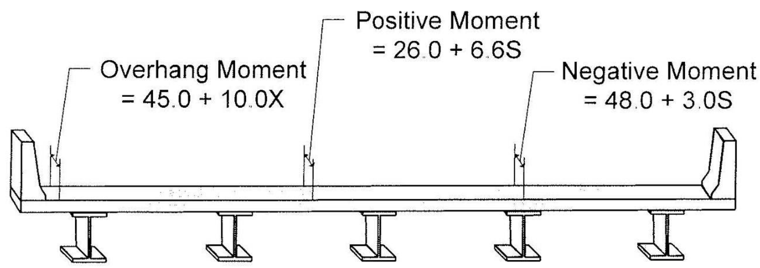
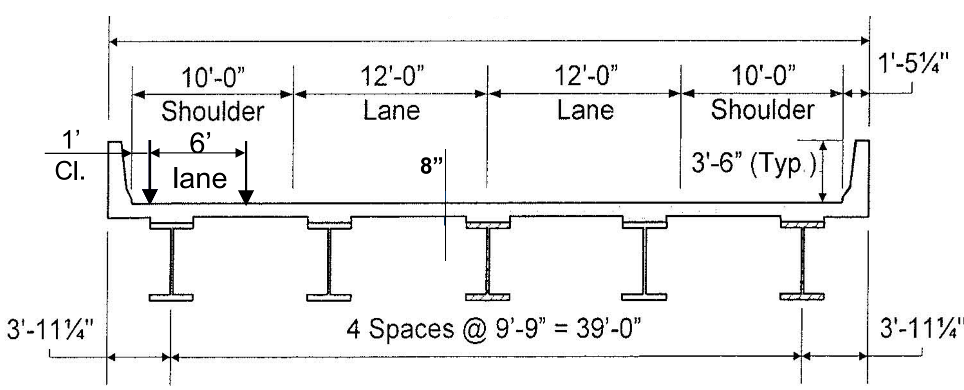


Figure 2-2 Equivalent Strip Equations for Various Parts of the Deck

- $S =$ spacing of supporting components (ft)
- $X =$ distance from load to point of support (ft)

Problem 4

- Calculate the strip widths for the bridge section shown below.



Deck Design

- $X = 47.25'' - 17.25'' - 12'' = 18'' = 1.5'$
- Overhang strip $= 45 + 10(1.5) = 60''$
- Positive Moment $= 26.0 + 6.6(9.75) = 90.3''$
- Negative Moment $= 48.0 + 3.0(9.75) = 77.2''$
- All < 144" therefore OK

Deck Design

Deck Load $= 0.150\ \text{kcf} \times 8''/12 = 0.100\ \text{klf/ft}$

$M_{DC} = w(l)^2/10 = 0.100(9.75)^2/10 = 0.95\ \text{K} - \text{ft/ft}$

Future Wearing Surface = 30 psf

$M_{DW} = 0.030(9.75)^2/10 = 0.29\ \text{K} - \text{ft/ft}$

Parapet $= 0.53\ \text{K/ft}$ (Composite loads may be spread evenly across the deck, therefore $0.53/46.88' = 0.011$)

$M_{DC} = 0.011(9.75)^2/10 = 0.11\ \text{K} - \text{ft/ft}$

Deck Design

- The live load portion of the factored design moments will be computed using *Table A4.1-1*. These moments per unit width include dynamic load allowance and multiple presence factors.
- The values are tabulated using the equivalent strip method.

Deck Design

Table A4-1 Maximum Live Load Moments Per Unit Width, kip-ft./ft.

S		Positive Moment	NEGATIVE MOMENT						
			Distance from CL of Girder to Design Section for Negative Moment						
			0.0 in.	3 in.	6 in.	9 in.	12 in.	18 in.	24 in.
4'	-0"	4.68	2.68	2.07	1.74	1.60	1.50	1.34	1.25
4'	-3"	4.66	2.73	2.25	1.95	1.74	1.57	1.33	1.20
4'	-6"	4.63	3.00	2.58	2.19	1.90	1.65	1.32	1.18
4'	-9"	4.64	3.38	2.90	2.43	2.07	1.74	1.29	1.20
5'	-0"	4.65	3.74	3.20	2.66	2.24	1.83	1.26	1.12
5'	-3"	4.67	4.06	3.47	2.89	2.41	1.95	1.28	0.98
5'	-6"	4.71	4.36	3.73	3.11	2.58	2.07	1.30	0.99
5'	-9"	4.77	4.63	3.97	3.31	2.73	2.19	1.32	1.02
6'	-0"	4.83	4.88	4.19	3.50	2.88	2.31	1.39	1.07
6'	-3"	4.91	5.10	4.39	3.68	3.02	2.42	1.45	1.13
6'	-6"	5.00	5.31	4.57	3.84	3.15	2.53	1.50	1.20
6'	-9"	5.10	5.50	4.74	3.99	3.27	2.64	1.58	1.28
7'	-0"	5.21	5.98	5.17	4.36	3.56	2.84	1.63	1.37
7'	-3"	5.32	6.13	5.31	4.49	3.68	2.96	1.65	1.51
7'	-6"	5.44	6.26	5.43	4.61	3.78	3.15	1.88	1.72
7'	-9"	5.56	6.38	5.54	4.71	3.88	3.30	2.21	1.94
8'	-0"	5.69	6.48	5.65	4.81	3.98	3.43	2.49	2.16
8'	-3"	5.83	6.58	5.74	4.90	4.06	3.53	2.74	2.37
8'	-6"	5.99	6.66	5.82	4.98	4.14	3.61	2.96	2.58
8'	-9"	6.14	6.74	5.90	5.06	4.22	3.67	3.15	2.79
9'	-0"	6.29	6.81	5.97	5.13	4.28	3.71	3.31	3.00
9'	-3"	6.44	6.87	6.03	5.19	4.40	3.82	3.47	3.20
9'	-6"	6.59	7.15	6.31	5.46	4.66	4.04	3.68	3.39
9'	-9"	6.74	7.51	6.65	5.80	4.94	4.21	3.89	3.58

Deck Design

For a girder spacing of 9'-9", the maximum unfactored positive live load moment is 6.74 K-ft/ft.

$$Mu_{LLpos} = \gamma_{LL}(6.74) = 1.75 \times 6.74$$

$$= 11.80K - \text{ft/ft}$$

Deck Design

$$Mu_{posdead} = \gamma_{pDCmax}(0.95\ \text{K} \cdot \text{ft/ft})$$

$$+\gamma_{pDCmax}(0.11\ \text{K} \cdot \text{ft/ft})$$

$$+\gamma_{pDWmax}(0.29\ \text{K} \cdot \text{ft/ft})$$

$\gamma_{pDCmax} = 1.25$ $\gamma_{pDWmax} = 1.50$

$\gamma_{pDCmin} = 0.90$ $\gamma_{pDWmin} = 0.65$

$$Mu_{posdead} = 1.25(0.95) + 1.25(0.11) + 1.50(0.29)$$

$$= 1.76\,\text{K} \cdot \text{ft/ft}$$

Deck Design

$Mu = 11.80 + 1.76 = 13.56\ \mathrm{K} \cdot \mathrm{ft/ft}$

Remember our general equation:

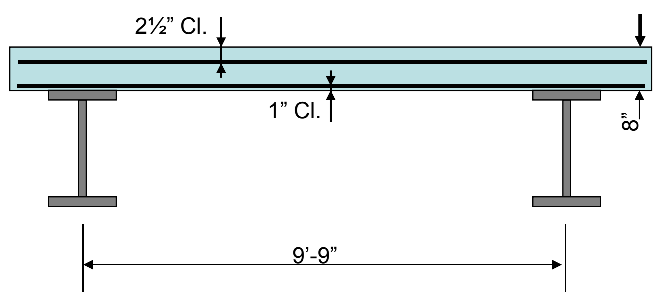
$Q = \sum \eta_i y_i Q_i \leq ØR$

For this case $\eta = 1.0$

$Ø = 0.90$ for Strength Limit State (5.5.4.2)

Deck Design

- Assume bar size
- Calculate as required
- Determine bar spacing
- Use $f'c = 4$ ksi
- Use $fy = 60$ ksi



Assume #5 bars

diam $= 0.625"$

Area $= 0.31\ in^2$

$d_e = t_s - Cover_b - diam/2 - 0.5"$ wearing surface

$d_e = 8" - 1" - (0.625"/2) - 0.5" = 6.19"$

Deck Design

- $\varnothing_f = 0.90$
- $b = 12''$
- $Rn = Mu_{TOT}(12''/') / [\varnothing_f(b)(d_e)^2]$

$= 13.56 \text{ ft} - \text{K}(12''/')/[0.90(12'')(6.19'')^2]$

$= 0.39 \ K/in^2$

$$\rho = 0.85\left(\frac{f_c'}{f_y}\right)\left[1.0 - \sqrt{1.0 - \frac{(2 \cdot Rn)}{(0.85 f_c')}}\right]$$

$p = 0.00698$

Deck Design

- $As = p(b)(d_e) = 0.00698(12)(6.19)$

$= \ 0.52 \text{ in}^2/\text{ft}$

- Required spacing $= (12''/')0.31/0.52$

$= 7.2 \text{ in}$

- Use #5 bars @ 7.0 in

Foundation

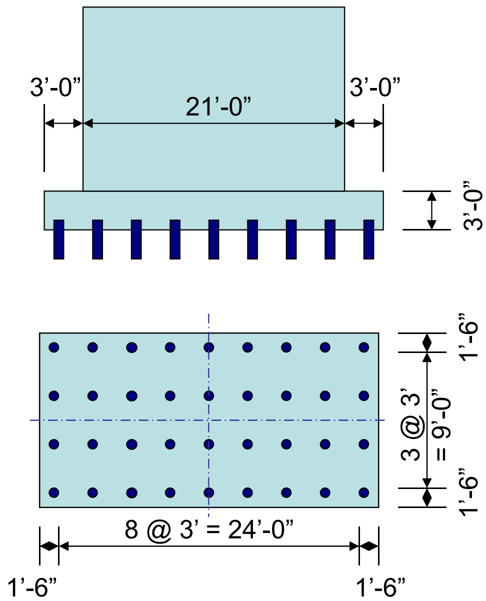
- Service Limit State:
  - Overall Stability and Movement
  - Pile Layout
- Strength Limit State:
  - Footing Shear
  - Footing Moment

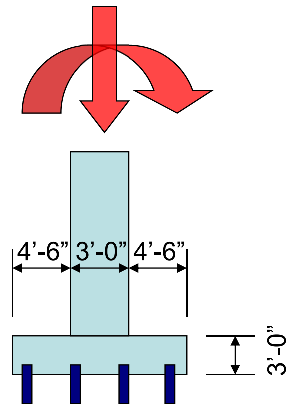
	Load Factors							
	Strength I		Strength III		Strength V		Service I	
Load	γ_{max}	γ_{min}	γ_{max}	γ_{min}	γ_{max}	γ_{min}	γ_{max}	γ_{min}
DC	1.25	0.90	1.25	0.90	1.25	0.90	1.00	1.00
DW	1.50	0.65	1.50	0.65	1.50	0.65	1.00	1.00
LL	1.75	1.75	-	-	1.35	1.35	1.00	1.00
BR	1.75	1.75	-	-	1.35	1.35	1.00	1.00
TU	1.20	0.50	1.20	0.50	1.20	0.50	1.20	1.00
WS	-	-	1.40	1.40	0.40	0.40	0.30	0.30
WL	-	-	-	-	1.00	1.00	1.00	1.00
EV	1.35	1.00	1.35	1.00	1.35	1.00	1.00	1.00

Foundation

- Sum and factor loads
- Calculate critical pile reaction
  - Or check critical soil pressure for spread footing
- Compute shear and check thickness of footing.
- Compute moment and design footing reinforcement.

Foundation





1'-0" pile embedment

$f'_c = 3000$ psi

$f_y = 60{,}000$ psi

Foundation

- Service loads at top of piles

Type	P (kip)	Mx (ft-kip)
LL	285	0
DC	2,500	0
WS	-120	1,820
WL	0	200
BR	0	600

- Compute Strength I Moments per Table 3.4.1-1 and 3.4.1-2

Load Combinations

Group	P(kip)	Mx(ft-kip)
Strength I_{max}	3,624	1,050
Strength I_{min}	2,749	1,050
Strength III_{max}	2,957	2,548
Strength III_{min}	2,082	2,548
Strength V_{max}	3,462	1,738
Strength V_{min}	2,587	1,738
Service I_{max}	2,749	1,346
Service I_{min}	2,749	1,346

Pile Group Properties

- $N = 4 \times 9 = 36$
- $Ix = 2 \times 9[(1.5')^2 + (4.5')^2]$

  $= 405.0$
- $Iy = 2 \times 4[(3')^2 + (6')^2 + (9')^2 + (12')^2]$

  $= 2{,}160.0$
- Pile Reaction $= Pu/N \pm M_x(c)/I_x \pm M_y(c)/I_y$

Factored Pile Reactions

Group	$Reaction_{max}$(kip)	$Reaction_{min}$(kip)
Strength I_{max}	112	89
Strength I_{min}	88	65
Strength III_{max}	110	54
Strength III_{min}	86	30
Strength V_{max}	115	77
Strength V_{min}	91	53
Service I_{max}	91	61
Service I_{max}	91	61

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.
- Bring AASHTO Manual next session.

Questions

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Biography

- Attended U.S. Coast Guard Academy
- BSCE from Purdue University
- 34 years of bridge design
  - Over 500 bridges
  - Reinforced Concrete, Prestressed Concrete, Steel Beam and Girder, Timber
  - Highway, Railroad, Pedestrian
- Co-wrote INDOT's LRFD Bridge Manual
- PE in Indiana, Ohio, Kentucky and Michigan
- Midwest Transportation Director for GAI Consultants, Inc.

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

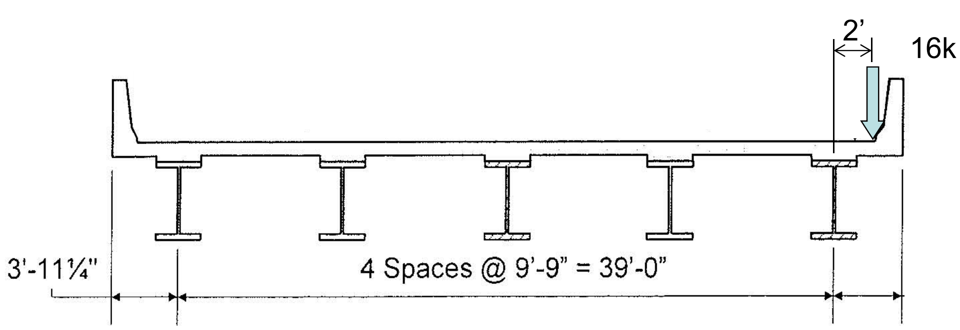
- http://www.fhwa.dot.gov/bridge/lrfd/examples.htm
  - Prestressed Concrete Girder Superstructure Example
  - Steel Girder Superstructure Example
  - Based on 2nd Edition and Interims through 2002
    - A number of sections have changed in the Code between 2002 and 2010 so be careful using this.
- http://www.aisc.org/contentNSBA.aspx?id=20244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

Reinforcing Steel Cover

Table 5.12.3-1 Cover for Unprotected Main Reinforcing Steel (in).	
Situation	Cover (in)
Direct exposure to salt water	4.0
Cast against earth	3.0
Coastal	3.0
Exposure to deicing salts	2.5
Deck surfaces subject to tire stud or chain wear	2.5
Exterior other than above	2.0
Interior other than above ▪ Up to No. 11 bar ▪ No. 14 and No. 18 bars	 1.5 2.0
Bottom of cast-in-place slabs ▪ Up to No. 11 bar ▪ No. 14 and No. 18 bars	 1.0 2.0
Precast soffit form panels	0.8
Precast reinforced piles ▪ Noncorrosive environments ▪ Corrosive environments	 2.0 3.0
Precast prestressed piles	2.0
Cast-in-place piles ▪ Noncorrosive environments ▪ Corrosive environments - General - Protected ▪ Shells ▪ Auger-cast, tremie concrete, or slurry construction	 2.0 3.0 3.0 2.0 3.0

- Be aware of this table since many of the clearances are greater than those in ACI.

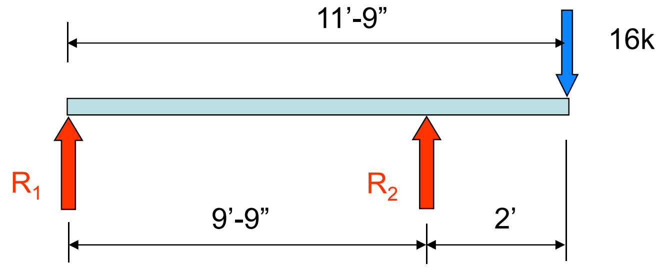
Lever Rule



Find reaction on exterior girder about first interior girder.

Assumes pinned connection at first interior girder.

Lever Rule



- R_1 is a pinned connection
  - Therefore moment @ $R_1 = 0$
- Sum moments about R_1
- $0^{\text{ft}-\text{k}} = 16\ \text{k} \times 11.75' - R_2 \times 9.75'$
- $R_2 = 16\ \text{k} \times 11.75'/9.75' = 19.3^{\text{ft}-\text{k}}$

ASCE KNOWLEDGE & LEARNING