A2—REINFORCED CONCRETE T-BEAM BRIDGE: EVALUATION OF AN INTERIOR BEAM

PART A—LOAD AND RESISTANCE FACTOR RATING METHOD

A2A.1—Bridge Data

Span:	26 ft
Year Built	1925
Materials:	
Concrete:	f'_c = 3 ksi
Reinforcing Steel:	Unknown f_y
Condition:	Minor deterioration has been observed, but no section loss. NBI Item 59 = 6
Riding Surface:	Field verified and documented: Smooth approach and deck
ADTT (one direction):	1,850
Skew:	0°

A2A.2—Dead-Load Analysis—Interior Beam

Permanent loads on the deck are distributed uniformly among the beams. LRFD Design 4.6.2.2.1

A2A.2.1—Components and Attachments, *DC*

Structural Concrete:

Consisting of deck + stem + haunches (conservatively, 2[1]/2-in. chamfers were not deducted)

$$\left[\frac{6\text{ in.}}{12}\times 6.52\text{ ft}+1.25\text{ ft}\times 2\text{ ft}+2\left(\frac{1}{2}\times\frac{6\text{ in.}}{12}\times\frac{6\text{ in.}}{12}\right)\right]\times(0.150\text{ kcf})$$

$$= 0.902\text{ kip/ft}$$

$$\text{Railing and curb } 0.200\text{ kip/ft}\times\frac{1}{2} = 0.100\text{ kip/ft}$$

$$\text{Total per beam, } DC = \overline{1.002\text{ kip/ft}}$$

$$M_{DC} = \frac{1}{8}\times 1.002\times 26^2 = 84.7\text{ kip-ft}$$

$$V_{DCmax} = 1.002(0.5\times 26) = 13.0\text{ kips}$$

A2A.2.2—Wearing Surface, *DW*

Thickness was field measured: 6A.2.2.3

Asphalt Overlay:

$$\left(\frac{5\text{ in.}}{12}\right)(22\text{ ft})(0.144\text{ kcf})\left(\frac{1}{4}\right) = 0.330\text{ kip/ft}$$

$$M_{DW} = \frac{1}{8}\times 0.330\times 26^2 = 27.9\text{ kip-ft}$$

$$V_{DWmax} = 0.33(0.5\times 26) = 4.3\text{ kips}$$

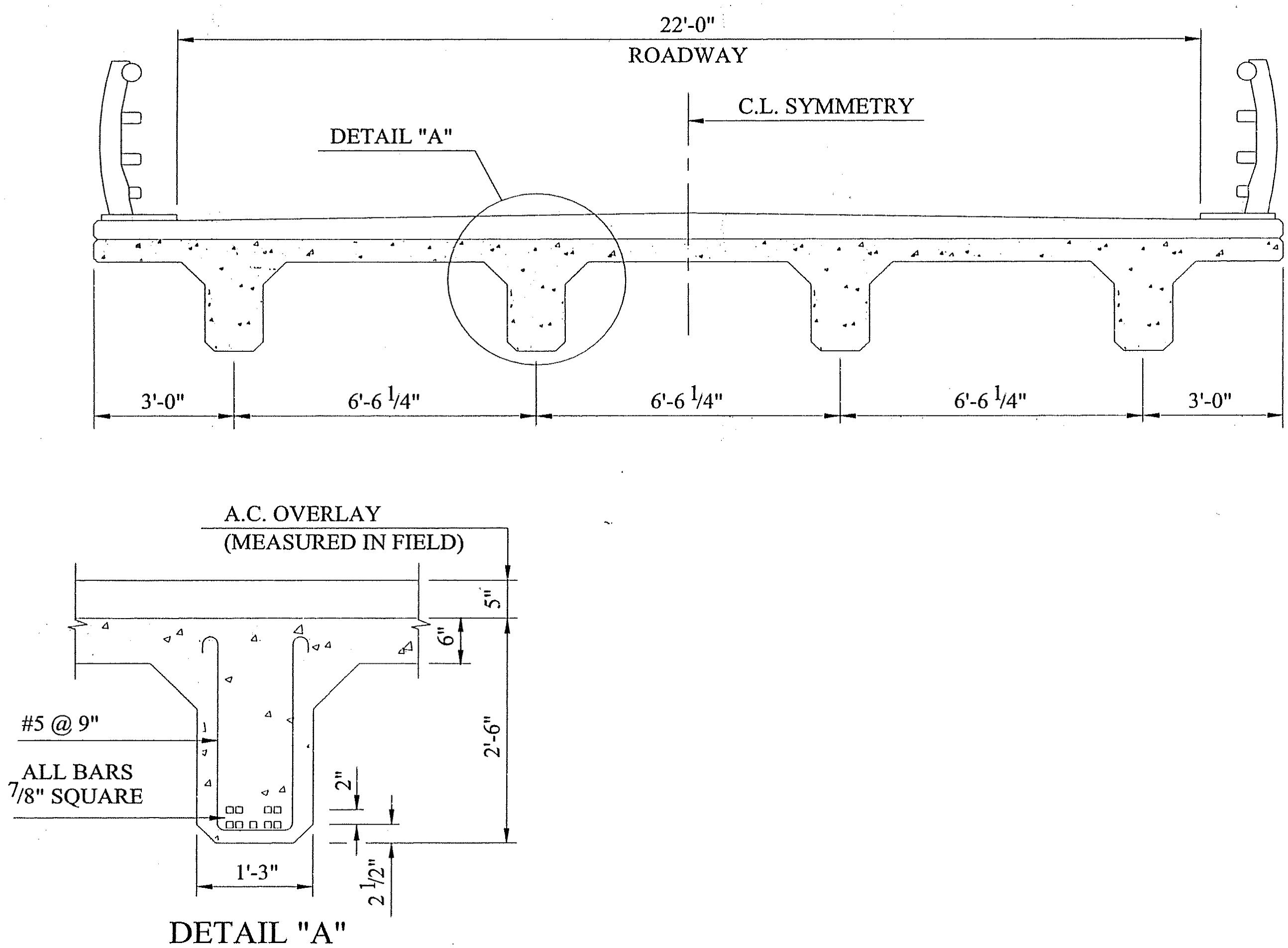

Figure A2A.2.2-1—Reinforced Concrete T-Beam Bridge

A2A.3—Live-Load Analysis—Interior Beam

A2A.3.1—Compute Live-Load Distribution Factor

AASHTO LRFD Type (e) cross section

LRFD Design Table 4.6.2.2.1-1

Longitudinal Stiffness Parameter, K_g

LRFD Design Eq. 4.6.2.2.1-1

$$K_g = n\left(I + Ae_g^2\right)$$

$$n = 1.0$$

$$I = \frac{1}{12} \times 15 \times 24^3 = 17{,}280 \text{ in.}^4$$

$$A = 15 \times 24 = 360 \text{ in.}^2$$

$$e_g = \frac{1}{2}(24+6) = 15 \text{ in.}$$

$$K_g = 1.0\,(17{,}280 + 360 \times 15^2) = 98{,}280 \text{ in.}^4$$

$$\frac{K_g}{12Lt_s^3} = \frac{98{,}280}{12 \times 26 \times 6^3} = 1.46$$

A2A.3.1.1—Distribution Factor for Moment, g_m (LRFD Design Table 4.6.2.2.2b-1)

One Lane Loaded:

$$g_{m1} = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

$$= 0.06 + \left(\frac{6.52}{14}\right)^{0.4}\left(\frac{6.52}{26}\right)^{0.3}(1.46)^{0.1}$$

$$= 0.565$$

Two or More Lanes Loaded:

$$g_{m2} = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12Lt_s^3}\right)^{0.1}$$

$$= 0.075 + \left(\frac{6.52}{9.5}\right)^{0.6}\left(\frac{6.52}{26}\right)^{0.2}(1.46)^{0.1}$$

$$= 0.703 > 0.565$$

$$\therefore \text{ use } g_m = 0.703$$

A2A.3.1.2—Distribution Factor for Shear, g_v (LRFD Design Table 4.6.2.2.3a-1)

One Lane Loaded:

$$g_{v1} = 0.36 + \frac{S}{25.0}$$

$$= 0.36 + \frac{6.52}{25.0}$$

$$= 0.621$$

Two or More Lanes Loaded:

$$g_{v2} = 0.2 + \frac{S}{12} - \left(\frac{S}{35}\right)^{2.0}$$

$$= 0.2 + \frac{6.52}{12} - \left(\frac{6.52}{35}\right)^{2.0}$$

$$= 0.709 > 0.62$$

$\therefore$ use $g_v =$ 0.709

A2A.3.2—Compute Maximum Live Load Effects

A2A.3.2.1—Maximum Design Live Load (HL-93) Moment at Midspan

Design Lane Load Moment = 54.1 kip-ft

Design Truck Moment = 208.0 kip-ft

Tandem Axles Moment = 275.0 kip-ft Governs

IM = 33 percent

LRFD Design Table 3.6.2.1-1

M_{LL+IM} = 54.1 + 275.0 × 1.33

= 419.9 kip-ft

A2A.3.2.2—Maximum Design Live Load Shear (HL-93) at Critical Section

See Article A2A.7.

A2A.3.2.3—Distributed Live Load Moments

Design Live Load HL-93:

M_{LL+IM} = 419.9 × 0.703

= 295.2 kip-ft

A2A.4—Compute Nominal Flexural Resistance

A2A.4.1—Compute Effective Flange Width, b_e (LRFD Design 4.6.2.6.1)

Effective Flange Width, b_e, may be taken as the tributary width perpendicular to the axis of the member.

$\therefore$ use $b_e = 78.25$ in.

A2A.4.2—Compute Distance to Neutral Axis, c

LRFD Design 5.6.3.1.1

Assume rectangular section behavior.

β_1 = 0.85 for $f'_c = 3{,}000$ psi

LRFD Design 5.6.2.2

$$c = \frac{A_s f_y}{0.85 f_c' \beta_1 b}$$ LRFD Design Eq. 5.6.3.1.1-4

$$A_s = 9\left(\frac{7}{8}\right)^2 = 6.89 \text{ in.}^2$$ (nine $^7/_8$-in.^{2} bars)

b = 78.25 in.

f_y = 33 ksi (unknown steel) Table 6A.5.2.2-1

$$c = \frac{6.89 \times 33}{0.85 \times 3.0 \times 0.85 \times 78.25}$$

= 1.34 in. < 6 in.

The neutral axis is within the slab. Therefore, there will be rectangular section behavior.

a = $c\beta_1$

= 1.34 × 0.85

= 1.14 in.

Distance from bottom of section to CG of reinforcement, $\bar{y}$

$$\bar{y} = \frac{4 \times 4.5 + 5 \times 2.5}{9}$$

$\bar{y}$ = 3.39 in.

d_s = $h - \bar{y}$

h = 30 in.

d_s = 30 in. – 3.39 in.

= 26.61 in.

$$M_n = A_s f_y \left(d_s - \frac{a}{2}\right)$$ LRFD Design 5.6.3.2.3, LRFD Design Eq. 5.6.3.2.2-1

$$= 6.89 \times 33 \left(26.61 - \frac{1.14}{2}\right)\frac{1}{12}$$

= 493.4 kip-ft

A2A.5—Maximum Reinforcement (6A.5.5)

The factored resistance (φ factor) of compression controlled sections shall be reduced in accordance with LRFD Design Article 5.5.4.2. This approach limits the capacity of over-reinforced (compression controlled) sections. C6A.5.5

The net tensile strain, ε_t, is the tensile strain at nominal strength and determined by strain compatibility using similar triangles. LRFD Design C5.6.2.1

Given an allowable concrete strain of 0.003 and depth to neutral axis c = 1.34 in.

$$\frac{\varepsilon_c}{c} = \frac{\varepsilon_t}{d - c}$$

$$\frac{0.003}{1.34 \text{ in.}} = \frac{\varepsilon_t}{26.61 \text{ in.} - 1.34 \text{ in.}}$$

Solving for ε_t, ε_t = 0.0566.

For ε_t = 0.0566 > 0.005, the section is tension controlled. LRFD Design 5.6.2.1

For conventional construction and tension controlled reinforced concrete sections, resistance factor φ shall be taken as 0.90. LRFD Design 5.5.4.2

A2A.6—Minimum Reinforcement (6A.5.6)

The amount of reinforcement must be sufficient to develop M_r equal to the lesser of: LRFD Design 5.6.3.3

$1.2M_{cr}$ or $1.33M_u$

$$M_r = \varphi_f M_n = 0.90 \times 493.4 \text{ kip-ft}$$
$$= 444.1 \text{ kip-ft}$$

1. $1.33M_u = 1.33\,(1.75 \times 295.2 + 1.25 \times 84.7 + 1.25 \times 27.9)$

$= 874.3 \text{ kip-ft} > 444.1 \text{ kip-ft}$ No Good

2. $$M_{cr} = \gamma_3\left[\left(\gamma_1 f_r + \gamma_2 f_{cpe}\right)S_c - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right)\right]$$ LRFD Design Eq. 5.6.3.3-1

M_{dnc} = 0 Total unfactored dead load moment acting on the monolithic or noncomposite section

f_{cpe} = 0 Compressive stress in concrete due to effective prestress forces only at extreme fiber of section where tensile stress is caused by externally applied loads

S_{nc} = $\dfrac{I}{y_t}$ Uncracked section modulus (neglect steel)

γ_1 = flexural cracking variability factor = 1.6

γ_2 = prestress variability factor = 0

γ_3 = ratio of specified minimum yield strength to ultimate tensile strength of nonprestressed reinforcement = 0.67

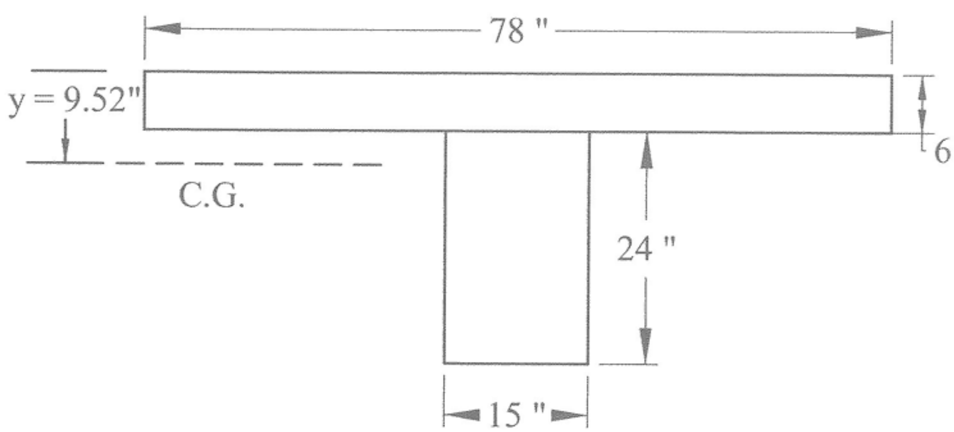

Figure A2A.6-1 Cross Section of Concete T-Beam—Depth to Centroid of Uncracked Section

$$y = \frac{\sum(A_i \times y_i)}{\sum A_i}$$

$$y = \frac{(78\times6\times3)+(24\times15\times18)}{(78\times6)+(24\times15)} = 9.52 \text{ in.}$$

from top of slab to centroid of uncracked section

$I \quad = \quad \sum\left(I_o + A_c d^2\right)$ where $I_0 = bh^3/12$

	y	A_c	$A_c y$	d	Ad^2	I_0
slab	3	468	1,404	6.52	19,895	1,404
stem	18	360	6,480	8.48	25,888	17,280
		828	7,884		45,783	18,684

$I \quad = \quad (18,684 + 45,783) = 64,467$

$y_b \quad = \quad 30 \text{ in.} - 9.52 \text{ in.} \quad = \quad 20.48 \text{ in.}$

$S_{bc} \quad = \quad \dfrac{64,467}{20.48} \quad = \quad 3,148 \text{ in.}^3$

$f_r \quad = \quad 0.24\sqrt{f'_c} \quad = \quad 0.24\sqrt{3.0} \quad = \quad 0.416 \text{ ksi}$

$M_{cr} \quad = \quad (0.67)\left[1.6(0.416+0)3,148-0\right] \quad = \quad 116.99 \text{ kip-ft}$ LRFD Design 5.4.2.6

$M_r \quad = \quad \varphi M_n \quad = \quad 0.9\ (493.4)$
$M_r \quad = \quad 444.1 \text{ kip-ft} \quad > \quad M_{cr} \quad = \quad 116.99 \text{ kip-ft}$ OK

The section meets the requirements for minimum reinforcement.

A2A.7—Compute Nominal Shear Resistance

Stirrups: #5 bars at 9 in.

$$A_v = 2 \times \frac{\pi}{4}\left(\frac{5}{8}\right)^2 = 0.6136 \text{ in.}^2$$

Unknown $f_y \rightarrow 33$ ksi

Critical section for shear: LRFD Design 5.7.3.2
Effective Shear Depth: d_v LRFD Design 5.7.2.8

Distance, meassured perpendicular to the neutral axis, between resultants of the tensile and compressive forces. It need not be taken to be less than the greater of:

$0.9d_e$

$0.72h$

1. $d_v \quad = \quad \dfrac{M_n}{A_s f_y + A_{ps} f_{ps}}$ LRFD Design Eq. C5.7.2.8-1

This quantity depends upon the transfer and development of the reinforcement. Conservatively, we will take d_v as the greater of the remaining criteria to reduce required calculations.

2. 0.9 (26.61) = 23.95 in.
3. 0.72 (30.0) = 21.60 in.

$d_v \quad = \quad 23.95$ in.

Assume θ = 45°

$0.5d_v \cot\theta$ = (0.5) (26.04) (cot 45) = $0.5d_v < d_v$ Use d_v

Critical section for shear at 23.95 in. from face of support.

Bearing pad width = 4 in.

Calculate shear at $23.95 + \frac{4}{2} = 25.95$ in. from centerline of bearing.

Maximum Shear at Critical Section Near Support (25.95 in.) calculated by statics:

V_{TANDEM} = 41.9 kips Governs
V_{TRUCK} = 41.4 kips
V_{LANE} = 7.0 kips

Total Live-Load Shear = (1.33) (41.9) + 7.0 = 62.7 kips — LRFD Design Table 3.6.2.1-1

(including 33 percent increase for dynamic load allowance)

Distributed Shear, V_{LL+IM} = (62.7) (0.709) = 44.5 kips

Dead-Load Shears:

$$V_{DC} = 1.002\left(0.5\times 26 - \frac{25.95}{12}\right) = 10.8 \text{ kips}$$

$$V_{DW} = 0.33\left(0.5\times 26 - \frac{25.95}{12}\right) = 3.6 \text{ kips}$$

Resistance:

The lesser of :

$$V_n = V_c + V_s + V_p$$ LRFD Design Eq. 5.7.3.3-1

$$V_n = 0.25f'_c b_v d_v + V_p$$ LRFD Design Eq. 5.7.3.3-2

In this case there is no V_p contribution, and:

Effective shear depth, $d_v = 23.95$ in. — LRFD Design Eq. 5.7.2.8-1

Minimum web width within the depth d_v, $b_v = 15$ in. — LRFD Design Eq. 5.7.2.8-1

$$V_c = 0.0316\beta\lambda\sqrt{f'_c}b_v d_v$$ LRFD Design Eq. 5.7.3.3-3

$$V_s = \frac{A_v f_y d_v \cot\theta}{s} \quad (\text{for } \alpha = 90°)$$ LRFD Design Eq. 5.7.3.3-4

Simplified Approach: — LRFD Design 5.7.3.4.1

β = 2.0
θ = 45

$$V_c = (0.0316)(2)\sqrt{3.0}(15)(23.95) = 39.3 \text{ kips}$$

$$V_s = \frac{(0.6136)(33)(23.95)\cot 45}{9} = 53.9 \text{ kips}$$

V_n = 39.3 + 53.9 = 93.2 kips

V_n = 0.25 × 3.0 × 15 × 23.95 = 269.4 kips

93.2 kips < 269.4 kips, therefore V_n = 93.2 kips

A2A.8—Summary for Interior Concrete T-Beam

	Dead Load *DC*	Dead Load *DW*	Live Load Distribution Factor	Dist. Live Load + Impact	Nominal Capacity
Moment, kip-ft	84.7	27.9	g_m = 0.703	295.2	493.4
Shear, kips	10.8	3.6	g_v = 0.709	44.5	93.2

A2A.9—General Load Rating Equation

$$RF = \frac{C-(\gamma_{DC})(DC)-(\gamma_{DW})(DW)\pm(\gamma_P)(P)}{(\gamma_L)(LL+IM)} \text{ s}$$ Eq. 6A.4.2.1-1

For Strength Limit States $C = (\varphi_c)(\varphi_s)(\varphi)R_n$

A2A.10—Evaluation Factors (for Strength Limit States)

1. Resistance Factor, φ — LRFD Design 5.5.4.2

φ = ~~1.0~~ 0.90 for flexure and shear of normal weight concrete

2. Condition Factor, φ_c — 6A.4.2.3

No member condition information available. NBI Item 59 = 6.

φ_c = 1.0

3. System Factor, φ_s — 6A.4.2.4

φ_s = 1.0 4-girder bridge with S > 4 ft (for flexure and shear)

A2A.11—Design Load Rating (6A.4.3)

A2A.11.1—Strength I Limit State

$$RF = \frac{(\varphi_c)(\varphi_s)(\varphi)R_n-(\gamma_{DC})(DC)-(\gamma_{DW})(DW)}{(\gamma_L)(LL+IM)}$$

A2A.11.2—Inventory Level (6A.5.4.1)

Table 6A.4.2.2-1

Load	Load Factor
DC	1.25
DW	1.25
LL	1.75

Thickness was field verified

Flexure:

$$RF = \frac{(1.0)(1.0)(0.90)(493.4)-[(1.25)(84.7)+(1.25)(27.9)]}{(1.75)(295.2)}$$

= 0.59

Shear:

$$RF = \frac{(1.0)(1.0)(0.90)(93.2) - [(1.25)(10.8) + (1.25)(3.6)]}{(1.75)(44.5)}$$

$$= 0.85$$

The shear ratings factors for Design Load Rating are calculated for illustration purposes only. In-service concrete bridges that show no visible signs of shear distress need not be checked for shear during design load or legal load ratings. 6A.5.8

A2A.11.3—Operating Level

Table 6A.4.2.2-1

Load	Load Factor γ
DC	1.25
DW	1.25
LL	1.35

For Strength I Operating Level only the live load factor changes; therefore the rating factor can be calculated by direct proportions.

Flexure:

$$RF = 0.59 \times \frac{1.75}{1.35}$$

$$= 0.76$$

Shear:

$$RF = 0.85 \times \frac{1.75}{1.35}$$

$$= 1.10$$

Note: The shear resistance using MCFT varies along the length. The simplified assumptions of $\beta = 2.0$ and $\theta = 45°$ in this example are conservative for high shear–low moment regions. Example A3 demonstrates a case where the shear rating must be performed at multiple locations along the length of the member. Tension in the longitudinal reinforcement caused by moment-shear interaction (LRFD Design Article 5.7.3.5) has not been checked in this example. Example A3 includes demonstrations of this check.

No service limit states apply to reinforced concrete bridge members at the design load check.

A2A.12—Legal Load Rating (6A.5.4.2)

Note: Since the Operating Level Design Load Rating produced $RF < 1.0$ for flexure, load ratings for legal loads should be performed to determine the need for posting.

Live Load: AASHTO Legal Loads—Types 3, 3S2, and 3-3 (Rate for all three) 6A.4.4.2.1

g_m = 0.703

L = 26 ft ($L < 40$ ft)

IM = 33 percent

Even though the condition of the wearing surface has been field evaluated as smooth, the length of the flexure members prevents the use of a reduced *IM*. C6A.4.4.3