Shear Capacity at the End panel $= CV_P$

$=1.00 \times 380.15$

$= 380.15$ kip

A1A.1.5.4—Demand Summary for Interior Stringer

Table A1A.1.5.4-1

	Dead Load DC_1	Dead Load DC_2	Live Load Distribution Factor	Dist. Live Load + Impact	Nominal Capacity
Moment, kip-ft	439.90	129.40	0.627	954.10	2,873.0
Shear, kips	27.10	8.0	0.767	78.90	380.15

A1A.1.6—General Load-Rating Equation

$$RF = \frac{C-(\gamma_{DC})(DC)-(\gamma_{DN})(DW)\pm(\gamma_P)(P)}{(\gamma_L)(LL+IM)}$$

Eq. 6A.4.2.1-1

A1A.1.7—Evaluation Factors (for Strength Limit States)

1. Resistance Factor, φ — LRFD Design 6.5.4.2
   $\varphi = 1.00$ for flexure and shear

2. Condition Factor, φ_c — 6A4.2.3
   $\varphi_c = 1.0\ 0$ Member is in good condition. NBI Item 59 = 7.

3. System Factor, φ_s — 6A.4.2.4
   $\varphi_s = 1.00$ 4-girder bridge, spacing > 4 ft (for flexure and shear).

A1A.1.8—Design Load Rating (6A.4.3)

A1A.1.8.1—Strength I Limit State (6A.6.4.1)

Capacity $C = (\varphi_c)(\varphi_s)(\varphi)R_n$

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

A1A.1.8.1a—Inventory Level

Load Factors — Table 6A.4.2.2-1

γ_{DC} 1.25
γ_{DW} 1.50
γ_{LL} 1.75

The dead load demands established for load cases DC_1 and DC_2 are permanent loads and therefore the load factor for these loads will be taken from the load case DC.

Flexure: $= \dfrac{RF = (1.0)(1.0)(1.0)(2{,}873.0)-(1.25)(439.9+129.4)}{(1.75)(954.10)}$

$= 1.29754$

Note: The general rule for simple spans carrying moving concentrated loads states: the maximum bending moment produced by moving concentrated loads occurs under one of the loads when that load is as far from one support as the center of gravity of all the moving loads on the beam is from the other support. In a refined analysis with the HL-93 truck located in such a manner, the resulting rating factor for flexure is RF = 1.2922 for this stringer. It should be understood that locating the precise critical section and load position for rating depends on the combined influence of dead load, live load, member capacity, and load factors that make up the general rating factor equation.

Shear: $$RF = \frac{(1.0)(1.0)(1.0)(360.15)-(1.25)(27.1+8.0)}{(1.75)(78.9)}$$

$$= 2.435$$

A1A.1.8.1b—Operating Level

Load Factor γ — Table 6A.4.2.2-1

Load	Load Factor γ
DC	1.25
DW	1.50
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: $$RF = 1.294\times\frac{1.75}{1.35}$$

$$= 1.677$$

Shear: $$RF = 2.435\times\frac{1.75}{1.35}$$

$$= 3.156$$

A1A.1.8.2—Service II Limit State (6A.6.4.1)

Capacity $C = f_R$

$$RF = \frac{f_R-(\gamma_{DC})(f_{DC})-(\gamma_{DW})(f_{DW})\pm(\gamma_P)(f_P)}{(\gamma_{LL})(f_{LL+IM})}$$ Eq. 6A.6.4.2.1-1

For this example, the terms:

$$(\gamma_{DW})(f_{DW})\pm(\gamma_P)(f_P)$$

do not contribute and the general equation reduces to:

$$RF = \frac{f_R-(\gamma_{DC})(f_{DC})}{(\gamma_{LL})(f_{LL+IM})}$$

A1A.1.8.2a—Inventory Level

Allowable Flange Stress for tension flange $f_R = 0.95R_hF_{yf}$ ($f_\ell = 0$) LRFD Design Eq. 6.10.4.2.2-2

Checking the tension flange as compression flanges typically do not govern for composite sections.

R_h = 1.0 for non-hybrid sections LRFD Design 6.10.1.10.1

f_R = 0.95 × 1.0 × 36

$= 34.2$ ksi

$f_{DC} = f_{DC_1} + f_{DC_2}$

$$f_{DC} = \frac{M_{DC1}}{S_b} + \frac{M_{DC2}}{S_{b3n}}$$

$$= \frac{439.9 \times 12}{563.8} + \frac{129.4 \times 12}{723.4}$$

$= 9.363 + 2.147 = 11.510$ ksi

$$f_{LL+IM} = \frac{M_{LL+IM}}{S_b n}$$

$$f_{LL+IM} = \frac{954.1 \times 12}{792.4} = 14.449 \text{ ksi}$$

$\gamma_{LL} = 1.30 \qquad \gamma_{DC} = 1.00$ Table 6A.4.2.2-1

$$RF = \frac{34.2 - (1.0)(11.510)}{(1.3)(14.449)}$$

$= 1.208$

A1A.1.8.2b—Operating Level

$\gamma_{LL} = 1.00 \qquad \gamma_{DC} = 1.00$ Table 6A.4.2.2-1

For Service II Operating Level, only the live-load factor changes; therefore, the rating factor can be calculated by direct proportions as well.

$$RF = 1.208 \times \frac{1.30}{1.00}$$

$= 1.570$

A1A.1.8.3—Fatigue Limit State (6A.6.4.1)

Determine if the bridge has any fatigue-prone details (Category C or lower).

The transverse welds detail connecting the ends of cover plates to the flange are fatigue-prone details. Use Category E′ details because the flange thicknesss = 0.855 in. is greater than 0.8 in. LRFD Design Table 6.6.1.2.3-1

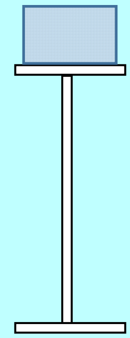
If $2.2(\Delta f)_{tension} > f_{dead\text{-}load\ compression}$, the detail may be prone to fatigue. Eq. 7.2.3-1

$f_{dead\text{-}load\ compression}$ 7.2.3

= 0.0 at cover plate at all locations because beam is a simple span and cover plate is located in the tension zone

∴ must consider fatigue; determine if the detail possesses infinite life.

Composite section properties without cover plate:



$$\bar{y} = \frac{(38.26)(16.55) + \left(\frac{88}{9.2} \times 7.25\right)(36.725)}{(38.26) + \left(\frac{88}{9.2} \times 7.25\right)}$$

$=$ 29.552 in. from bottom of section to centroid

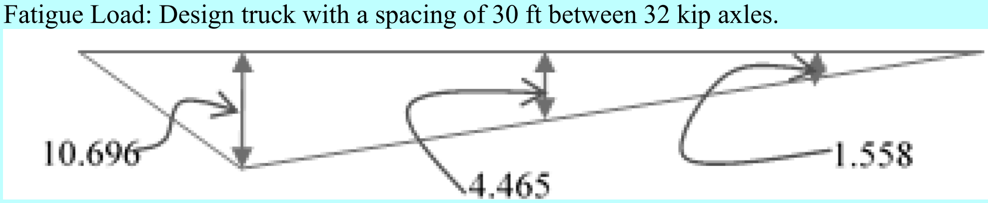
$$I_x = 6{,}699 + (38.26)(13.002)^2 + \frac{\left(\frac{88}{9.2}\right)(7.25)^3}{12} + \frac{88}{9.2}(7.25)(7.173)^2$$

$=$ 17,038.8 in.^{4}

$S_b = \dfrac{17{,}038.8}{29.552} = 576.57$ in.^{3} Section Modulus at bottom of steel

Live Load at Cover Plate Cut-Off (13.5 ft. from centerline of bearing)

Fatigue Load: Design truck with a spacing of 30 ft between 32 kip axles. LRFD Design 3.6.1.4.1 and Figure 3.6.1.2.2-1



Influence line ordinates for moment at 13.5 ft from support

M_{LL} = (32 kips) (10.696 ft) + (32 kips) (4.465 ft) + (8 kips) (1.558 ft) Using influence lines.

= 497.62 kip-ft = 5,971.0 kip-in.

IM = 15 percent LRFD Design Table 3.6.2.1-1

M_{LL+IM} = (1.15) (5,971) = 6,866.7 kip-in.

A1A.1.8.3a—Load Distribution for Fatigue LRFD Design 3.6.1.4.3b

The single-lane distribution factor will be used for fatigue. LRFD Design 3.6.1.1.2

Remove multiple presence factor from the single-lane distribution. LRFD Design C3.6.1.1.2

$$g_{Fatigue} = \frac{g_{m1}}{1.20}$$

$$= \frac{0.460}{1.20}$$

$= 0.383$

Distributed Live-Load Moment:

gM_{LL+IM} = (0.383) (6866.7)

= 2,629.9 kip-in.

Fatigue Load Stress Range:

$$\Delta f_{LL+IM} = \frac{2{,}629.9}{576.57}$$

= 4.561 ksi at the cover plate weld

Nominal fatigue resistance for infinite life:

$(\Delta F)_{TH}$ = 2.60 ksi for Detail Category E′ LRFD Design Table 6.6.1.2.5-3

Infinite-Life Fatigue Check: 7.2.4

$(ADTT_{present}) = 700$
Span Length $(L) = 65.00$ ft
Number of lanes $(n_L) = 2$

R_p $= 0.988 + 6.87\text{x}10^{-5}$ Span Length $+ 4.01\text{x}10^{-6}\ (ADTT)_{PRESENT} +$ 0.0107 / Number of lanes 7.2.2.1
$= 0.988 + 6.87\text{x}10^{-5}*65 + 4.01\text{x}10^{-6}*700+0.0107/2$
$= 1.00062$

$(\Delta f)_{max} = (R_p)(\Delta f_{FATIGUE\ I}) = (1.00)(1.75)(4.56)$ 7.2.4
$= 1.00062 \times 1.75 \times 4.561$ LRFD Design
$= 7.987$ ksi > 2.6 ksi Table 3.4.1-1

Fatigue Rating Factor for Infinite Life

$$RF_{infinite} = \frac{(\Delta F)_{TH}}{(\Delta f_{LL+IM})_{\max}} = \frac{2.60}{7.987} = 0.326$$

(2018 Interim) Table 6A.4.2.2-1

And, $(\Delta f)_{max} > (\Delta F)_{TH}$

Therefore, the detail does not possess infinite fatigure life.

Evaluate the estimated remaining fatigue life using procedures given in Section 7.

Fatigue Rating Factor for Finite Life

$(\Delta f)_{max} = R_p \times \Delta_{FATIGUE\text{-}II}$
$= R_p \times Y_{LL\text{-}fatigue\text{-}II} \times \Delta f_{LL+IM}$
$= 1.00062 \times 0.80 \times 4.561$
$= 3.651$ ksi > 2.6 ksi

$$RF_{infinite} = \frac{(\Delta F)_{TH}}{(\Delta f_{LL+IM})_{\max}} = \frac{2.60}{3.651} = 0.712$$

A1A.1.8.3b—Calculation of Finite Fatigue Life

Fatigue life determination will be based upon the finite fatigue life.

ADTT (One Direction) = 700 (present value) LRFD Design
$[ADTT_{SL}]_{PRESENT} = 0.85(700) = 595$ Table 3.6.1.4.2-1

Traffic Growth Rate g: 1.0 percent is applied over the life of the bridge (input as 0.010)
Bridge Age a: (2019–1964) = 55 years

Assume Evaluation 1 Life to be used for bridge assessment.
Hence, $R_R = 1.30$ Table 7.2.5.1-1

Calculate effective stress range:

$R_p = 1.00062$
$R_{sa} = 1.000$ Table 7.2.2.1-1
$R_{st} = 1.000$
$R_s = R_{sa} \times R_{st} = 1.000$

$\Delta f_{eff} = (R_p)(R_s)(\Delta f_{FATIGUE\ II}) = (1.00062)(1.000)(0.80)(4.561) = 3.651$ ksi 7.2.2

$A = 3.90 \times 10^8$ ksi^3 LRFD Design Table 6.6.1.2.5-1

$n = 1.00$ simple span girders LRFD Design Table 6.6.1.2.5-2

Check that there is remaining fatigue life at the present age. Noting that $(ADTT_{SL})_{PRESENT} \neq (ADTT_{SL})_o$), that is, 7.2.5.1

$$N_{av} > N_1$$

$$N_{av} = \frac{R_R A}{\left(\Delta f_{eff}\right)^3} = \frac{1.3 \times 3.9 \times 10^8}{3.651^3} = 10{,}417{,}718 \text{ cycles}$$

$$N_1 = 365n\left(ADTT_{SL}\right)_{PRESENT}\left[\frac{1-\dfrac{\left(ADTT_{SL}\right)_o}{\left(ADTT_{SL}\right)_{PRESENT}}}{\left(\dfrac{\left(ADTT_{SL}\right)_{PRESENT}}{\left(ADTT_{SL}\right)_o}\right)^{\frac{1}{a}}-1}+1\right]$$

$$N_1 = 365(1)(595)\left[\frac{1-\dfrac{200}{595}}{\left(\dfrac{595}{200}\right)^{\frac{1}{55}}-1}+1\right] = 7{,}418{,}583\ cycles < N_{av} \text{ OK}$$

Calculate the estimated remaining fatigue life, Y_{REM}, of the fatigue-prone detail as follows:

$$Y_{REM} = \frac{\log\left[\left(\dfrac{g}{g+1}\right)\left(\dfrac{N_{av}-N_1}{365n\left(ADTT_{SL}\right)_{PRESENT}}\right)+1\right]}{\log(1+g)}$$

$$= \frac{\log\left[\left(\dfrac{0.01}{1+0.01}\right)\left(\dfrac{10{,}417{,}718-N_1}{365*1*595}\right)+1\right]}{\log(1+0.01)} = 12.8 \text{ years}$$

Check the following:

$$\left(ADTT_{SL}\right)_{FUTURE} \leq \left(ADTT_{SL}\right)_{LIMIT}$$

$$\left(ADTT_{SL}\right)_{FUTURE} = \left[\left(ADDT_{SL}\right)_{PRESENT}\right](1+g)^{Y_{REM}}$$

$$= (595)(1+0.01)^{12.8}$$

$$= 676 < \left(ADTT_{SL}\right)_{LIMIT} = 1{,}200 \text{ OK}$$

A1A.1.8.3c—Calculation of Fatigue Serviceability Index

Fatigue Serviceability Index $Q = \left(\dfrac{Y-a}{N}\right)GRI$ Eq. 7.2.6.1-1

No. of load paths (in this case, girders) = 4

$G = 1.00$ Table 7.2.6.1-1

No. of Spans = 1

$R = 0.90$ Table 7.2.6.1-2

$Y = a + Y_{REM} = 54 + 12.8 = 66.8$ years
N = (larger of 100 or Y) = 100.0 7.2.6.1

Since this bridge is on an Interstate Highway, Table 7.2.6.1-3

$I = 0.9$

$$Q = \left(\frac{66.8 - 54}{100}\right)(1.0)(0.9)(0.9) = 0.1040$$

Based on the value of the Fatigue Serviceability Index, the bridge owner will need to define the inspection frequency based upon the importance of the structure. Note, however, that the Fatigue Serviceability Index value could be increased if the owner decided to accept a greater risk of fatigue cracking and use an Evaluation 2 Life estimate instead of the Evaluation 1 Life estimate. This is illustrated below for the same example.

Assume that Evaluation 2 Life is used for the bridge fatigue assessment.
Hence, $R_R = 1.60$ Table 7.2.5.1-1
Calculate effective stress range:
$(\Delta f)_{eff} = 3.65$ ksi See calculations in A1A.1.8.3b

$A = 3.90 \times 10^8$ LRFD Design Table 6.6.1.2.5-1

$n = 1.0$ simple span girders LRFD Design Table 6.6.1.2.5-2

$$N_{av} = \frac{R_R A}{\left[\left(\Delta f_{eff}\right)^3\right]} = \frac{1.6 \times 3.9 \times 10^8}{\left(3.651\right)^3} = 12{,}821{,}803 \text{ cycles}$$

$$Y = \frac{\log\left[\left(\frac{0.01}{1+0.01}\right)\left(\frac{12{,}821{,}803 - 7{,}418{,}583}{365(1.0)(595)}\right)+1\right]}{\log\left(1+0.01\right)} = 22.1 \text{ years}$$

FATIGUE SERVICEABILITY INDEX

Fatigue Serviceability Index $Q = \left(\frac{Y-a}{N}\right)GRI$ 7.2.6.1

No. of load paths (in this case, girders) = 4
$G = 1.00$ Table 7.2.6.1-1
No. of Spans = 1
$R = 0.90$ Table 7.2.6.1-2
N = (larger of 100 or Y) = 100
$Y = Y_{REM} = 25.2 + 48 = 73.2$
Assuming that the bridge is on an Interstate Highway, $I = 0.9$ Table 7.2.6.1-3

$$Q = \left(\frac{76.1 - 54}{100}\right)(1.0)(0.9)(0.9) = 0.179$$

Note that the Fatigue Serviceability Index, Q, has increased from 0.104 to 0.179 as a result of accepting a greater risk of fatigue cracking at the critical detail.

A1A.1.9—Legal Load Rating 6A.6.4.2

Note: The Inventory Design Load Rating produced rating factors greater than 1.0 (with the exception of fatigue). This indicates that the bridge has adequate load capacity to carry all legal loads within LRFD exclusion limits (as stated in LRFD Design Article C3.6.1.2.1) and need not be subject to legal load ratings.

The load rating computations that follow have been done for illustrative purposes. Shear ratings have not been illustrated.

A1A.1.9a—Live Load: AASHTO Legal Loads—"Routine Comercial Traffic"—Type 3, 3S2, 3-3 (rate for all three)

From previous calculations, g_m = 0.627
From previous calculations, g_v = 0.767

IM = 20 percent Please note that the standard dynamic load allowance of 33 percent is decreased based on a field evaluation verifying that the approach and bridge riding surfaces have only minor surface deviations or depressions. Table C6A.4.4.3-1

The following table compares interpolating to determine M_{LL} without impact for 65 ft span with exact values determined by statics. Note that for the Type 3-3, interpolating M_{LL} results in a value that is 1.5 percent greater than the true value. Judgement should be exercised whether to interpolate tabulated values. Table E6A-1 (with 33 percent *IM*)

Since shear demands for simple spans are not listed in the MBE, the shear demands (without impact) are established using statics and listed below. Hand Calculations (not shown)

Table A1A.1.9-1—AASHTO Routine Legal Load Demands for Interior Stringer

	Type 3	Type 3S2	Type 3-3	
M_{LL} interpolated	660.70	707.20	654.40	kip-ft
M_{LL} statics	660.77	707.03	644.68	kip-ft
$g_m\ M_{LL+IM}$	497.2	532.0	485.1	kip-ft
V_{LL} statics	44.28	51.38	50.58	kip
$g_v\ M_{LL+IM}$	40.75	47.29	46.55	kip

A1A.1.9b—Live Load: AASHTO Legal Loads—Specialized Hauling Vehicles and Notional Rating Load— SU4, SU5, SU6, SU7, and NRL

Interpolated values are used for the Specialized Hauling Vehicles in this example for illustrative purposes and to familiarize the reader with the Appendix A tables. Table E6A-2 (with 33 percent *IM*)

The moment demands are established by interpolating demands listed in Table 6A-2; the shear demands are established using statics.

Table A1A.1.9-2—AASHTO Specialized Hauling Vehicles Load Demands for Interior Stringer

	SU4	SU5	SU6	SU7	NRL	Unit
M_{LL+IM} interpolated	744.7	821.2	913.5	994.1	1,037	kip-ft
$g_m\ M_{LL+IM}$	560.3	617.9	687.3	748	780.2	kip-ft
V_{LL+IM} statics	48.65	54.43	58.31	62.04	63.01	kip
$g_v\ V_{LL+IM}$	44.78	50.10	53.67	57.10	57.99	kip

A1A.1.9.1—Strength I Limit State 6A.6.4.2.1

For Types 3, 3S2, and 3-3

Dead Load DC: γ_{DC} = 1.25 Table 6A.4.2.2-1

$ADTT$ (One Direction) = 700

Generalized Live-Load Factor for Legal Loads, γ_{LL} = 1.30 Table 6A.4.4.2.3b-1

Flexure: $RF = \dfrac{(1.0)(1.0)(1.0)(2{,}873.0)-(1.25)(439.9+129.4)}{(1.30)(M_{LL+IM})}$

Shear: $RF = \dfrac{(1.0)(1.0)(1.0)(380.15)-(1.25)(27.1+8.0)}{(1.30)(V_{LL+IM})}$

Table A1A.1.9.1-1—(Strength I) Rating Factors for AASHTO SHV Vehicles

	SU4	SU5	SU6	SU7	NRL
RF (Flexure)	2.967	2.691	2.419	2.223	2.131
RF (Shear)	5.777	5.163	4.820	4.530	4.461

A1A.1.9.2—Service II Limit State 6A.6.4.2.2

For Types 3, 3S2, and 3-3, and for Specialized Hauling Units and NRL

Generalized Live-Load Factor for Legal Loads: $\gamma_{LL} = 1.3$ Table 6A.4.2.2-1
Dead Load *DC*: $\gamma_{DC} = 1.0$

$f_R = 34.200$ ksi

$f_D = f_{DC_1} + f_{DC_2} = 11.510$ ksi

$$f_{LL+IM} = \frac{M_{LL+IM} \times 12}{792.4}$$

$$RF = \frac{34.2-(1.0)(11.510)}{(1.3)(f_{LL+IM})}$$

Table A1A.1.9.2-1—(Service) Rating Factors for AASHTO Routine Legal Vehicles

	Type 3	Type 3S2	Type 3-3
f_{LL+IM}	7.530	8.057	7.346
RF (Service II)	2.318	2.166	2.376

Table A1A.1.9.2-2—(Service) Rating Factors for AASHTO SHV Legal Vehicles

	SU4	SU5	SU6	SU7	NRL
f_{LL+IM}	8.485	9.357	10.408	11.328	11.815
RF (Service II)	2.057	1.865	1.677	1.541	1.477

No posting required as $RF > 1.0$.

A1A.1.9.3—Summary

Safe Load Capacity (tons), $RT = RF \times W$ Eq. 6A.4.4.4-1

Truck	Type 3	Type 3S2	Type 3-3
Weight (tons)	25	36	40
RF (Service II—Controls)	2.318	2.166	2.376
Safe Load Capacity (tons)	58	78	95

Truck	SU4	SU5	SU6	SU7	NRL
Weight (tons)	27	31	34.75	38.75	40
RF (Service II—Controls)	2.057	1.865	1.677	1.541	1.477
Safe Load Capacity (tons)	56	58	58	60	59

The NRL rating demonstrates Article C6A.4.4.2.1b: "Bridges that rate for the NRL loading will have adequate load capacity for all legal Formula B truck configurations up to 80 kips." Example A1 shows this holding true: NRL $RF > 1$ and all SU $RF > 1$, while Example A2 shows when NRL $RF < 1$, RF for the SUs may or may not be >1 and need to be checked on an individual basis.

A1A.1.10—Permit Load Rating

Permit Type: Special (Single-Trip, Escorted)
Permit Weight: 220 kips
Permit Vehicle: Shown in Figure A1A.1.10-1
ADTT (one direction): 700

Demand from one percent permit truck without impact from live load analysis by computer program:

Maximum $M_{LL} =$ 2,115.0 kip-ft

Maximum $V_{LL} =$ 143.5 kips

A1A.1.10.1—Strength II Limit State

$\gamma_{LL} = 1.10$ Table 6A.4.5.4.2a-1

Use one-lane distribution factor and divide out the 1.2 multiple presence factor. 6A.4.5.4.2b

$$g_{m1\text{-}permit} = \frac{g_{ml}}{1.20} = \frac{0.460}{1.20} = 0.383$$

$$g_{v1\text{-}permit} = \frac{g_{v1}}{1.20} = \frac{0.653}{1.20} = 0.544$$

IM = 20 percent (no speed control, minor surface deviations) 6A.4.5.5

Distributed Live-Load Effects:

$$M_{LL+IM} = 2{,}115 \times 0.383 \times 1.2$$
$$= 972.1 \text{ kip-ft}$$

$$V_{LL+IM} = (143.5)\ (0.544)\ (1.20)$$
$$= 94.90 \text{ kips}$$

Flexure: $$RF = \frac{(1.0)(1.0)(1.0)(2{,}873.0)-(1.25)(439.9+129.4)}{(1.10)(972.1)} = 2.021$$

Shear: $$RF = \frac{(1.0)(1.0)(1.0)(380.15)-(1.25)(27.1+8.0)}{(1.10)(94.9)} = 3.221$$

A1A.1.10.2—Service II Limit State (Optional) 6A.6.4.2.2

$$RF = \frac{f_R-(\gamma_{DC})(f_{DC})}{\gamma_L(f_{LL+IM})}$$

IM = 20 percent (no speed control, minor surface deviations)

Generalized Live-Load Factor: $\gamma_L = 1.00$ Table 6A.4.2.2-1
Dead Load DC: $\gamma_D = 1.00$