STRUCTURAL CALCULATIONS

for

HARTLAND IM 091-1(68)
BRIDGE NO. D37, TOWN HIGHWAY 41 OVER INTERSTATE 91
HARTLAND, VERMONT

Prepared for:

VTrans Working to Get You There
Vermont Agency of Transportation

2ND SET OF STRUCTURAL CALCULATIONS
NOVEMBER 2019
CONTRACT PLAN SUBMISSION

Prepared by:

GILL ENGINEERING

63 Kendrick Street
Needham, MA 02494

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GILL ENGINEERING

CLIENT VTrans
PROJECT Hartland IM 091-1(68)
BRIDGE NO. Bridge D37
SUBJECT Final Design
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DATE June 2019

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SUPERSTRUCTURE CALCULATIONS

Hartland - Span 1 Dead Loads

References:
1. AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017
2. Vtrans Structures Design Manual, Fifth Edition 2010

Bridge Geometry:

exterior girder top bf =	16.00 in	(min)	t_{haunch} =	2.00 in
exterior girder top tf =	0.88 in	(max)	concrete density =	150.0 plf
overhang left =	2.90 ft	(max)	wearing surface thickness =	2.50 in
overhang right =	1.98 ft	(average)	wearing surface density =	150.0 plf
Roadway Width =	27.00 ft		sidewalk width =	0.00 ft
No. Girders =	5		Assumed Exterior Diaphragm Weight =	20.0 plf
Additional Haunch Width =	0.00 in		Assumed Interior Diaphragm Weight =	40.0 plf
interior girder top tf =	0.88 in	(max)		

Calculate DC_1 Loads:

DC_1 = Dead load of structural components and their attachments, acting on non-composite section.

Note: MDX manually calculates the selfweight of the girder as well as the tributary deck weight and haunch and therefore are not included in these calculations.

Girder 1
Additional Concrete:

Area =	77.05 in2 =	(2.00 in + 2 +	0.88 in)* 0.00 in)	(2.90 ft*	12-	16.00 in/
Weight =	80.3 plf =	77.05 in2/	144*	150.0 plf		

Diaphragms:

Weight = 20.0 plf

Girders 2-4
Additional Concrete:

Area =	0.00 in2 =	(2.00 in +	0.88 in)*	0.00 in
Weight =	0.0 plf =	0.00 in2/	144*	150.0 plf

Diaphragms:

Weight = 40.0 plf (assumed)

Girder 5
Additional Concrete:

Area =	45.31 in2 =	(2.00 in + 2 +	0.88 in)* 0.00 in)	(1.98 ft*	12-	16.00 in/
Weight =	47.2 plf =	45.31 in2/	144*	150.0 plf		

Hartland - Span 1 Dead Loads

Calculate DC_2 Loads:

DC_2 = Dead load of structural components and their attachments, acting on composite section.

Ref 2 - 4.3.1.3 - Superimposed dead loads shall be equally distributed to all beams.

	Weight	Quantity
Rail =	50 plf	2
Snow Fence =	20 plf	2
Safety Curb =	238 plf	2

Girders 1-5

Weight = (100.0 plf+ 40.0 plf + 475.0 plf)/ 6 = 123.0 plf

Calculate DW_1 Loads:

DW_1 = Dead load due to utilities

No utilities present on bridge.

Calculate DW_2 Loads:

DW_2 = Dead load due to wearing surface, acting on composite section.

Wearing Surface

Weight = 843.8 plf = 27.00 ft* 2.50 in/ 12* 150.0 plf

All girders = 168.8 plf = 843.8 plf/ 5

MDX input Summary:

Steel weight in addition to girder web and flanges:

Girder 1,5 = 0.020 k/ft
Girders 2-4 = 0.040 k/ft

Concrete weight in addition to slab weight:

Girder 1 = 0.080 k/ft
Girders 2-4 = 0.000 k/ft
Girder 5 = 0.047 k/ft

Composite dead load not including wearing surface:

Girders 1-5 = 0.123 k/ft

Wearing surface load:

Girders 1-5 = 0.169 k/ft

Hartland - Span 2 - Dead Loads

References:
1. AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017
2. Vtrans Structures Design Manual, 2010

Bridge Geometry:

exterior girder top bf =	16.00 in	(min)	t_{haunch} =	2.00 in
exterior girder top tf =	0.88 in	(max)	concrete density =	150.0 plf
overhang left =	3.13 ft	(max)	wearing surface thickness =	2.50 in
overhang right =	1.82 ft	(average)	wearing surface density =	150.0 plf
Roadway Width =	27.00 ft		sidewalk width =	0.00 ft
No. Girders =	5		Assumed Exterior Diaphragm Weight =	20.0 plf
Additional Haunch Width =	0.00 in		Assumed Interior Diaphragm Weight =	40.0 plf
interior girder top tf =	0.88 in	(max)		

Calculate DC_1 Loads:

DC_1 = Dead load of structural components and their attachments, acting on non-composite section.

Note: MDX manually calculates the selfweight of the girder as well as the tributary deck weight and haunch and therefore are not included in these calculations.

Girder 1
Additional Concrete:

Area =	84.99 in2 =	(2.00 in + 2 +	0.88 in)* 0.00 in)	(3.13 ft*	12-	16.00 in/
Weight =	88.5 plf =	84.99 in2/	144*	150.0 plf		

Diaphragms:
Weight = 20.0 plf

Girders 2-4
Additional Concrete:

Area =	0.00 in2 =	(2.00 in +	0.88 in)*	0.00 in
Weight =	0.0 plf =	0.00 in2/	144*	150.0 plf

Diaphragms:
Weight = 40.0 plf (assumed)

Girder 5
Additional Concrete:

Area =	39.79 in2 =	(2.00 in + 2 +	0.88 in)* 0.00 in)	(1.82 ft*	12-	16.00 in/
Weight =	41.4 plf =	39.79 in2/	144*	150.0 plf		

Diaphragms:
Weight = 20.0 plf

Hartland - Span 2 - Dead Loads

Calculate DC_2 Loads:

DC_2 = Dead load of structural components and their attachments, acting on composite section.

Ref 2 - 4.3.1.3 - Superimposed dead loads shall be equally distributed to all beams.

	Weight	Quantity
Rail =	50 plf	2
Snow Fence =	20 plf	2
Safety Curb =	238 plf	2

Girders 1-5

Weight = (100.0 plf+ 40.0 plf + 475.0 plf)/ 6 = 123.0 plf

Calculate DW_1 Loads:

DW_1 = Dead load due to utilities

No utilities present on bridge.

Calculate DW_2 Loads:

DW_2 = Dead load due to wearing surface, acting on composite section.

Wearing Surface

Weight = 843.8 plf = 27.00 ft* 2.50 in/ 12* 150.0 plf

All girders = 168.8 plf = 843.8 plf/ 5

MDX input Summary:
Steel weight in addition to girder web and flanges:

Girder 1,5 = 0.020 k/ft
Girders 2-4 = 0.040 k/ft

Concrete weight in addition to slab weight:

Girder 1 = 0.089 k/ft
Girders 2-4 = 0.000 k/ft
Girder 5 = 0.041 k/ft

Composite dead load not including wearing surface:

Girders 1-5 = 0.123 k/ft

Wearing surface load:

Girders 1-5 = 0.169 k/ft

Hartland - Span 1 - Live Load Distribution Factors

References:
1. AASHTO LRFD Bridge Design Specifications, 2017, 8th Edition
2. Vtrans 2010 Structures Design Manual

S_{int} =	6.67 ft	$W_{scurb\ L}$ =	2.00 ft	L =	83.94 ft
$S_{scurb\ L}$ =	6.67 ft	$W_{scurb\ R}$ =	2.00 ft	N_{beams} =	5
$S_{scurb\ R}$ =	6.67 ft	$O_{hang\text{-}scurb\ L}$ =	2.90 ft	n =	7.27
t_s =	9.00 in	$O_{hang\text{-}scurb\ R}$ =	2.17 ft	skew =	10.49 deg
W_r =	27.00 ft			N_L =	2

	G2-G4		G1		G5
I =	11007 in4	I =	11007 in4	I =	11007 in4
A =	46.88 in2	A =	46.88 in2	A =	46.88 in2
d =	36.00 in	d =	36.00 in	d =	36.00 in
N.A. =	20.12 in	N.A. =	20.12 in	N.A. =	20.12 in

Multiple Presence	
M-1 =	1.2
M-2 =	1
M-3 =	0.85
M->3 =	0.65

Pedestrian load is considered as a loaded lane for the purpose of determing the multiple presence factor. However, this use of the pedestrian load as a loaded lane shall only apply to the design of exterior beams or other interior sidewalk beams and not for the design of interior roadway beams, even though part of the pedestrian load is applied.

<u>Moment Distribution Factor Calculations</u>

Interior Beams:

Per Ref 2 - Section 4.3.2, Distribution of live load to interior beams shall be calculated using Ref 1 - Section 4.6.2.2.2

Ref 1 - Table 4.6.2.2.2b-1:

Range of Applicability					
3.5	≤	S ≤	16		OK
4.5	≤	t_S ≤	12		OK
20	≤	L ≤	240		OK
		N_b ≥	4		OK
10000	≤	Kg ≤	7000000		OK

For one design lane loaded:

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

$$K_g = n\left(I + Ae_g^2\right) \qquad n = \frac{E_b}{E_d}$$

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

e_g 24.62 in = 4.5 in + 20.12 in

K_g 286782 in4 = 7.27 x (11007 in4 + 46.88 in2 x (24.62 in)^2)

$DF_{M\text{-}INT\text{-}1}$ 0.376 $= 0.06 + \left(\frac{6.67\text{ ft}}{14}\right)^{0.4}\left(\frac{6.67\text{ ft}}{83.94\text{ ft}}\right)^{0.3}\left(\frac{286782\text{ in4}}{12 \times 83.94\text{ ft} \times (9.00\text{ in})^3}\right)^{0.1}$

For two or more design lanes loaded:

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

$DF_{M\text{-}INT\text{-}2}$ 0.518 $= 0.08 + \left(\frac{6.67\text{ ft}}{9.5}\right)^{0.6}\left(\frac{6.67\text{ ft}}{83.94\text{ ft}}\right)^{0.2}\left(\frac{286782\text{ in4}}{12 \times 83.94\text{ ft} \times (9.00\text{ in})^3}\right)^{0.1}$

Hartland - Span 1 - Live Load Distribution Factors

Exterior Beams under Safety Curb or Barriers:

Per Ref 2 - Section 4.3.2, Distribution of live load to exterior beams under safety curb or barrier shall be calculated using Ref 1 - Section 4.6.2.2.2

Per Ref 1 - Section 4.6.2.2.2d - Exterior beam distribution factors shall be taken as the larger of those calculated using Table 4.6.2.2.2d-1, or the pile cap analogy as outlined in C4.6.2.2.2d

Safety Curb Beam (G1)

Ref 1 - Table 4.6.2.2.2b-1:

Range of Applicability				
-1.0	$\le$	d_e $\le$	5.5	OK
3	$<$	N_b		OK

e_g 24.62 in = 4.50 in + 20.12 in

K_g 286782 in4 = 7.27 x (11007 in4 + 46.88 in2 x (24.62 in)^2)

One Design Lane Loaded:

Lever Rule

$DF_{M\text{-}EXT\text{-}1}$ **0.501** = ((0.5 x 66.80 in + 0.5* 0.00 in)/ 80.00 in) x 1.20

Two Design Lanes Loaded:

d_e 0.90 ft = 2.90 ft - 2.00 ft

$e = 0.77 + d_e/9.1 =$ 0.87 = 0.77+ 0.90 ft/ 9.1

$DF_{M\text{-}EXT\text{-}2}$ **0.451** = 0.87 x 0.52

"Rigid" Superstructure (Pile Cap Analogy)

$$R = \frac{N_L}{N_b} + \frac{X_{ext}\sum^{N_L} e}{\sum^{N_b} x^2}$$

	X	X^2
G1	13.33 ft	177.78 ft2
G2	6.67 ft	44.44 ft2
G3	0.00 ft	0.00 ft2
G4	-6.67 ft	44.44 ft2
G5	-13.33 ft	177.78 ft2
	Σ =	444.44 ft2

N_L = Number of loaded lanes under consideration

e = Eccentricity of a design truck or a design lane load from the C.O.G. of the pattern of girders (ft)

x = Horizontal distance from the C.O.G. of the pattern of girders to each girder (ft)

X_{ext} = Horizontal distance from the C.O.G. of the pattern of girders to the exterior girder (ft)

e_1 9.23 ft = 13.33 ft + 2.90 ft - 2.00 ft - 2.00 ft - 3.00 ft

e_2 -2.77 ft = 13.33 ft + 2.90 ft - 2.00 ft - 2.00 ft - 15.00 ft

R_1 $0.48 = \dfrac{1.00}{5.00} + \dfrac{(13.33\text{ ft} \times 9.23\text{ ft})}{444.44\text{ ft2}}$

R_2 $0.59 = \dfrac{2.00}{5.00} + \dfrac{(13.33\text{ ft} \times (9.23\text{ ft} + -2.77\text{ ft}))}{444.44\text{ ft2}}$

For one design lane loaded

$DF_{M\text{-}EXT\text{-}1}$ **0.572** = 0.48 x 1.2

For two design lanes loaded

$DF_{M\text{-}EXT\text{-}2}$ **0.594** = 0.59 x 1.00

Hartland - Span 1 - Live Load Distribution Factors

Safety Curb Beam (G5)
Ref 1 - Table 4.6.2.2.2b-1:

Range of Applicability				
-1.0	≤ d_e ≤	5.5		OK
3	< N_b			OK

e_g 24.62 in = 4.50 in + 20.12 in

K_g 286782 in4 = 7.27 x $\left(11007\ \text{in4} + 46.88\ \text{in2} \times \left(24.62\ \text{in}\right)^2\right)$

One Design Lane Loaded:
Lever Rule

$DF_{M\text{-}EXT\text{-}1}$ **0.435** = ((0.5 x 58.00 in + 0.5* 0.00 in)/ 80.00 in) x 1.20

Two Design Lanes Loaded:

d_e 0.17 ft = 2.17 ft - 2.00 ft

$e = 0.77 + d_e/9.1$ = 0.79 = 0.77+ 0.17 ft/ 9.1

$DF_{M\text{-}EXT\text{-}2}$ **0.409** = 0.79 x 0.52

"Rigid" Superstructure (Pile Cap Analogy)

	X	X^2
G1	13.33 ft	177.78 ft2
G2	6.67 ft	44.44 ft2
G3	0.00 ft	0.00 ft2
G4	-6.67 ft	44.44 ft2
G5	-13.33 ft	177.78 ft2
	Σ =	444.44 ft2

$$R = \frac{N_L}{N_b} + \frac{X_{ext}\sum^{N_L} e}{\sum^{N_b} x^2}$$

N_L = Number of loaded lanes under consideration

e = Eccentricity of a design truck or a design lane load from the C.O.G. of the pattern of girders (ft)

x = Horizontal distance from the C.O.G. of the pattern of girders to each girder (ft)

X_{ext} = Horizontal distance from the C.O.G. of the pattern of girders to the exterior girder (ft)

e_1 8.50 ft = 13.33 ft + 2.17 ft - 2.00 ft - 2.00 ft - 3.00 ft

e_2 -3.50 ft = 13.33 ft + 2.17 ft - 2.00 ft - 2.00 ft - 15.00 ft

R_1 0.46 = $\frac{1.00}{5.00} + \frac{(13.33\ \text{ft} \times 8.50\ \text{ft})}{444.44\ \text{ft2}}$

R_2 0.55 = $\frac{2.00}{5.00} + \frac{(13.33\ \text{ft} \times (8.50\ \text{ft} + -3.50\ \text{ft}))}{444.44\ \text{ft2}}$

For one design lane loaded

$DF_{M\text{-}EXT\text{-}1}$ **0.546** = 0.46 x 1.2

For two design lanes loaded

$DF_{M\text{-}EXT\text{-}2}$ **0.550** = 0.55 x 1.00

Hartland - Span 1 - Live Load Distribution Factors

Skew Reduction Factor: Ref 1 - Table 4.6.2.2.2e-1

For any number of lanes loaded

$$Reduction\ Factor = 1 - c_1(tan\theta)^{1.5}$$

$$c_1 = 0.25\left(\frac{K_g}{12.0Lt_s^3}\right)^{.25}\left(\frac{S}{L}\right)^{0.5}$$

Range of Applicability

30	≤ Skew ≤	60	C1 = 0
3.5	≤ S ≤	16	OK
20	≤ L ≤	240	OK
4	≤ N_b		OK

$c_{1\ interior\ (G2\text{-}4)}$ 0.000 $= 0.25 \times \left(\frac{286782\ in4}{12 \times 83.94\ ft \times (9.00\ in)^3}\right)^{0.25}\left(\frac{6.67\ ft}{83.94\ ft}\right)^{0.5}$

$c_{1\ safety\ curb\ (G1)}$ 0.000 $= 0.25 \times \left(\frac{286782\ in4}{12 \times 83.94\ ft \times (9.00\ in)^3}\right)^{0.25}\left(\frac{6.67\ ft}{83.94\ ft}\right)^{0.5}$

$c_{1\ safety\ curb\ (G5)}$ 0.000 $= 0.25 \times \left(\frac{286782\ in4}{12 \times 83.94\ ft \times (9.00\ in)^3}\right)^{0.25}\left(\frac{6.67\ ft}{83.94\ ft}\right)^{0.5}$

$RF_{interior\ (G2\text{-}4)}$ 1.000 $= 1 - 0.000 \times (0.185\ rad)^{1.5}$

$RF_{safety\ curb\ (G1)}$ 1.000 $= 1 - 0.000 \times (0.185\ rad)^{1.5}$

$RF_{safety\ curb\ (G5)}$ 1.000 $= 1 - 0.000 \times (0.185\ rad)^{1.5}$

<u>Shear Distribution Factor Calculations</u>

Interior Beams: Ref 1 - Table 4.6.2.2.3A-1

Range of Applicability

3.5 ≤	S	≤ 16	OK
4.5 ≤	t_S	≤ 12	OK
20 ≤	L	≤ 240	OK
	N_b	≥ 4	OK

For one design lane loaded

$$DF = 0.36 + \frac{S}{25}$$

$DF_{S\text{-}INT\text{-}1}$ 0.627 $= 0.36 + (6.7\ ft / 25)$

For two or more design lanes loaded

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

$DF_{S\text{-}INT\text{-}2}$ 0.719 $= 0.2 + \frac{6.67\ ft}{12} - \left(\frac{6.67\ ft}{35}\right)^{2.0}$

Hartland - Span 1 - Live Load Distribution Factors

Exterior Beams: Ref 1 - Table 4.6.2.2.3b-1, C4.6.2.2.2d

Safety Curb Beam (G1)

Range of Applicability					
-1.0	≤	d_e	≤	5.5	OK
3	<	N_b			OK

For one design lane loaded
Use lever rule (See moment distribution factor calculations)

$DF_{S\text{-}EXT\text{-}1}$ 0.501

For two or more design lanes loaded

$e = 0.60 + d_e/10 =$ 0.69 = 0.6+ 0.90 ft/ 10

$DF_{S\text{-}EXT\text{-}2}$ 0.496 = 0.69 * 0.72

Pile Cap Analogy (See moment distribution factor calculations)

$DF_{S\text{-}EXT\text{-}1}$ 0.572
$DF_{S\text{-}EXT\text{-}2}$ 0.594

Safety Curb Beam (G5)

Range of Applicability					
-1.0	≤	d_e	≤	5.5	OK
3	<	N_b			OK

For one design lane loaded
Use lever rule (See moment distribution factor calculations)

$DF_{S\text{-}EXT\text{-}1}$ 0.435

For two or more design lanes loaded

$e = 0.60 + d_e/10 =$ 0.62 = 0.6+ 0.17 ft/ 10

$DF_{S\text{-}EXT\text{-}2}$ 0.444 = 0.62 * 0.72

Pile Cap Analogy (See moment distribution factor calculations)

$DF_{S\text{-}EXT\text{-}1}$ 0.546
$DF_{S\text{-}EXT\text{-}2}$ 0.550

Hartland - Span 1 - Live Load Distribution Factors

Obtuse Corner Correction Factor: Ref 1 - Table 4.6.2.2.3c-1

For any number of lanes loaded

$$CORRECTION - FACTOR = 1.0 + 0.20(\frac{12.0Lt_s^3}{K_g})^{0.3} \tan\theta$$

Range of Applicability

3.5	≤ S ≤	16	OK
0	≤ θ ≤	60	OK
20	≤ L ≤	240	OK
	N_b ≥	4	OK

CF interior (G2-4) 1.05 = 1 + 0.20 x $\left(\frac{12 \text{ x } 83.94 \text{ ft x } (9.00 \text{ in})^3}{286782 \text{ in4}}\right)^{0.3}$ x 0.185 rad

CF safety curb (G1) 1.05 = 1 + 0.20 x $\left(\frac{12 \text{ x } 83.94 \text{ ft x } (9.00 \text{ in})^3}{286782 \text{ in4}}\right)^{0.3}$ x 0.185 rad

CF safety curb (G5) 1.05 = 1 + 0.20 x $\left(\frac{12 \text{ x } 83.94 \text{ ft x } (9.00 \text{ in})^3}{286782 \text{ in4}}\right)^{0.3}$ x 0.185 rad

Distribution Factor Summary						
	Moment			Shear		
Girder/Load Case	1 Lane	Mult Lanes	Fatigue	1 Lane	Mult Lanes	Fatigue
G1	0.57	0.59	0.48	0.60	0.62	0.50
G2-G4	0.38	0.52	0.31	0.66	0.75	0.55
G5	0.55	0.55	0.46	0.57	0.58	0.48

<u>Deflection Distribution Factor Calculations</u>

All Girders: Ref 1 - C2.5.2.6.2

For a straight multibeam bridge, the distribution factor for defleciton is equal to the number of lanes divided by the number of beams.

DF = MP*(N_L/N_b) = 0.40

Hartland - Span 2 - Live Load Distribution Factors

References:
1. AASHTO LRFD Bridge Design Specifications, 2017, 8th Edition
2. Vtrans 2010 Structures Design Manual

S_{int} =	6.67 ft	$W_{scurb\ L}$ =	2.00 ft	L =	85.94 ft
$S_{scurb\ L}$ =	6.67 ft	$W_{scurb\ R}$ =	2.00 ft	N_{beams} =	5
$S_{scurb\ R}$ =	6.67 ft	$O_{hang\text{-}scurb\ L}$ =	3.13 ft	n =	7.27
t_s =	9.00 in	$O_{hang\text{-}scurb\ R}$ =	2.17 ft	skew =	28.66 deg
W_r =	27.00 ft			N_L =	2

	G2-G4		G1		G5
I =	11007 in4	I =	11007 in4	I =	11007 in4
A =	46.88 in2	A =	46.88 in2	A =	46.88 in2
d =	36.00 in	d =	36.00 in	d =	36.00 in
N.A. =	20.12 in	N.A. =	20.12 in	N.A. =	20.12 in

Multiple Presence	
M-1 =	1.2
M-2 =	1
M-3 =	0.85
M->3 =	0.65

Pedestrian load is considered as a loaded lane for the purpose of determing the multiple presence factor. However, this use of the pedestrian load as a loaded lane shall only apply to the design of exterior beams or other interior sidewalk beams and not for the design of interior roadway beams, even though part of the pedestrian load is applied.

Moment Distribution Factor Calculations

Interior Beams:

Per Ref 2 - Section 4.3.2, Distribution of live load to interior beams shall be calculated using Ref 1 - Section 4.6.2.2.2

Ref 1 - Table 4.6.2.2.2b-1:

Range of Applicability						
3.5	≤	S	≤	16		OK
4.5	≤	t_S	≤	12		OK
20	≤	L	≤	240		OK
		N_b	≥	4		OK
10000	≤	Kg	≤	7000000		OK

For one design lane loaded:

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

$$K_g = \mathrm{n}\left(I + Ae_g^2\right) \qquad n = \frac{E_b}{E_d}$$

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

e_g 24.62 in = 4.5 in + 20.12 in

K_g 286782 in4 = 7.27 x (11007 in4 + 46.88 in2 x (24.62 in)^2)

$$DF_{M\text{-}INT\text{-}1} \quad 0.373 = 0.06 + \left(\frac{6.67\text{ ft}}{14}\right)^{0.4}\left(\frac{6.67\text{ ft}}{85.94\text{ ft}}\right)^{0.3}\left(\frac{286782\text{ in4}}{12 \times 85.94\text{ ft} \times (9.00\text{ in})^3}\right)^{0.1}$$

For two or more design lanes loaded:

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

$$DF_{M\text{-}INT\text{-}2} \quad 0.515 = 0.08 + \left(\frac{6.67\text{ ft}}{9.5}\right)^{0.6}\left(\frac{6.67\text{ ft}}{85.94\text{ ft}}\right)^{0.2}\left(\frac{286782\text{ in4}}{12 \times 85.94\text{ ft} \times (9.00\text{ in})^3}\right)^{0.1}$$

Hartland - Span 2 - Live Load Distribution Factors

Exterior Beams under Safety Curb or Barriers:
Per Ref 2 - Section 4.3.2, Distribution of live load to exterior beams under safety curb or barrier shall be calculated using Ref 1 - Section 4.6.2.2.2
Per Ref 1 - Section 4.6.2.2.2d - Exterior beam distribution factors shall be taken as the larger of those calculated using Table 4.6.2.2.2d-1, or the pile cap analogy as outlined in C4.6.2.2.2d

Safety Curb Beam (G1)
Ref 1 - Table 4.6.2.2.2b-1:

Range of Applicability		
$-1.0 \le d_e \le 5.5$		OK
$3 < N_b$		OK

e_g 24.62 in = 4.50 in + 20.12 in
K_g 286782 in4 = 7.27 x (11007 in4 + 46.88 in2 x (24.62 in)^2)

One Design Lane Loaded:
Lever Rule

$DF_{M\text{-}EXT\text{-}1}$ **0.522** = ((0.5 x 69.56 in + 0.5* 0.00 in)/ 80.00 in) x 1.20

Two Design Lanes Loaded:
d_e 1.13 ft = 3.13 ft - 2.00 ft
$e = 0.77 + d_e/9.1$ = 0.89 = 0.77+ 1.13 ft/ 9.1

$DF_{M\text{-}EXT\text{-}2}$ **0.461** = 0.89 x 0.52

"Rigid" Superstructure (Pile Cap Analogy)

	X	X^2
G1	13.33 ft	177.78 ft2
G2	6.67 ft	44.44 ft2
G3	0.00 ft	0.00 ft2
G4	-6.67 ft	44.44 ft2
G5	-13.33 ft	177.78 ft2
	Σ =	444.44 ft2

$$R = \frac{N_L}{N_b} + \frac{X_{ext}\sum^{N_L} e}{\sum^{N_b} x^2}$$

N_L = Number of loaded lanes under consideration
e = Eccentricity of a design truck or a design lane load from the C.O.G. of the pattern of girders (ft)
x = Horizontal distance from the C.O.G. of the pattern of girders to each girder (ft)
X_{ext} = Horizontal distance from the C.O.G. of the pattern of girders to the exterior girder (ft)

e_1 9.46 ft = 13.33 ft + 3.13 ft - 2.00 ft - 2.00 ft - 3.00 ft
e_2 -2.54 ft = 13.33 ft + 3.13 ft - 2.00 ft - 2.00 ft - 15.00 ft

R_1 0.48 = $\frac{1.00}{5.00}$ + $\frac{(13.33\text{ ft x } 9.46\text{ ft})}{444.44\text{ ft2}}$

R_2 0.61 = $\frac{2.00}{5.00}$ + $\frac{(13.33\text{ ft x } (9.46\text{ ft} + -2.54\text{ ft}))}{444.44\text{ ft2}}$

For one design lane loaded
$DF_{M\text{-}EXT\text{-}1}$ **0.581** = 0.48 x 1.2

For two design lanes loaded
$DF_{M\text{-}EXT\text{-}2}$ **0.608** = 0.61 x 1.00

Hartland - Span 2 - Live Load Distribution Factors

Safety Curb Beam (G5)
Ref 1 - Table 4.6.2.2.2b-1:

Range of Applicability

-1.0	≤ d_e ≤	5.5		OK
3	< N_b			OK

e_g	24.62 in	=	4.50 in +	20.12 in		
K_g	286782 in4	=	7.27 x	[11007 in4 +	46.88 in2 x	[24.62 in]^2]

One Design Lane Loaded:
Lever Rule

$DF_{M\text{-}EXT\text{-}1}$	**0.435 =**	((0.5 x	58.00 in +	0.5*	0.00 in)/	80.00 in) x	1.20

Two Design Lanes Loaded:

d_e	0.17 ft	=	2.17 ft -	2.00 ft	
$e = 0.77 + d_e/9.1 =$	0.79 =		0.77+	0.17 ft/	9.1
$DF_{M\text{-}EXT\text{-}2}$	**0.406**	=	0.79 x	0.52	

"Rigid" Superstructure (Pile Cap Analogy)

	X	X^2
G1	13.33 ft	177.78 ft2
G2	6.67 ft	44.44 ft2
G3	0.00 ft	0.00 ft2
G4	-6.67 ft	44.44 ft2
G5	-13.33 ft	177.78 ft2
	Σ =	444.44 ft2

$$R = \frac{N_L}{N_b} + \frac{X_{ext}\sum^{N_L} e}{\sum^{N_b} x^2}$$

N_L = Number of loaded lanes under consideration
e = Eccentricity of a design truck or a design lane load from the C.O.G. of the pattern of girders (ft)
x = Horizontal distance from the C.O.G. of the pattern of girders to each girder (ft)
X_{ext} = Horizontal distance from the C.O.G. of the pattern of girders to the exterior girder (ft)

e_1	8.50 ft	=	13.33 ft +	2.17 ft -	2.00 ft -	2.00 ft -	3.00 ft
e_2	-3.50 ft	=	13.33 ft +	2.17 ft -	2.00 ft -	2.00 ft -	15.00 ft

$$R_1 \quad 0.46 = \frac{1.00}{5.00} + \frac{(13.33\text{ ft x } 8.50\text{ ft})}{444.44\text{ ft2}}$$

$$R_2 \quad 0.55 = \frac{2.00}{5.00} + \frac{(13.33\text{ ft x } (8.50\text{ ft}+ \; -3.50\text{ ft}))}{444.44\text{ ft2}}$$

For one design lane loaded

$DF_{M\text{-}EXT\text{-}1}$	**0.546**	=	0.46 x	1.2

For two design lanes loaded

$DF_{M\text{-}EXT\text{-}2}$	**0.550**	=	0.55 x	1.00

Hartland - Span 2 - Live Load Distribution Factors

Skew Reduction Factor: Ref 1 - Table 4.6.2.2.2e-1

For any number of lanes loaded

$$Reduction\ Factor = 1 - c_1(tan\theta)^{1.5}$$

$$c_1 = 0.25\left(\frac{K_g}{12.0Lt_s^3}\right)^{.25}\left(\frac{S}{L}\right)^{0.5}$$

Range of Applicability

30	≤ Skew ≤	60	C1 = 0
3.5	≤ S ≤	16	OK
20	≤ L ≤	240	OK
4	≤ N_b		OK

c_1 interior (G2-4) 0.000 $= 0.25 \times \left(\frac{286782\ in4}{12 \times 85.94\ ft \times (9.00\ in)^3}\right)^{0.25}\left(\frac{6.67\ ft}{85.94\ ft}\right)^{0.5}$

c_1 safety curb (G1) 0.000 $= 0.25 \times \left(\frac{286782\ in4}{12 \times 85.94\ ft \times (9.00\ in)^3}\right)^{0.25}\left(\frac{6.67\ ft}{85.94\ ft}\right)^{0.5}$

c_1 safety curb (G5) 0.000 $= 0.25 \times \left(\frac{286782\ in4}{12 \times 85.94\ ft \times (9.00\ in)^3}\right)^{0.25}\left(\frac{6.67\ ft}{85.94\ ft}\right)^{0.5}$

RF interior (G2-4) 1.000 $= 1 - 0.000 \times (0.547\ rad)^{1.5}$

RF safety curb (G1) 1.000 $= 1 - 0.000 \times (0.547\ rad)^{1.5}$

RF safety curb (G5) 1.000 $= 1 - 0.000 \times (0.547\ rad)^{1.5}$

<u>Shear Distribution Factor Calculations</u>

Interior Beams: Ref 1 - Table 4.6.2.2.3A-1

Range of Applicability

3.5	≤ S ≤	16	OK
4.5	≤ t_S ≤	12	OK
20	≤ L ≤	240	OK
	N_b ≥	4	OK

For one design lane loaded

$$DF = 0.36 + \frac{S}{25}$$

$DF_{S\text{-}INT\text{-}1}$ 0.627 = 0.36+ (6.7 ft/ 25)

For two or more design lanes loaded

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

$DF_{S\text{-}INT\text{-}2}$ 0.719 $= 0.2 + \frac{6.67\ ft}{12} - \left(\frac{6.67\ ft}{35}\right)^{2.0}$

Hartland - Span 2 - Live Load Distribution Factors

Exterior Beams: Ref 1 - Table 4.6.2.2.3b-1, C4.6.2.2.2d

Safety Curb Beam (G1)

Range of Applicability					
-1.0	≤	d_e	≤	5.5	OK
3	<	N_b			OK

For one design lane loaded
Use lever rule (See moment distribution factor calculations)

$DF_{S\text{-}EXT\text{-}1}$ 0.522

For two or more design lanes loaded

$e = 0.60 + d_e/10 =$ 0.71 = 0.6+ 1.13 ft/ 10

$DF_{S\text{-}EXT\text{-}2}$ 0.513 = 0.71 * 0.72

Pile Cap Analogy (See moment distribution factor calculations)

$DF_{S\text{-}EXT\text{-}1}$ 0.581
$DF_{S\text{-}EXT\text{-}2}$ 0.608

Safety Curb Beam (G5)

Range of Applicability					
-1.0	≤	d_e	≤	5.5	OK
3	<	N_b			OK

For one design lane loaded
Use lever rule (See moment distribution factor calculations)

$DF_{S\text{-}EXT\text{-}1}$ 0.435

For two or more design lanes loaded

$e = 0.60 + d_e/10 =$ 0.62 = 0.6+ 0.17 ft/ 10

$DF_{S\text{-}EXT\text{-}2}$ 0.444 = 0.62 * 0.72

Pile Cap Analogy (See moment distribution factor calculations)

$DF_{S\text{-}EXT\text{-}1}$ 0.546
$DF_{S\text{-}EXT\text{-}2}$ 0.550

Hartland - Span 2 - Live Load Distribution Factors

Obtuse Corner Correction Factor: Ref 1 - Table 4.6.2.2.3c-1

For any number of lanes loaded

$$CORRECTION-FACTOR = 1.0 + 0.20\left(\frac{12.0Lt_s^3}{K_g}\right)^{0.3}\tan\theta$$

Range of Applicability					
3.5	≤	S	≤	16	OK
0	≤	θ	≤	60	OK
20	≤	L	≤	240	OK
		N_b	≥	4	OK

$CF_{\text{interior (G2-4)}}$ 1.15 = 1 + 0.20 x $\left(\frac{12 \times 85.94\text{ ft} \times (9.00\text{ in})^3}{286782\text{ in4}}\right)^{0.3}$ x 0.547 rad

$CF_{\text{safety curb (G1)}}$ 1.15 = 1 + 0.20 x $\left(\frac{12 \times 85.94\text{ ft} \times (9.00\text{ in})^3}{286782\text{ in4}}\right)^{0.3}$ x 0.547 rad

$CF_{\text{safety curb (G5)}}$ 1.15 = 1 + 0.20 x $\left(\frac{12 \times 85.94\text{ ft} \times (9.00\text{ in})^3}{286782\text{ in4}}\right)^{0.3}$ x 0.547 rad

Distribution Factor Summary						
	Moment			Shear		
Girder/Load Case	1 Lane	Mult Lanes	Fatigue	1 Lane	Mult Lanes	Fatigue
G1	0.58	0.61	0.48	0.67	0.70	0.55
G2-G4	0.37	0.52	0.31	0.72	0.82	0.60
G5	0.55	0.55	0.46	0.63	0.63	0.52

<u>Deflection Distribution Factor Calculations</u>

All Girders: Ref 1 - C2.5.2.6.2

For a straight multibeam bridge, the distribution factor for defleciton is equal to the number of lanes divided by the number of beams.

DF = MP*(N_L/N_b) = 0.40

Span 1 - Critical Girder Output

FILE=R1.WRN

1. Appendix A6 will be considered unless IGNORE APPENDIX A6 is given in input.

2. LRFR RATINGS condition not given. Program will rate this girder according just to LRFD.

(Above warnings may not be comprehensive.)

FILE=R1.OUT

```
***************************************************************
MDX Steel Highway Girder Design Program, Version 6.5.4113
Load and Resistance Factor Design - Composite Girder

Copyright (c) 1996-2014 MDX Software Inc.
Ph:573-446-3221 Fax:573-446-3278 Email: support@mdxsoftware.com
***************************************************************
 Apr 26, 2019 - 10:10 am
 Files: Span 1 - North Exterior.R1
        R1.OUT

    Contents of data file-

       **************************************************
       ID: SPAN 1 - NORTH EXTERIOR
       CONDITIONS
         ASSUME SLAB ON FLANGE FOR SECTION PROPERTIES
         ENGLISH INPUT
         ENGLISH OUTPUT
         LRFD METHOD
         M270-50 STEEL
         M270-50 STIFFENER STEEL
```

Span 1 - Critical Girder Output

```
   NO INTERMEDIATE TRANSVERSE STIFFENERS
   RATE MODE
   SELF WEIGHT FOR DEAD LOAD 1
   SINGLE BEARING STIFFENERS EACH SIDE
DATA
   ADTT 50
   BNGSKEW 99.24  99.24
   BR 20.99  20.99  20.99  20.97
   ESLABW 57.13  63.13  67.88  71.5  73.75  74.75  74.5  73.  70.25
    66.25  61.
   FILLET 2.
   FPC 4.
   LANED  0.4
   LANEM  0.59
   LANEMF  0.48
   LANEV  0.62
   LANEVF  0.5333
   LIFE 75
   NMOD 8.
   NSTUDL 2
   NSUPBR 1 1
   SLABT 9.
   SLABWEAR  0.75
   SPLBFT 1.1875
   SPLBFW 16.
   SPLTFT  0.8125
   SPLTFW 16.
   SPLWD 34.
   SPLWT  0.4375
   SPN 83.94
```

Span 1 - Critical Girder Output

```
SS 1.
STD  0.875
STH 6.
SUPBST  0.75
SUPBSW 7.75
TSLABW 57.13  63.13  67.88  71.5  73.75  74.75  74.5  73.  70.25
 66.25  61.
WAC  0.08
WAS  0.02
WCONC 150.
WEAR  0.169
WSDL  0.123
GO
**************************************************
```

♀ Case Data

Case Data - SPAN 1 - NORTH EXTERIOR

AASHTO Specification

Load and Resistance Factor Method
6th Edition LRFD Bridge Design Specifications
2nd Edition Manual for Bridge Evaluation

Dimensions (additional information available in Dimensions table)

Given dimensions-

Span 1 - Critical Girder Output

Web Depth	34.00	in		
Web Thickness	0.44	in		
Bearing Stiff. Width	7.75	in	7.75	in
Bearing Stiff. Thickness	0.75	in	0.75	in

Execution Mode

Rate Mode

Geometry

Brace locations

0.00 ft   20.99 ft   41.98 ft   62.97 ft
83.94 ft

Cover plates

No cover plates

Curvature

No curvature

Flange splices

Girder Type

Span 1 - Critical Girder Output

Plate girder

Interior girder

Hinges

No interior hinges

Span lengths

Spans	83.94 ft

Stiffeners

Bearing stiffeners

Single bearing stiffeners each side

Longitudinal stiffeners

No longitudinal stiffener

Transverse stiffeners

No transverse stiffeners

Web haunches

Web splices

Span 1 - Critical Girder Output

Fatigue

Average Single Lane Daily Truck Traffic: 50

Allowable weld stress 18.00 ksi

AWS minimum welds

Fatigue life: 75

Composite Behavior

Composite region for composite loading- 0. - 83.94 ft

Lane fraction for strength limit state

Shear	0.6200	0.6200	0.6200	0.6200	0.6200
	0.6200	0.6200	0.6200	0.6200	0.6200
	0.6200				
Moment	0.5900	0.5900	0.5900	0.5900	0.5900
	0.5900	0.5900	0.5900	0.5900	0.5900
	0.5900				

Lane fraction for fatigue limit state

Shear	0.5333	0.5333	0.5333	0.5333	0.5333
	0.5333	0.5333	0.5333	0.5333	0.5333
	0.5333				
Moment	0.4800	0.4800	0.4800	0.4800	0.4800
	0.4800	0.4800	0.4800	0.4800	0.4800

Span 1 - Critical Girder Output

0.4800

Lane fraction for live-load deflection

0.4000

Loaded lanes 2

Tandem truck multiplier: 1.0000

Design truck multiplier: 1.0000

Influence lines not displayed

Unshored construction

Tenth pt values of distributed dead load carried by steel alone

0.828 k/ft 0.885 k/ft 0.929 k/ft 0.963 k/ft 0.984 k/ft
0.994 k/ft 0.991 k/ft 0.977 k/ft 0.951 k/ft 0.914 k/ft
0.865 k/ft

Steel wt in addition to cross section included in dist dead load

0.020 k/ft 0.020 k/ft 0.020 k/ft 0.020 k/ft 0.020 k/ft
0.020 k/ft 0.020 k/ft 0.020 k/ft 0.020 k/ft 0.020 k/ft
0.020 k/ft

Conc wt in addition to slab thickness and haunch included in dist dead load

0.080 k/ft 0.080 k/ft 0.080 k/ft 0.080 k/ft 0.080 k/ft
0.080 k/ft 0.080 k/ft 0.080 k/ft 0.080 k/ft 0.080 k/ft
0.080 k/ft

Superimposed dead load

0.123 k/ft 0.123 k/ft 0.123 k/ft 0.123 k/ft
0.123 k/ft 0.123 k/ft 0.123 k/ft 0.123 k/ft
0.123 k/ft 0.123 k/ft 0.123 k/ft

Wearing surface dead load

0.169 k/ft 0.169 k/ft 0.169 k/ft 0.169 k/ft
0.169 k/ft 0.169 k/ft 0.169 k/ft 0.169 k/ft
0.169 k/ft 0.169 k/ft 0.169 k/ft

Span 1 - Critical Girder Output

Load Factors

DC1,DC2	1.250
DW	1.500
HL93 LL+I	1.750
Constructibility	1.250

Load Modifiers

Ductility	1.00
Redundancy	1.00
Operational Classification	1.00

Reactions

Max unfactored live load+impact reactions

69.42 k   69.42 k

Min unfactored live load+impact reactions

0.00 k   0.00 k

Max unfactored live reactions - No dynamic load allowance

56.33 k   56.33 k

Min unfactored live reactions - No dynamic load allowance

0.00 k   0.00 k

Total unfactored dead load DC1+DC2 reactions

45.19 k   45.73 k

Total unfactored dead load DW reactions

7.09 k   7.09 k

Support skew for shear and moment modification

90.00   90.00

Bearing skew for redistribution qualification

99.24   99.24

Span 1 - Critical Girder Output

Material

Concrete

Concrete strength	4.00 ksi			
Unit wt of concrete	150. lb/cu ft			
Aggregate source correction factor K1	1.00			
Slab T for strength	8.25 in			
Effective slab width	57.13 in	63.13 in	67.88 in	71.50 in
	73.75 in	74.75 in	74.50 in	73.00 in
	70.25 in	66.25 in	61.00 in	
Fillet	2.00 in			
Effective slab width by tenth pt				
	57.13 in	63.13 in	67.88 in	71.50 in
	73.75 in	74.75 in	74.50 in	73.00 in
	70.25 in	66.25 in	61.00 in	
Self weight slab width by tenth pt				
	57.13 in	63.13 in	67.88 in	71.50 in
	73.75 in	74.75 in	74.50 in	73.00 in
	70.25 in	66.25 in	61.00 in	

Steel

Web splice section 1

Steel grade	M270-50
Rebar yield	60.00 ksi

Span 1 - Critical Girder Output

Output

Standard resolution summary tables

Units

Input units: U.S. cust.

Output units: U.S. cust.

♀ Service Shear

Service Shear - k - (load modifier not included)

Tenth Point	Loc	DC1	DC2	DW	Design Truck+ Lane LL+I (+)	Design Truck+ Lane LL+I (-)	Tandem Truck+ Lane LL+I	Fatigue Truck Range LL+I
0	0.00	39.35	5.16	7.09	69.42	0.00	56.90	32.69
1	8.39	32.16	4.13	5.67	60.76	-2.81	49.61	29.29
2	16.79	24.54	3.10	4.26	52.00	-6.16	42.66	27.17
3	25.18	16.60	2.06	2.84	43.56	-13.37	36.04	25.12
4	33.58	8.43	1.03	1.42	35.46	-20.25	29.75	23.16
5	41.97	0.13	0.00	0.00	27.69	-27.69	23.80	22.39
6	50.36	-8.20	-1.03	-1.42	20.25	-35.46	-29.75	24.02
7	58.76	-16.47	-2.06	-2.84	13.37	-43.56	-36.04	25.98

8	67.15	-24.56	-3.10	-4.26	7.26	-52.00	-42.66	27.95
9	75.55	-32.39	-4.13	-5.67	2.81	-60.76	-49.61	30.24
10	83.94	-39.85	-5.16	-7.09	0.00	-69.42	-56.90	32.69

	Fatigue Trk LL+I	Permit Trk LL+I	Permit Range	Sidewalk
0	32.69	0.00	0.00	0.00
1	28.31	0.00	0.00	0.00
2	24.22	0.00	0.00	0.00
3	19.81	0.00	0.00	0.00
4	15.39	0.00	0.00	0.00
5	11.45	0.00	0.00	0.00
6	-15.03	0.00	0.00	0.00
7	-19.44	0.00	0.00	0.00
8	-23.86	0.00	0.00	0.00
9	-28.27	0.00	0.00	0.00
10	-32.69	0.00	0.00	0.00

♀ Reactions

Girder 1 Factored Reactions - Strength I - k

Location	DC1	DC2	DW	LL+I Max	LL+I Min	Max Total	Min Total

Includes ductility, redundancy, and operational factors.

Span 1 - Critical Girder Output

0.00	50.03	6.45	10.64	121.49	0.00	188.62	45.28
Steel	9.62						
Conc	40.42						
83.94	50.72	6.45	10.64	121.49	0.00	189.30	45.77
Steel	9.62						
Conc	41.10						

See 5th Ed.LRFD, commentary pg 3.11, for min total calcs

Note: DC1 reactions at end supports in girder output account for the additional weight of the extension of the girder past the end bearing locations, based on GDREXT girder input (which defaults to 1.0 ft if not specified).

Unfactored Reactions - k

Location	DC1	DC2	DW	LL+I Max	LL+I Min	Max Total	Min Total

Span 1 - Critical Girder Output

0.00	40.03	5.16	7.09	69.42	0.00	121.71	52.28
Steel	7.69						
Conc	32.33						

83.94	40.57	5.16	7.09	69.42	0.00	122.25	52.83
Steel	7.69						
Conc	32.88						

♀ Service Moment

Service Moment - k-ft - (load modifier not included)

Tenth Point	Loc	DC1	DC2	DW	Design Truck+ Lane Max LL+I	Design Truck+ Lane Min LL+I	Tandem Truck+ Lane Max LL+I	Tandem Truck+ Lane Min LL+I	Fatigue Truck Range LL+I
0	0.0	0.	0.	0.	0.	0.	0.	0.	0.
1	8.4	300.	39.	54.	487.	0.	408.	0.	229.
2	16.8	539.	69.	95.	852.	0.	724.	0.	390.
3	25.2	712.	91.	125.	1108.	0.	947.	0.	498.

Span 1 - Critical Girder Output

4	33.6	817.	104.	143.	1264.	0.	1078.	0.	552.
5	42.0	853.	108.	149.	1298.	0.	1117.	0.	538.
6	50.4	819.	104.	143.	1264.	0.	1078.	0.	552.
7	58.8	715.	91.	125.	1108.	0.	947.	0.	498.
8	67.2	543.	69.	95.	852.	0.	724.	0.	390.
9	75.5	304.	39.	54.	487.	0.	408.	0.	229.
10	83.9	0.	0.	0.	0.	0.	0.	0.	0.

Bracing

21.0	634.	81.	112.	993.	0.	847.	0.
42.0	853.	108.	149.	1298.	0.	1117.	0.
63.0	637.	81.	112.	993.	0.	847.	0.

	Sidewalk Max	Sidewalk Min	Max LL+I Fat Trk	Min LL+I Fat Trk	Max LL+I Prmt Trk	Min LL+I Prmt Trk
0	0.	0.	0.	0.	0.	0.
1	0.	0.	229.	0.	0.	0.
2	0.	0.	390.	0.	0.	0.
3	0.	0.	498.	0.	0.	0.
4	0.	0.	552.	0.	0.	0.
5	0.	0.	538.	0.	0.	0.

Span 1 - Critical Girder Output

6	0.	0.	552.	0.	0.	0.
7	0.	0.	498.	0.	0.	0.
8	0.	0.	390.	0.	0.	0.
9	0.	0.	229.	0.	0.	0.
10	0.	0.	0.	0.	0.	0.

♀ Service Force Summary Tables

Girder 1 Force Summary at Tenth Points (k,k-ft)

Location	DL-S	SDL-S	LLI-S	DL-M	SDL-M	LLI+M	LLI-M	DL-T	SDL-T	LLI-T
0.0	39	12	69	0	0	0	0	0	0	0
8.3	32	10	61	300	93	487	0	0	0	0
16.7	25	7	52	539	165	852	0	0	0	0
25.1	17	5	44	712	216	1108	0	0	0	0
33.5	8	2	35	817	247	1264	0	0	0	0
41.9	0	0	28	853	257	1298	0	0	0	0
50.3	-8	-2	-35	819	247	1264	0	0	0	0
58.7	-16	-5	-44	715	216	1108	0	0	0	0
67.1	-25	-7	-52	543	165	852	0	0	0	0
75.5	-32	-10	-61	304	93	487	0	0	0	0
83.9	-40	-12	-69	0	0	0	0	0	0	0

Span 1 - Critical Girder Output

Girder 1 Force Summary at Quarter Points (k,k-ft)

Location	DL-S	SDL-S	LLI-S	DL-M	SDL-M	LLI+M	LLI-M	DL-T	SDL-T	LLI-T
0.0	39	12	69	0	0	0	0	0	0	0
20.9	21	6	48	633	193	994	0	0	0	0
41.9	0	0	28	853	257	1298	0	0	0	0
62.9	-21	-6	-48	637	193	994	0	0	0	0
83.9	-40	-12	-69	0	0	0	0	0	0	0

Girder 1 Force Summary at Brace Locations (k,k-ft)

Location	DL-S	SDL-S	LLI-S	DL-M	SDL-M	LLI+M	LLI-M	DL-T	SDL-T	LLI-T
20.9	21	6	48	634	193	993	0	0	0	0
41.9	0	0	28	853	257	1298	0	0	0	0
41.9	0	0	28	853	257	1298	0	0	0	0
62.9	-21	-6	-48	637	193	993	0	0	0	0
83.9	0	0	0	0	0	0	0	0	0	0

Key

Span 1 - Critical Girder Output

DL-S	Noncomp dead shear
SDL-S	Superimposed dead shear
LLI-S	Live+impact shear
DL-M	Noncomp dead moment
SDL-M	Superimposed dead moment
LLI+M	Max live+impact moment
LLI-M	Min live+impact moment
DL-T	Noncomp dead torque
SDL-M	Superimposed dead torque
LLI-T	Max live+impact torque

Shear on right side of pier is found in Service Shear table.

♀ Combined Factored Moment - Strength I Loading

Combined Factored Moment - Strength I - HL93 - k-ft

Load modifiers included

Tenth Pt	Loc	DC1+DC2+DW+Max LL+I	DC1+DC2+DW+Min LL+I
0	0.00	0.00	0.00
1	8.39	1356.57	504.65
2	16.79	2394.16	902.86
3	25.18	3130.20	1190.69
4	33.58	3577.38	1365.19
5	41.97	3696.80	1424.47

Span 1 - Critical Girder Output

6	50.36	3579.92	1367.73
7	58.76	3134.66	1195.15
8	67.15	2399.29	907.98
9	75.55	1360.43	508.50
10	83.94	0.00	0.00

♀ Dimensions

Dimensions

Tenth Point	Loc	-Top Flange- tfw	tft	Weld	----Web---- wd	wt	Weld	-Bot Flange- bfw	bft	Area
0	0.00	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88
1	8.39	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88
2	16.79	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88
3	25.18	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88
4	33.58	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88
5	41.97	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88
6	50.36	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88
7	58.76	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88
8	67.15	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88
9	75.55	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88
10	83.94	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88

Bearing Stiffeners

Span 1 - Critical Girder Output

Location	Width	Thickness
0.00	7.75	0.750
83.94	7.75	0.750

♀ Elastic Section Properties for Stiffness Analysis

Elastic Section Properties for Stiffness Analysis

	----- Noncomp -----			------ Comp ------ (n = 8.00)			------ Comp ------ (3n =24.00)		
Tenth Point	I	na from Bott	na from Top Flg	I	na from Bott	na from Top Flg	I	na from Bott	na from Top Flg
0	11007.	15.88	20.12	26682.	29.38	6.62	19252.	23.04	12.96
1	11007.	15.88	20.12	27391.	29.98	6.02	19847.	23.55	12.45
2	11007.	15.88	20.12	27902.	30.40	5.60	20294.	23.94	12.06
3	11007.	15.88	20.12	28266.	30.70	5.30	20622.	24.22	11.78
4	11007.	15.88	20.12	28481.	30.88	5.12	20820.	24.39	11.61
5	11007.	15.88	20.12	28574.	30.96	5.04	20906.	24.47	11.53
6	11007.	15.88	20.12	28551.	30.94	5.06	20885.	24.45	11.55
7	11007.	15.88	20.12	28410.	30.82	5.18	20754.	24.34	11.66
8	11007.	15.88	20.12	28143.	30.60	5.40	20510.	24.13	11.87
9	11007.	15.88	20.12	27732.	30.26	5.74	20143.	23.81	12.19
10	11007.	15.88	20.12	27148.	29.77	6.23	19640.	23.38	12.62

Span 1 - Critical Girder Output

♀ Elastic Section Properties for Pos Mom Stress

Elastic Section Properties for Pos Mom Stress

	----- Noncomp -----			------ Comp ------ (n = 8.00)			------ Comp ------ (3n =24.00)		
Tenth Point	I	na from Bott	na from Top Flg	I	na from Bott	na from Top Flg	I	na from Bott	na from Top Flg
0	11007.	15.88	20.12	26682.	29.38	6.62	19252.	23.04	12.96
1	11007.	15.88	20.12	27391.	29.98	6.02	19847.	23.55	12.45
2	11007.	15.88	20.12	27902.	30.40	5.60	20294.	23.94	12.06
3	11007.	15.88	20.12	28266.	30.70	5.30	20622.	24.22	11.78
4	11007.	15.88	20.12	28481.	30.88	5.12	20820.	24.39	11.61
5	11007.	15.88	20.12	28574.	30.96	5.04	20906.	24.47	11.53
6	11007.	15.88	20.12	28551.	30.94	5.06	20885.	24.45	11.55
7	11007.	15.88	20.12	28410.	30.82	5.18	20754.	24.34	11.66
8	11007.	15.88	20.12	28143.	30.60	5.40	20510.	24.13	11.87
9	11007.	15.88	20.12	27732.	30.26	5.74	20143.	23.81	12.19
10	11007.	15.88	20.12	27148.	29.77	6.23	19640.	23.38	12.62

♀ Plastic Section Properties

Plastic Section Properties

Span 1 - Critical Girder Output

Tenth Point	Ps+Pr	Ptf+Pw +Pbf	PNA Frm tab D6.1-1 +Mp	-Mp	Pos Mp (k-ft)	Case	Neg Mp (k-ft)	Case
0	1602.	2344.	8.71	26.02	4466.	II	3428.	I
1	1771.	2344.	8.61	26.02	4529.	II	3428.	I
2	1904.	2344.	8.52	26.02	4578.	II	3428.	I
3	2006.	2344.	8.46	26.02	4615.	II	3428.	I
4	2069.	2344.	8.42	26.02	4638.	II	3428.	I
5	2097.	2344.	8.40	26.02	4648.	II	3428.	I
6	2090.	2344.	8.41	26.02	4646.	II	3428.	I
7	2048.	2344.	8.44	26.02	4631.	II	3428.	I
8	1971.	2344.	8.48	26.02	4603.	II	3428.	I
9	1858.	2344.	8.55	26.02	4562.	II	3428.	I
10	1711.	2344.	8.65	26.02	4507.	II	3428.	I

Note: PNA is shown as measured from top of slab

♀ Top Flange Factored Bending Stress - Strength I Loading

Factored Bending Stresses - Strength I - ksi

Tenth Point	LOC	DC1	DC2+DW	Max LL+I	Min LL+I	LL+I Rng	Tot	Allow	Ratio

(Columns DC1 through Ratio are under the heading: ------------------ Top Flange -------------------)

Span 1 - Critical Girder Output

0	0.00	0.0	0.0	0.0	0.0	0.0	0.0	50.0	0.000
1	8.39	-8.2	-1.0	-2.2	0.0	2.2	-11.5	50.0	0.229
2	16.79	-14.8	-1.6	-3.6	0.0	3.6	-20.0	50.0	0.400
3	25.18	-19.5	-2.1	-4.4	0.0	4.4	-25.9	50.0	0.519
4	33.58	-22.4	-2.3	-4.8	0.0	4.8	-29.5	50.0	0.589
5	41.97	-23.4	-2.4	-4.8	0.0	4.8	-30.6	50.0	0.611
6	50.36	-22.4	-2.3	-4.7	0.0	4.7	-29.4	50.0	0.589
7	58.76	-19.6	-2.0	-4.2	0.0	4.2	-25.9	50.0	0.518
8	67.15	-14.9	-1.6	-3.4	0.0	3.4	-19.9	50.0	0.398
9	75.55	-8.3	-0.9	-2.1	0.0	2.1	-11.4	50.0	0.227
10	83.94	0.0	0.0	0.0	0.0	0.0	0.0	50.0	0.000

Governing expression for allowable compression stress

Tenth Pt	Expression			
1	(6.10.8.2.3-2)	Cb 1.5848	Rb 1.0000	Rh 1.0000
2	(6.10.8.2.3-2)	Cb 1.5848	Rb 1.0000	Rh 1.0000
3	(6.10.8.2.3-2)	Cb 1.0570	Rb 1.0000	Rh 1.0000
4	(6.10.8.2.3-2)	Cb 1.0570	Rb 1.0000	Rh 1.0000
5	(6.10.8.2.3-2)	Cb 1.0570	Rb 1.0000	Rh 1.0000
6	(6.10.8.2.3-2)	Cb 1.0521	Rb 1.0000	Rh 1.0000
7	(6.10.8.2.3-2)	Cb 1.0521	Rb 1.0000	Rh 1.0000
8	(6.10.8.2.3-2)	Cb 1.5510	Rb 1.0000	Rh 1.0000
9	(6.10.8.2.3-2)	Cb 1.5535	Rb 1.0000	Rh 1.0000

Span 1 - Critical Girder Output

Perf. ratio for compact section is (factored mom/mom strength)

if this is less than (total stress/allowable stress)

♀ Shear Stress

Shear Stress - ksi

Tenth	Loc	Total	Allow	Ratio
0	0.00	12.6	21.9	0.577
1	8.39	10.8	21.9	0.493
2	16.79	8.9	21.9	0.406
3	25.18	7.0	21.9	0.319
4	33.58	5.1	21.9	0.234
5	41.97	3.3	21.9	0.149
6	50.36	5.1	21.9	0.233
7	58.76	7.0	21.9	0.319
8	67.15	8.9	21.9	0.406
9	75.55	10.8	21.9	0.494
10	83.94	12.7	21.9	0.579

♀ Bottom Flange Factored Bending Stress - Strength I Loading

Factored Bending Stresses - Strength I - ksi

Tenth Loc ----------------- Bottom Flange -----------------

Span 1 - Critical Girder Output

Point		DC1	DC2+DW	Max LL+I	Min LL+I	LL+I Rng	Wind	Tot	Allow	Ratio
0	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	50.0	0.000
1	8.39	6.5	1.8	11.2	0.0	11.2	0.0	19.5	50.0	0.319
2	16.79	11.7	3.2	19.5	0.0	19.5	0.0	34.4	50.0	0.556
3	25.18	15.4	4.2	25.3	0.0	25.3	0.0	44.9	50.0	0.720
4	33.58		Factored stresses in inelastic range. **							0.818
5	41.97		Factored stresses in inelastic range. **							0.844
6	50.36		Factored stresses in inelastic range. **							0.817
7	58.76	15.5	4.2	25.3	0.0	25.3	0.0	45.0	50.0	0.718
8	67.15	11.7	3.2	19.5	0.0	19.5	0.0	34.4	50.0	0.554
9	75.55	6.6	1.8	11.2	0.0	11.2	0.0	19.6	50.0	0.317
10	83.94	0.0	0.0	0.0	0.0	0.0	0.0	0.0	50.0	0.000

** Permitted for compact section (or Appendix A6), if the factored moment strength is not exceeded by the factored moment.

Governing expression for allowable compression stress

Tenth Pt   Expression

Span 1 - Critical Girder Output

Perf. ratio for compact section is (factored mom/mom strength)

if this is less than (total stress/allowable stress)

♀ Factored Slab and Rebar Stresses

Factored Bending Stresses - ksi

Tenth	Loc	----- Slab -----			---- Rebars ----		
Point		Comp	Max	Tot	Comp	Min	Tot
		Dead	LL+I		Dead	LL+I	
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	8.39	-0.12	-0.76	-0.87	0.00	0.00	0.00
2	16.79	-0.20	-1.27	-1.47	0.00	0.00	0.00
3	25.18	-0.25	-1.60	-1.85	0.00	0.00	0.00
4	33.58	-0.28	-1.79	-2.07	0.00	0.00	0.00
5	41.97	-0.29	-1.82	-2.11	0.00	0.00	0.00
6	50.36	-0.28	-1.78	-2.06	0.00	0.00	0.00
7	58.76	-0.25	-1.58	-1.83	0.00	0.00	0.00
8	67.15	-0.19	-1.24	-1.44	0.00	0.00	0.00
9	75.55	-0.11	-0.74	-0.85	0.00	0.00	0.00
10	83.94	0.00	0.00	0.00	0.00	0.00	0.00

Only slab compression stresses tabulated.

♀ Constructibility of Web in Bending

Compression Bending Stress for Constructibility - ksi

Span 1 - Critical Girder Output

Tenth Point	Noncomp. Dead Factored Mom	Noncomp. Dead Web Bending Stress	Rb	Rh	Web Allow 6.10.1.9	Ratio
0	0.00	0.00	1.0000	1.0000	50.00	0.000
1	375.52	7.90	1.0000	1.0000	48.50*	0.170
2	673.30	14.17	1.0000	1.0000	48.50*	0.304
3	889.40	18.72	1.0000	1.0000	44.23*	0.441
4	1020.85	21.48	1.0000	1.0000	44.23*	0.506
5	1065.79	22.43	1.0000	1.0000	44.23*	0.528
6	1023.39	21.54	1.0000	1.0000	44.03*	0.510
7	893.86	18.81	1.0000	1.0000	44.03*	0.445
8	678.43	14.28	1.0000	1.0000	48.50*	0.307
9	379.38	7.98	1.0000	1.0000	48.50*	0.172
10	0.00	0.00	1.0000	1.0000	50.00	0.000

* Compression flange allowable stress, based on lateral-torsional or local buckling limits, with the ratio representing (flange stress)/(flange allowable stress).

♀ Top Flange Permanent Deflection Control Stress

Top Flange 6.10.4.2 Permanent Deflection Control - ksi

Tenth	Location	DC1	DC2+ DW	1.3LL+I Max	1.3LL+I Min	Total	Allowable	Ratio

Span 1 - Critical Girder Output

0	0.00	0.00	0.00	0.00	0.00	0.00	47.50	0.000
1	8.39	-6.59	-0.70	-1.67	0.00	-8.96	47.50	0.189
2	16.79	-11.81	-1.17	-2.67	0.00	-15.65	47.50	0.330
3	25.18	-15.60	-1.48	-3.24	0.00	-20.32	47.50	0.428
4	33.58	-17.91	-1.65	-3.54	0.00	-23.11	47.50	0.486
5	41.97	-18.70	-1.70	-3.57	0.00	-23.98	47.50	0.505
6	50.36	-17.96	-1.64	-3.50	0.00	-23.09	47.50	0.486
7	58.76	-15.68	-1.46	-3.15	0.00	-20.29	47.50	0.427
8	67.15	-11.90	-1.14	-2.55	0.00	-15.60	47.50	0.328
9	75.55	-6.66	-0.67	-1.57	0.00	-8.90	47.50	0.187
10	83.94	0.00	0.00	0.00	0.00	0.00	47.50	0.000

♀ Bottom Flange Permanent Deflection Control Stress

Bottom Flange 6.10.4.2 Permanent Deflection Control - ksi

Tenth	Location	DC1	DC2+ DW	1.3LL+I Max	1.3LL+I Min	Total	Allowable	Ratio
0	0.00	0.00	0.00	0.00	0.00	0.00	47.50	0.000
1	8.39	5.20	1.32	8.31	0.00	14.83	47.50	0.312
2	16.79	9.33	2.33	14.49	0.00	26.14	47.50	0.550
3	25.18	12.32	3.04	18.78	0.00	34.15	47.50	0.719
4	33.58	14.14	3.47	21.38	0.00	39.00	47.50	0.821
5	41.97	14.76	3.61	21.95	0.00	40.32	47.50	0.849
6	50.36	14.18	3.47	21.37	0.00	39.02	47.50	0.821
7	58.76	12.38	3.04	18.76	0.00	34.18	47.50	0.720

Span 1 - Critical Girder Output

8	67.15	9.40	2.32	14.46	0.00	26.18	47.50	0.551
9	75.55	5.26	1.31	8.29	0.00	14.86	47.50	0.313
10	83.94	0.00	0.00	0.00	0.00	0.00	47.50	0.000

♀ Factored Noncomp Dead Wet Concrete Stress

Factored Noncomp Dead Wet Concrete Stresses - ksi

[While the constructibility load factor is used for this table, slab pouring is not. Actual stresses are dependent on the pouring sequence and may be significantly different from those printed here.]

Tenth Point	Loc	Top Flange	Allow-able	Bottom Flange	Allow-able
0	0.0	0.0	50.0	0.0	50.0
1	8.4	-8.2	48.5	6.5	50.0
2	16.8	-14.8	48.5	11.7	50.0
3	25.2	-19.5	44.2	15.4	50.0
4	33.6	-22.4	44.2	17.7	50.0
5	42.0	-23.4	44.2	18.5	50.0
6	50.4	-22.4	44.0	17.7	50.0
7	58.8	-19.6	44.0	15.5	50.0
8	67.2	-14.9	48.5	11.7	50.0
9	75.5	-8.3	48.5	6.6	50.0

10	83.9	0.0	50.0	0.0	50.0

Construction load factor 1.25 being used for this table.

♀ Factored Strengths

Factored Strengths

Forces include ductility, redundancy, and operational factors

Tenth Point	Loc	Factored Shear (k)	Shear Strength (k)	Ratio	Factored Moment (k-ft)	Bending Strength (k-ft)	Ratio
		HL93			HL93		
0	0.00	187.8	325.2 (5)	0.577	0.0	4189.2 (20)	0.000
1	8.39	160.2	325.2 (5)	0.493	1356.6 I	4256.1 (20)	0.319
2	16.79	131.9	325.2 (5)	0.406	2394.2 I	4308.2 (20)	0.556
3	25.18	103.8	325.2 (5)	0.319	3130.2 I	4347.5 (20)	0.720
4	33.58	76.0	325.2 (5)	0.234	3577.4 I	4371.6 (20)	0.818
5	41.97	48.6	325.2 (5)	0.149	3696.8 I	4382.3 (20)	0.844
6	50.36	75.7	325.2 (5)	0.233	3579.9 I	4379.6 (20)	0.817
7	58.76	103.7	325.2 (5)	0.319	3134.7 I	4363.6 (20)	0.718

Span 1 - Critical Girder Output

8	67.15	132.0	325.2	(5)	0.406	2399.3 I	4334.0	(20)	0.554
9	75.55	160.5	325.2	(5)	0.494	1360.4 I	4290.4	(20)	0.317
10	83.94	188.4	325.2	(5)	0.579	0.0	4232.5	(20)	0.000

Absolute values of factored moment are shown with determining strength

(5) C 0.58 Fyw D t

(20) Compact section

♀ Lrfd Ratings

[This table uses the rating equation (6B.4.1-1) of the 2011 edition of the Manual for Bridge Evaluation where A1D is the sum of factored composite dead, noncomposite dead, and factored wearing surface loads. The denominator A2L(1+I) is the factored live load. If equation (6a.4.2.1-1) is to be used for LRFR ratings the condition LRFR RATINGS must be used in the girder input file.]

Span 1 - Critical Girder Output
HL93

Strength I

Span 1

Location	Compct Mom Cap/Noncmpt Allow Stress	Shear Capacity	Rating Factors Bending Inv.		Bending Oper.	Shear Inv.	Shear Oper.
0.00	4189. C	325.20	>999.00	B	>999.00	2.13	3.73
8.39	4256. C	325.20	4.40	B	7.71	2.55	4.47
16.79	4308. C	325.20	2.28	B	4.00	3.12	5.47
25.18	4347. C	325.20	1.63	B	2.85	3.90	6.83
33.58	4372. C	325.20	1.36	B	2.38	5.02	8.78
41.97	4382. C	325.20	1.30	B	2.28	6.71	11.74
50.36	4380. C	325.20	1.36	B	2.38	5.02	8.79
58.76	4364. C	325.20	1.63	B	2.86	3.91	6.84
67.15	4334. C	325.20	2.30	B	4.02	3.12	5.47
75.55	4290. C	325.20	4.44	B	7.77	2.55	4.46
83.94	4233. C	325.20	>999.00	B	>999.00	2.13	3.72

Span 1 - Critical Girder Output

Service II

Span 1

Location	Allowable Stress		Rating Factors Bending Inv.		Oper.
0.00	47500.	S	> 999.00	B	>999.00
8.39	47500.	S	4.93	B	6.41
16.79	47500.	S	2.47	B	3.22
25.18	47500.	S	1.71	B	2.22
33.58	47500.	S	1.40	B	1.82
41.97	47500.	S	1.33	B	1.73
50.36	47500.	S	1.40	B	1.82
58.76	47500.	S	1.71	B	2.22
67.15	47500.	S	2.48	B	3.22
75.55	47500.	S	4.94	B	6.42
83.94	47500.	S	> 999.00	B	>999.00

****

Minimum rating is 1.30 at location 41.97 in span 1.

Span 1 - Critical Girder Output
****

Rating Codes:

T - Top steel governs

B - Bottom steel governs

C - Concrete governs

R - Rebar governs

V - Shear governs

S - Serviceability governs

Mom Strength Codes:

C - Compact

B - Braced non-compact

U - Unbraced non-compact

T - Transition between compact and braced non-compact

S - Serviceability

Noncompact shapes ratings based on stress, as

```
          Fb   -  factored dead load stress
     IR = --------------------------------
                factored LL+I stress
```

♀ Bearing Stiffeners

Bearing Stiffeners

Location   Factored   Allowable   Ratio   Allowable   Ratio

Span 1 - Critical Girder Output

from Left End of Web Sect. (ft)	Reaction (k)	Column Force (k)		Bearing Force (k)	
0.00	188.62	676.26	0.279	708.75	0.266
83.94	189.25	676.26	0.280	708.75	0.267

♀ Fatigue

Fatigue Summary

Condition	Location from Left End of Girder	Category (Table 6.6.1.2.3-1)	Force Range (k-ft)	Actual Stress Range	Allowable Stress Range	Ratio
Base metal	8.39	B	171.43	2.25	20.62	0.109
Base metal	16.79	B	292.82	3.83	20.62	0.186
Base metal	25.18	B	373.75	4.87	20.62	0.236
Base metal	33.58	B	413.70	5.38	20.62	0.261
Base metal	41.97	B	403.62	5.25	20.62	0.254
Base metal	50.36	B	413.70	5.38	20.62	0.261
Base metal	58.76	B	373.75	4.87	20.62	0.236
Base metal	67.15	B	292.82	3.82	20.62	0.185
Base metal	75.55	B	171.43	2.24	20.62	0.109
Near flg-web weld	8.39	B	171.43	2.16	20.62	0.105
Near flg-web weld	16.79	B	292.82	3.68	20.62	0.178

Span 1 - Critical Girder Output

Near flg-web weld	25.18	B	373.75	4.68	20.62	0.227
Near flg-web weld	33.58	B	413.70	5.18	20.62	0.251
Near flg-web weld	41.97	B	403.62	5.05	20.62	0.245
Near flg-web weld	50.36	B	413.70	5.17	20.62	0.251
Near flg-web weld	58.76	B	373.75	4.68	20.62	0.227
Near flg-web weld	67.15	B	292.82	3.67	20.62	0.178
Near flg-web weld	75.55	B	171.43	2.16	20.62	0.105
Full depth conn PL	20.99	C'	333.33	4.18	14.76	0.283
Full depth conn PL	41.98	C'	403.63	5.05	14.76	0.342
Full depth conn PL	62.97	C'	333.14	4.18	14.76	0.283

♀ Max Performance Ratios

Maximum Performance Ratios

Criterion	Location ft	Max Performance Ratio
Design and tandem trucks		
Shear	83.94	0.579
Bending	41.97	0.849
Fatigue truck		
Fatigue	41.98	0.342

Span 1 - Critical Girder Output

Bearing Stf. 83.94 0.280

♀ Shear Connectors

Girder 1 Shear Connectors

Maximum Pitch for 0.875 X 6.00 Stud Connectors for Fatigue

Tenth Point	Location	Shear Range (k)	Q	I	Studs in a Line	Max Pitch (in)
0	0.00	24.52	632.8	26682.	2	21.69
1	8.39	21.97	660.6	27391.	2	23.80
2	16.79	20.38	680.6	27902.	2	25.38 *
3	25.18	18.84	694.7	28266.	2	27.24 *
4	33.58	17.37	703.0	28481.	2	29.42 *
5	41.97	16.79	706.6	28574.	2	30.38 *
6	50.36	18.02	705.7	28551.	2	28.32 *
7	58.76	19.49	700.3	28410.	2	26.26 *
8	67.15	20.96	689.9	28143.	2	24.55 *
9	75.55	22.68	673.9	27732.	2	22.89
10	83.94	24.52	651.1	27148.	2	21.45

Span 1 - Critical Girder Output

HL93 fatigue truck shear range used for the above.

* Use 24 inches as per 6.10.10.1.2

Total Number Required for Strength

Span 1

58 between tenth points 0 and 5

58 between tenth points 5 and 10

* * * * * * * * * * * * * * * R A T E M O D E * * * * * * * * * * * * * * *

♀ Weight

Girder 1 Weight

Volume Weight

Span 1 - Critical Girder Output

	cu ft	k	
Top Flange	7.58	3.71	
Web	8.67	4.25	
Bottom Flange	11.08	5.43	
Flange-Web Weld		0.06	
Bearing Stiffeners	0.46	0.22	
Steel in extensions		0.32	
Additional Steel		1.68	
Total Steel			15.67
Slab	363.11	54.47	
Flange Haunch	18.65	2.80	
Concrete in extensions		1.08	
Additional Concrete		6.72	
Total Concrete			65.06
Total Steel and Concrete			80.73

Span 1 - Critical Girder Output

Noncomposite weight data used in girder input is used for analysis loading.

♀ Deflections

Service Deflections - in

	Noncomp Dead Total	Noncomp Dead Steel Only	Noncomp Dead Slab Only	Comp Dead Excldg Wearng	Comp Dead Wearng Surfce Only	Live+I Max Up	Live+I Max Down
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	1.050	0.197	0.852	0.072	0.098	0.000	0.285
2	1.988	0.373	1.615	0.135	0.186	0.000	0.544
3	2.725	0.511	2.214	0.185	0.254	0.000	0.743
4	3.194	0.599	2.596	0.216	0.297	0.000	0.877
5	3.356	0.629	2.727	0.227	0.311	0.000	0.915
6	3.197	0.599	2.598	0.216	0.297	0.000	0.876
7	2.729	0.511	2.217	0.184	0.253	0.000	0.742
8	1.992	0.373	1.619	0.135	0.185	0.000	0.542
9	1.052	0.197	0.855	0.071	0.098	0.000	0.285
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Includes sidewalk deflections.

Wearing surface deflections are for future wearing surface.

Span 1 - Critical Girder Output

Loading for defl uses larger of des trk alone

or 0.25 des truck + full lane

Loc								
Brace	20.99	2.391	0.449	1.942	0.162	0.223	0.000	0.652
Brace	41.98	3.356	0.629	2.727	0.227	0.311	0.000	0.915
Brace	62.97	2.384	0.447	1.938	0.161	0.222	0.000	0.649

Positive dead load deflection is downward.

Live load deflection as indicated in column heading.

♀ Support Rotations

Service Support Rotations

Clockwise Positive

Location	Total Dead Load		Live Load Range			
	Deg	Rad	Deg		Rad	
			Low	High	Low	High

0.00	0.664	0.01159	0.000	0.276	0.00000	0.00481
83.94	-0.665	-0.01161	-0.261	0.000	-0.00456	0.00000

Factored Support Rotations - degrees

	Clockwise	Counterclockwise
0.00	1.312	0.000
83.94	0.000	-1.289

♀ Concurrent Max Live Reactions and Slopes

Concurrent Max Live Service Reactions and Slopes

Location ft	Max Reaction k	Concurrent Slope Deg	Max Slope Deg	Concurrent Reaction k
0.00	69.42	0.013	0.276	46.17
83.94	69.42	-0.013	-0.261	38.02

Span 1 - Critical Girder Output

♀ Status

run completed

Span 1 - Reactions

****

MDX Steel Highway Girder Design Program, Version 6.5.4074

Load and Resistance Factor Design - Composite Girder

Ph:573-446-3221 Fax:573-446-3278 Email: support@mdxsoftware.com

****

Apr 2, 2019 - 1:30 pm

Files: Span 1 - South Exterior.R1

R1.OUT

♀ Reactions

Girder 1 Factored Reactions - Strength I - k

Includes ductility, redundancy, and operational factors.

Location	DC1	DC2	DW	LL+I Max	LL+I Min	Max Total	Min Total
0.00	45.99	6.45	10.64	113.65	0.00	176.74	42.37
Steel	10.37						
Conc	35.62						
83.94	46.65	6.45	10.64	113.65	0.00	177.40	42.85
Steel	10.37						
Conc	36.28						

See 5th Ed.LRFD, commentary pg 3.11, for min total calcs

Note: DC1 reactions at end supports in girder output account

for the additional weight of the extension of the girder past

Span 1 - Reactions
the end bearing locations, based on GDREXT girder input

(which defaults to 1.0 ft if not specified).

Unfactored Reactions - k

Location	DC1	DC2	DW	LL+I Max	LL+I Min	Max Total	Min Total
0.00	36.80	5.16	7.09	64.94	0.00	114.00	49.05
Steel	8.30						
Conc	28.50						
83.94	37.32	5.16	7.09	64.94	0.00	114.52	49.58
Steel	8.30						
Conc	29.03						

****

MDX Steel Highway Girder Design Program, Version 6.5.4074

Load and Resistance Factor Design - Composite Girder

****

Apr 2, 2019 - 1:32 pm

Files: SPAN 1 - TYPICAL INTERIOR.R1

R1.OUT

♀ Reactions

Girder 1 Factored Reactions - Strength I - k

Location	DC1	DC2	DW	LL+I Max	LL+I Min	Max Total	Min Total

Span 1 - Reactions
Includes ductility, redundancy, and operational factors.

0.00	53.43	6.45	10.64	146.96	0.00	217.49	47.73
Steel	11.42						
Conc	42.01						
83.94	53.43	6.45	10.64	146.96	0.00	217.49	47.73
Steel	11.42						
Conc	42.01						

See 5th Ed.LRFD, commentary pg 3.11, for min total calcs

Note: DC1 reactions at end supports in girder output account for the additional weight of the extension of the girder past the end bearing locations, based on GDREXT girder input (which defaults to 1.0 ft if not specified).

Unfactored Reactions - k

Location	DC1	DC2	DW	LL+I Max	LL+I Min	Max Total	Min Total
0.00	42.75	5.16	7.09	83.98	0.00	138.98	55.00
Steel	9.14						

Span 1 - Reactions

Conc 33.61

83.94	42.75	5.16	7.09	83.98	0.00	138.98	55.00
Steel	9.14						
Conc	33.61						

****

MDX Steel Highway Girder Design Program, Version 6.5.4074

Load and Resistance Factor Design - Composite Girder

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****

Apr 3, 2019 - 12:11 pm

Files: Span 1 - North Exterior.R1

R1.OUT

♀ Reactions

Girder 1 Factored Reactions - Strength I - k

Location	DC1	DC2	DW	LL+I Max	LL+I Min	Max Total	Min Total
Includes ductility, redundancy, and operational factors.							
0.00	50.79	6.45	10.64	121.49	0.00	189.37	45.82
Steel	10.37						
Conc	40.42						
83.94	51.47	6.45	10.64	121.49	0.00	190.05	46.31
Steel	10.37						
Conc	41.10						

Span 1 - Reactions

See 5th Ed.LRFD, commentary pg 3.11, for min total calcs

Note: DC1 reactions at end supports in girder output account for the additional weight of the extension of the girder past the end bearing locations, based on GDREXT girder input (which defaults to 1.0 ft if not specified).

Unfactored Reactions - k

Location	DC1	DC2	DW	LL+I	LL+I	Max	Min
0.00	40.63	5.16	7.09	69.42	0.00	122.31	52.88
Steel	8.30						
Conc	32.33						
83.94	41.17	5.16	7.09	69.42	0.00	122.85	53.43
Steel	8.30						
Conc	32.88						

OUTPUT

```
FILE=R1.WRN
1. Appendix A6 will be considered unless IGNORE APPENDIX A6 is given in input.


2. LRFR RATINGS condition not given. Program will rate this girder according
   just to LRFD.


(Above warnings may not be comprehensive.)
FILE=R1.OUT



***************************************************************
MDX Steel Highway Girder Design Program, Version 6.5.4074
Load and Resistance Factor Design - Composite Girder


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Ph:573-446-3221 Fax:573-446-3278 Email: support@mdxsoftware.com
***************************************************************
 Jun 17, 2019 -  3:31 pm
 Files: Span 2 - North Exterior.R1
        R1.OUT


    Contents of data file-


       **************************************************
       ID: SPAN 2 - NORTH EXTERIOR
       CONDITIONS
         ASSUME SLAB ON FLANGE FOR SECTION PROPERTIES
         ENGLISH INPUT
         ENGLISH OUTPUT
         LRFD METHOD
         M270-50 STEEL
         M270-50 STIFFENER STEEL
```

OUTPUT

```
  NO INTERMEDIATE TRANSVERSE STIFFENERS
  RATE MODE
  SELF WEIGHT FOR DEAD LOAD 1
  SINGLE BEARING STIFFENERS EACH SIDE
DATA
  ADTT 50
  BNGSKEW 99.24  99.24
  BR 21.49  21.49  21.49 21.47
  ESLABW 54.75  61.88  67.63  72.  75.25  77.  77.5  76.63  74.5  71.13
   66.25
  ETAD 1.
  ETAI 1.
  ETAR 1.
  FILLET 2.
  FPC 4.
  LANED  0.4
  LANEM  0.61
  LANEMF  0.48
  LANEV  0.7
  LANEVF  0.59
  LIFE 75
  NMOD 8.
  NSTUDL 2
  NSUPBR 1 1
  SLABT 9.
  SLABWEAR  0.75
  SPLBFT 1.1875
  SPLBFW 16.
  SPLTFT  0.8125
  SPLTFW 16.
```

OUTPUT

```
    SPLWD 34.
    SPLWT  0.4375
    SPN 85.94
    SS 1.
    STD  0.875
    STH 6.
    SUPBST  0.75
    SUPBSW 7.75
    TSLABW 54.75  61.88  67.63  72.  75.25  77.  77.5  76.63  74.5  71.13
     66.25
    WAC  0.089
    WAS  0.02
    WCONC 150.
    WEAR  0.169
    WSDL  0.123
  GO
  **************************************************
```

♀ Case Data

Case Data - SPAN 2 - NORTH EXTERIOR

AASHTO Specification

Load and Resistance Factor Method

6th Edition LRFD Bridge Design Specifications

2nd Edition Manual for Bridge Evaluation

OUTPUT

Dimensions (additional information available in Dimensions table)

Given dimensions-

Web Depth	34.00	in		
Web Thickness	0.44	in		
Bearing Stiff. Width	7.75	in	7.75	in
Bearing Stiff. Thickness	0.75	in	0.75	in

Execution Mode

Rate Mode

Geometry

Brace locations

0.00 ft 21.49 ft 42.98 ft 64.47 ft
85.94 ft

Cover plates

No cover plates

Curvature

No curvature

Flange splices

OUTPUT

Girder Type

Plate girder

Interior girder

Hinges

No interior hinges

Span lengths

Spans	85.94 ft

Stiffeners

Bearing stiffeners

Single bearing stiffeners each side

Longitudinal stiffeners

No longitudinal stiffener

Transverse stiffeners

No transverse stiffeners

Web haunches

OUTPUT

Web splices

Fatigue

Average Single Lane Daily Truck Traffic: 50

Allowable weld stress   18.00 ksi

AWS minimum welds

Fatigue life: 75

Composite Behavior

Composite region for composite loading-   0. -   85.94 ft

Lane fraction for strength limit state

Shear	0.7000	0.7000	0.7000	0.7000	0.7000
	0.7000	0.7000	0.7000	0.7000	0.7000
	0.7000				
Moment	0.6100	0.6100	0.6100	0.6100	0.6100
	0.6100	0.6100	0.6100	0.6100	0.6100
	0.6100				

Lane fraction for fatigue limit state

Shear	0.5900	0.5900	0.5900	0.5900	0.5900
	0.5900	0.5900	0.5900	0.5900	0.5900
	0.5900				

OUTPUT

```
  Moment      0.4800   0.4800   0.4800   0.4800   0.4800
              0.4800   0.4800   0.4800   0.4800   0.4800
              0.4800


Lane fraction for live-load deflection
              0.4000


Loaded lanes         2
Tandem truck multiplier: 1.0000
Design truck multiplier: 1.0000
Influence lines not displayed
Unshored construction
Tenth pt values of distributed dead load carried by steel alone
               0.815 k/ft   0.882 k/ft   0.936 k/ft   0.977 k/ft   1.007 k/ft
               1.024 k/ft   1.028 k/ft   1.020 k/ft   1.000 k/ft   0.969 k/ft
               0.923 k/ft
Steel wt in addition to cross section included in dist dead load
               0.020 k/ft   0.020 k/ft   0.020 k/ft   0.020 k/ft   0.020 k/ft
               0.020 k/ft   0.020 k/ft   0.020 k/ft   0.020 k/ft   0.020 k/ft
               0.020 k/ft
Conc wt in addition to slab thickness and haunch included in dist dead load
               0.089 k/ft   0.089 k/ft   0.089 k/ft   0.089 k/ft   0.089 k/ft
               0.089 k/ft   0.089 k/ft   0.089 k/ft   0.089 k/ft   0.089 k/ft
               0.089 k/ft
Superimposed dead load
                                0.123 k/ft   0.123 k/ft   0.123 k/ft   0.123 k/ft
                                0.123 k/ft   0.123 k/ft   0.123 k/ft   0.123 k/ft
                                0.123 k/ft   0.123 k/ft   0.123 k/ft
Wearing surface dead load
                                0.169 k/ft   0.169 k/ft   0.169 k/ft   0.169 k/ft
```

OUTPUT

0.169 k/ft 0.169 k/ft 0.169 k/ft 0.169 k/ft
0.169 k/ft 0.169 k/ft 0.169 k/ft

Load Factors

DC1,DC2	1.250
DW	1.500
HL93 LL+I	1.750
Constructibility	1.250

Load Modifiers

Ductility	1.00
Redundancy	1.00
Operational Classification	1.00

Reactions

Max unfactored live load+impact reactions

79.00 k 79.00 k

Min unfactored live load+impact reactions

0.00 k 0.00 k

Max unfactored live reactions - No dynamic load allowance

64.18 k 64.18 k

Min unfactored live reactions - No dynamic load allowance

0.00 k 0.00 k

Total unfactored dead load DC1+DC2 reactions

46.91 k 48.56 k

Total unfactored dead load DW reactions

7.26 k 7.26 k

Support skew for shear and moment modification

OUTPUT

90.00 90.00

Bearing skew for redistribution qualification

99.24 99.24

Material

Concrete

Concrete strength	4.00 ksi			
Unit wt of concrete	150. lb/cu ft			
Aggregate source correction factor K1	1.00			
Slab T for strength	8.25 in			
Effective slab width	54.75 in	61.88 in	67.63 in	72.00 in
	75.25 in	77.00 in	77.50 in	76.63 in
	74.50 in	71.13 in	66.25 in	
Fillet	2.00 in			
Effective slab width by tenth pt				
	54.75 in	61.88 in	67.63 in	72.00 in
	75.25 in	77.00 in	77.50 in	76.63 in
	74.50 in	71.13 in	66.25 in	
Self weight slab width by tenth pt				
	54.75 in	61.88 in	67.63 in	72.00 in
	75.25 in	77.00 in	77.50 in	76.63 in
	74.50 in	71.13 in	66.25 in	

Steel

GILL ENGINEERING ASSOCIATES, INC.
63 KENDRICK STREET, NEEDHAM, MA 02494

OUTPUT

Web splice section 1

Steel grade M270-50

Rebar yield 60.00 ksi

Output

Standard resolution summary tables

Units

Input units: U.S. cust.

Output units: U.S. cust.

♀ Service Shear

Service Shear - k - (load modifier not included)

Tenth Point	Loc	DC1	DC2	DW	Design Truck+ Lane LL+I (+)	Design Truck+ Lane LL+I (-)	Tandem Truck+ Lane LL+I	Fatigue Truck Range LL+I
0	0.00	40.96	5.29	7.26	79.00	0.00	64.72	38.45
1	8.59	33.67	4.23	5.81	69.03	-3.17	56.40	34.65
2	17.19	25.86	3.17	4.36	59.05	-7.00	48.48	31.94
3	25.78	17.64	2.11	2.90	49.46	-15.14	40.93	29.69
4	34.38	9.11	1.06	1.45	40.25	-23.00	33.78	27.52

OUTPUT

5	42.97	0.39	0.00	0.00	31.43	-31.43	27.00	28.19
6	51.56	-8.43	-1.06	-1.45	23.00	-40.25	-33.78	28.88
7	60.16	-17.24	-2.11	-2.90	15.14	-49.46	-40.93	31.05
8	68.75	-25.92	-3.17	-4.36	8.22	-59.05	-48.48	33.22
9	77.35	-34.38	-4.23	-5.81	3.17	-69.03	-56.40	35.73
10	85.94	-42.51	-5.29	-7.26	0.00	-79.00	-64.72	38.45

	Fatigue Trk LL+I	Permit Trk LL+I	Permit Range	Sidewalk
0	38.45	0.00	0.00	0.00
1	33.56	0.00	0.00	0.00
2	28.68	0.00	0.00	0.00
3	23.79	0.00	0.00	0.00
4	18.91	0.00	0.00	0.00
5	-14.09	0.00	0.00	0.00
6	-18.91	0.00	0.00	0.00
7	-23.79	0.00	0.00	0.00
8	-28.68	0.00	0.00	0.00
9	-33.56	0.00	0.00	0.00
10	-38.45	0.00	0.00	0.00

♀ Reactions

Girder 1 Factored Reactions - Strength I - k

OUTPUT

Location	DC1	DC2	DW	LL+I Max	LL+I Min	Max Total	Min Total
0.00	52.03	6.61	10.89	138.25	0.00	207.78	46.94
Steel	9.84						
Conc	42.18						
85.94	54.09	6.61	10.89	138.25	0.00	209.85	48.42
Steel	9.84						
Conc	44.25						

Includes ductility, redundancy, and operational factors.

See 5th Ed.LRFD, commentary pg 3.11, for min total calcs

Note: DC1 reactions at end supports in girder output account for the additional weight of the extension of the girder past the end bearing locations, based on GDREXT girder input (which defaults to 1.0 ft if not specified).

Unfactored Reactions - k

OUTPUT

Location	DC1	DC2	DW	LL+I Max	LL+I Min	Max Total	Min Total
0.00	41.62	5.29	7.26	79.00	0.00	133.17	54.17
Steel	7.87						
Conc	33.75						
85.94	43.27	5.29	7.26	79.00	0.00	134.82	55.82
Steel	7.87						
Conc	35.40						

♀ Service Moment

Service Moment - k-ft - (load modifier not included)

Tenth Point	Loc	DC1	DC2	DW	Design Truck+ Lane Max LL+I	Design Truck+ Lane Min LL+I	Tandem Truck+ Lane Max LL+I	Tandem Truck+ Lane Min LL+I	Fatigue Truck Range LL+I
0	0.0	0.	0.	0.	0.	0.	0.	0.	0.

OUTPUT

1	8.6	321.	41.	56.	519.	0.	435.	0.	246.
2	17.2	577.	73.	100.	908.	0.	772.	0.	423.
3	25.8	764.	95.	131.	1184.	0.	1011.	0.	532.
4	34.4	879.	109.	150.	1351.	0.	1150.	0.	573.
5	43.0	920.	114.	156.	1388.	0.	1191.	0.	564.
6	51.6	886.	109.	150.	1351.	0.	1150.	0.	573.
7	60.2	776.	95.	131.	1184.	0.	1011.	0.	532.
8	68.8	590.	73.	100.	908.	0.	772.	0.	423.
9	77.3	331.	41.	56.	519.	0.	435.	0.	246.
10	85.9	0.	0.	0.	0.	0.	0.	0.	0.

Bracing

21.5	680.	85.	117.	1060.	0.	904.	0.
43.0	920.	114.	156.	1388.	0.	1191.	0.
64.5	692.	85.	117.	1060.	0.	903.	0.

	Sidewalk Max	Sidewalk Min	Max LL+I Fat Trk	Min LL+I Fat Trk	Max LL+I Prmt Trk	Min LL+I Prmt Trk
0	0.	0.	0.	0.	0.	0.
1	0.	0.	246.	0.	0.	0.
2	0.	0.	423.	0.	0.	0.

OUTPUT

3	0.	0.	532.	0.	0.	0.
4	0.	0.	573.	0.	0.	0.
5	0.	0.	564.	0.	0.	0.
6	0.	0.	573.	0.	0.	0.
7	0.	0.	532.	0.	0.	0.
8	0.	0.	423.	0.	0.	0.
9	0.	0.	246.	0.	0.	0.
10	0.	0.	0.	0.	0.	0.

♀ Service Force Summary Tables

Girder 1 Force Summary at Tenth Points (k,k-ft)

Location	DL-S	SDL-S	LLI-S	DL-M	SDL-M	LLI+M	LLI-M	DL-T	SDL-T	LLI-T
0.0	41	13	79	0	0	0	0	0	0	0
8.5	34	10	69	321	97	519	0	0	0	0
17.1	26	8	59	577	173	908	0	0	0	0
25.7	18	5	49	764	226	1184	0	0	0	0
34.3	9	3	40	879	259	1351	0	0	0	0
42.9	0	0	31	920	270	1388	0	0	0	0
51.5	-8	-3	-40	886	259	1351	0	0	0	0
60.1	-17	-5	-49	776	226	1184	0	0	0	0
68.7	-26	-8	-59	590	173	908	0	0	0	0
77.3	-34	-10	-69	331	97	519	0	0	0	0

OUTPUT

85.9	-43	-13	-79	0	0	0	0	0	0	0

Girder 1 Force Summary at Quarter Points (k,k-ft)

Location	DL-S	SDL-S	LLI-S	DL-M	SDL-M	LLI+M	LLI-M	DL-T	SDL-T	LLI-T
0.0	41	13	79	0	0	0	0	0	0	0
21.4	22	6	54	680	202	1060	0	0	0	0
42.9	0	0	31	920	270	1388	0	0	0	0
64.4	-22	-6	-54	692	202	1060	0	0	0	0
85.9	-43	-13	-79	0	0	0	0	0	0	0

Girder 1 Force Summary at Brace Locations (k,k-ft)

Location	DL-S	SDL-S	LLI-S	DL-M	SDL-M	LLI+M	LLI-M	DL-T	SDL-T	LLI-T
21.4	22	6	54	680	202	1060	0	0	0	0
42.9	0	0	31	920	270	1388	0	0	0	0
42.9	0	0	31	920	270	1388	0	0	0	0
64.4	-22	-6	-54	692	202	1060	0	0	0	0

85.9	-43	-13	-79	0	0	0	0	0	0	0

Key

DL-S	Noncomp dead shear
SDL-S	Superimposed dead shear
LLI-S	Live+impact shear
DL-M	Noncomp dead moment
SDL-M	Superimposed dead moment
LLI+M	Max live+impact moment
LLI-M	Min live+impact moment
DL-T	Noncomp dead torque
SDL-M	Superimposed dead torque
LLI-T	Max live+impact torque

Shear on right side of pier is found in Service Shear table.

♀ Combined Factored Moment - Strength I Loading

Combined Factored Moment - Strength I - HL93 - k-ft

Tenth Pt	Loc	DC1+DC2+DW+Max LL+I	DC1+DC2+DW+Min LL+I
Load modifiers included			
0	0.00	0.00	0.00
1	8.59	1444.55	536.72
2	17.19	2551.64	962.13

OUTPUT

3	25.78	3343.83	1271.27
4	34.38	3824.41	1460.31
5	42.97	3955.96	1526.48
6	51.56	3832.36	1468.25
7	60.16	3357.77	1285.20
8	68.75	2567.57	978.06
9	77.35	1456.51	548.68
10	85.94	0.00	0.00

♀ Dimensions

Dimensions

Tenth Point	Loc	-Top Flange- tfw	tft	Weld	----Web---- wd	wt	Weld	-Bot Flange- bfw	bft	Area
0	0.00	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88
1	8.59	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88
2	17.19	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88
3	25.78	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88
4	34.38	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88
5	42.97	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88
6	51.56	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88
7	60.16	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88
8	68.75	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88
9	77.35	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88
10	85.94	16.00	0.812	5/16	34.00	0.438	5/16	16.00	1.188	46.88

OUTPUT

Bearing Stiffeners

Location	Width	Thickness
0.00	7.75	0.750
85.94	7.75	0.750

♀ Elastic Section Properties for Stiffness Analysis

Elastic Section Properties for Stiffness Analysis

	----- Noncomp -----			------ Comp ------ (n = 8.00)			------ Comp ------ (3n =24.00)		
Tenth Point	I	na from Bott	na from Top Flg	I	na from Bott	na from Top Flg	I	na from Bott	na from Top Flg
0	11007.	15.88	20.12	26378.	29.13	6.87	19005.	22.83	13.17
1	11007.	15.88	20.12	27250.	29.86	6.14	19726.	23.45	12.55
2	11007.	15.88	20.12	27877.	30.38	5.62	20271.	23.92	12.08
3	11007.	15.88	20.12	28314.	30.74	5.26	20666.	24.26	11.74
4	11007.	15.88	20.12	28620.	31.00	5.00	20949.	24.50	11.50
5	11007.	15.88	20.12	28779.	31.13	4.87	21098.	24.63	11.37
6	11007.	15.88	20.12	28823.	31.16	4.84	21140.	24.67	11.33
7	11007.	15.88	20.12	28745.	31.10	4.90	21067.	24.61	11.39

OUTPUT

8	11007.	15.88	20.12	28551.	30.94	5.06	20885.	24.45	11.55
9	11007.	15.88	20.12	28229.	30.67	5.33	20589.	24.19	11.81
10	11007.	15.88	20.12	27732.	30.26	5.74	20143.	23.81	12.19

♀ Elastic Section Properties for Pos Mom Stress

Elastic Section Properties for Pos Mom Stress

	----- Noncomp -----			------ Comp ------ (n = 8.00)			------ Comp ------ (3n =24.00)		
Tenth Point	I	na from Bott	na from Top Flg	I	na from Bott	na from Top Flg	I	na from Bott	na from Top Flg
0	11007.	15.88	20.12	26378.	29.13	6.87	19005.	22.83	13.17
1	11007.	15.88	20.12	27250.	29.86	6.14	19726.	23.45	12.55
2	11007.	15.88	20.12	27877.	30.38	5.62	20271.	23.92	12.08
3	11007.	15.88	20.12	28314.	30.74	5.26	20666.	24.26	11.74
4	11007.	15.88	20.12	28620.	31.00	5.00	20949.	24.50	11.50
5	11007.	15.88	20.12	28779.	31.13	4.87	21098.	24.63	11.37
6	11007.	15.88	20.12	28823.	31.16	4.84	21140.	24.67	11.33
7	11007.	15.88	20.12	28745.	31.10	4.90	21067.	24.61	11.39
8	11007.	15.88	20.12	28551.	30.94	5.06	20885.	24.45	11.55
9	11007.	15.88	20.12	28229.	30.67	5.33	20589.	24.19	11.81
10	11007.	15.88	20.12	27732.	30.26	5.74	20143.	23.81	12.19

♀ Plastic Section Properties

OUTPUT

Plastic Section Properties

Tenth Point	Ps+Pr	Ptf+Pw +Pbf	PNA Frm tab D6.1-1 +Mp	PNA -Mp	Pos Mp (k-ft)	Case	Neg Mp (k-ft)	Case
0	1536.	2344.	8.76	25.42	4440.	II	3476.	I
1	1736.	2344.	8.63	25.42	4516.	II	3476.	I
2	1897.	2344.	8.53	25.42	4576.	II	3476.	I
3	2020.	2344.	8.45	25.42	4621.	II	3476.	I
4	2111.	2344.	8.40	25.42	4653.	II	3476.	I
5	2160.	2344.	8.36	25.42	4671.	II	3476.	I
6	2174.	2344.	8.36	25.42	4676.	II	3476.	I
7	2149.	2344.	8.37	25.42	4667.	II	3476.	I
8	2090.	2344.	8.41	25.42	4646.	II	3476.	I
9	1995.	2344.	8.47	25.42	4612.	II	3476.	I
10	1858.	2344.	8.55	25.42	4562.	II	3476.	I

Note: PNA is shown as measured from top of slab

♀ Top Flange Factored Bending Stress - Strength I Loading

Factored Bending Stresses - Strength I - ksi

Tenth Point	LOC	------------------ Top Flange -------------------							
		DC1	DC2+DW	Max	Min	LL+I	Tot	Allow	Ratio

OUTPUT

				LL+I	LL+I	Rng			
0	0.00	0.0	0.0	0.0	0.0	0.0	0.0	50.0	0.000
1	8.59	-8.8	-1.0	-2.5	0.0	2.5	-12.3	50.0	0.246
2	17.19	-15.8	-1.7	-3.8	0.0	3.8	-21.4	50.0	0.428
3	25.78	-21.0	-2.2	-4.6	0.0	4.6	-27.7	50.0	0.554
4	34.38	-24.1	-2.4	-5.0	0.0	5.0	-31.4	50.0	0.629
5	42.97	-25.2	-2.4	-4.9	0.0	4.9	-32.6	50.0	0.652
6	51.56	-24.3	-2.3	-4.8	0.0	4.8	-31.4	50.0	0.627
7	60.16	-21.3	-2.0	-4.2	0.0	4.2	-27.5	50.0	0.551
8	68.75	-16.2	-1.6	-3.4	0.0	3.4	-21.2	50.0	0.423
9	77.35	-9.1	-0.9	-2.1	0.0	2.1	-12.1	50.0	0.241
10	85.94	0.0	0.0	0.0	0.0	0.0	0.0	50.0	0.000

Governing expression for allowable compression stress

Tenth Pt	Expression			
1	(6.10.8.2.3-2)	Cb 1.5879	Rb 1.0000	Rh 1.0000
2	(6.10.8.2.3-2)	Cb 1.5879	Rb 1.0000	Rh 1.0000
3	(6.10.8.2.3-2)	Cb 1.0569	Rb 1.0000	Rh 1.0000
4	(6.10.8.2.3-2)	Cb 1.0569	Rb 1.0000	Rh 1.0000
5	(6.10.8.2.3-2)	Cb 1.0569	Rb 1.0000	Rh 1.0000
6	(6.10.8.2.3-2)	Cb 1.0553	Rb 1.0000	Rh 1.0000
7	(6.10.8.2.3-2)	Cb 1.0553	Rb 1.0000	Rh 1.0000
8	(6.10.8.2.3-2)	Cb 1.5576	Rb 1.0000	Rh 1.0000

OUTPUT

9 (6.10.8.2.3-2) Cb 1.5599 Rb 1.0000 Rh 1.0000

Perf. ratio for compact section is (factored mom/mom strength)
if this is less than (total stress/allowable stress)

♀ Shear Stress

Shear Stress - ksi

Tenth	Loc	Total	Allow	Ratio
0	0.00	13.9	21.9	0.636
1	8.59	11.9	21.9	0.544
2	17.19	9.8	21.9	0.449
3	25.78	7.8	21.9	0.355
4	34.38	5.7	21.9	0.262
5	42.97	3.7	21.9	0.171
6	51.56	5.7	21.9	0.260
7	60.16	7.7	21.9	0.354
8	68.75	9.8	21.9	0.450
9	77.35	12.0	21.9	0.547
10	85.94	14.0	21.9	0.642

♀ Bottom Flange Factored Bending Stress - Strength I Loading

Factored Bending Stresses - Strength I - ksi

OUTPUT

Tenth Point	Loc	DC1	DC2+DW	Max LL+I	Min LL+I	LL+I Rng	Wind	Tot	Allow	Ratio
		Bottom Flange								
0	0.00	0.0	0.0	0.0	0.0	0.0	0.0	0.0	50.0	0.000
1	8.59	7.0	1.9	11.9	0.0	11.9	0.0	20.8	50.0	0.341
2	17.19	12.5	3.4	20.8	0.0	20.8	0.0	36.7	50.0	0.593
3	25.78	16.5	4.4	27.0	0.0	27.0	0.0	48.0	50.0	0.768
4	34.38		Factored stresses in inelastic range. **							0.872
5	42.97		Factored stresses in inelastic range. **							0.898
6	51.56		Factored stresses in inelastic range. **							0.869
7	60.16	16.8	4.4	26.9	0.0	26.9	0.0	48.1	50.0	0.763
8	68.75	12.8	3.4	20.7	0.0	20.7	0.0	36.8	50.0	0.586
9	77.35	7.2	1.9	11.8	0.0	11.8	0.0	20.9	50.0	0.335
10	85.94	0.0	0.0	0.0	0.0	0.0	0.0	0.0	50.0	0.000

** Permitted for compact section (or Appendix A6), if the factored moment strength is not exceeded by the factored moment.

Governing expression for allowable compression stress

Tenth Pt Expression

OUTPUT

Perf. ratio for compact section is (factored mom/mom strength)
if this is less than (total stress/allowable stress)

♀ Factored Slab and Rebar Stresses

Factored Bending Stresses - ksi

Tenth Point	Loc	Slab Comp Dead	Slab Max LL+I	Slab Tot	Rebars Comp Dead	Rebars Min LL+I	Rebars Tot
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	8.59	-0.12	-0.82	-0.94	0.00	0.00	0.00
2	17.19	-0.21	-1.36	-1.56	0.00	0.00	0.00
3	25.78	-0.26	-1.70	-1.96	0.00	0.00	0.00
4	34.38	-0.29	-1.89	-2.18	0.00	0.00	0.00
5	42.97	-0.30	-1.92	-2.21	0.00	0.00	0.00
6	51.56	-0.28	-1.86	-2.14	0.00	0.00	0.00
7	60.16	-0.25	-1.64	-1.89	0.00	0.00	0.00
8	68.75	-0.19	-1.28	-1.47	0.00	0.00	0.00
9	77.35	-0.11	-0.75	-0.86	0.00	0.00	0.00
10	85.94	0.00	0.00	0.00	0.00	0.00	0.00

Only slab compression stresses tabulated.

♀ Constructibility of Web in Bending

OUTPUT

Compression Bending Stress for Constructibility - ksi

Tenth Point	Noncomp. Dead Factored Mom	Noncomp. Dead Web Bending Stress	Rb	Rh	Web Allow 6.10.1.9	Ratio
0	0.00	0.00	1.0000	1.0000	50.00	0.000
1	401.37	8.45	1.0000	1.0000	48.50*	0.181
2	721.51	15.18	1.0000	1.0000	48.50*	0.326
3	955.45	20.11	1.0000	1.0000	43.89*	0.477
4	1099.37	23.14	1.0000	1.0000	43.89*	0.549
5	1150.51	24.21	1.0000	1.0000	43.89*	0.575
6	1107.32	23.30	1.0000	1.0000	43.82*	0.554
7	969.38	20.40	1.0000	1.0000	43.82*	0.485
8	737.44	15.52	1.0000	1.0000	48.50*	0.333
9	413.32	8.70	1.0000	1.0000	48.50*	0.187
10	0.00	0.00	1.0000	1.0000	50.00	0.000

* Compression flange allowable stress, based on lateral-torsional or local buckling limits, with the ratio representing (flange stress)/(flange allowable stress).

♀ Top Flange Permanent Deflection Control Stress

Top Flange 6.10.4.2 Permanent Deflection Control - ksi

OUTPUT

Tenth	Location	DC1	DC2+ DW	1.3LL+I Max	1.3LL+I Min	Total	Allowable	Ratio
0	0.00	0.00	0.00	0.00	0.00	0.00	47.50	0.000
1	8.59	-7.04	-0.74	-1.82	0.00	-9.61	47.50	0.202
2	17.19	-12.66	-1.23	-2.86	0.00	-16.75	47.50	0.353
3	25.78	-16.76	-1.54	-3.43	0.00	-21.74	47.50	0.458
4	34.38	-19.29	-1.70	-3.68	0.00	-24.68	47.50	0.520
5	42.97	-20.19	-1.74	-3.67	0.00	-25.60	47.50	0.539
6	51.56	-19.43	-1.66	-3.54	0.00	-24.63	47.50	0.519
7	60.16	-17.01	-1.47	-3.15	0.00	-21.63	47.50	0.455
8	68.75	-12.94	-1.15	-2.51	0.00	-16.60	47.50	0.349
9	77.35	-7.25	-0.67	-1.53	0.00	-9.45	47.50	0.199
10	85.94	0.00	0.00	0.00	0.00	0.00	47.50	0.000

♀ Bottom Flange Permanent Deflection Control Stress

Bottom Flange 6.10.4.2 Permanent Deflection Control - ksi

Tenth	Location	DC1	DC2+ DW	1.3LL+I Max	1.3LL+I Min	Total	Allowable	Ratio
0	0.00	0.00	0.00	0.00	0.00	0.00	47.50	0.000
1	8.59	5.56	1.38	8.87	0.00	15.81	47.50	0.333
2	17.19	10.00	2.44	15.44	0.00	27.88	47.50	0.587
3	25.78	13.24	3.19	20.06	0.00	36.49	47.50	0.768
4	34.38	15.23	3.63	22.82	0.00	41.69	47.50	0.878

OUTPUT

5	42.97	15.94	3.78	23.42	0.00	43.14	47.50	0.908
6	51.56	15.34	3.62	22.79	0.00	41.75	47.50	0.879
7	60.16	13.43	3.17	19.99	0.00	36.59	47.50	0.770
8	68.75	10.22	2.42	15.35	0.00	27.99	47.50	0.589
9	77.35	5.73	1.37	8.79	0.00	15.89	47.50	0.334
10	85.94	0.00	0.00	0.00	0.00	0.00	47.50	0.000

♀ Factored Noncomp Dead Wet Concrete Stress

Factored Noncomp Dead Wet Concrete Stresses - ksi

[While the constructibility load factor is used for this table, slab pouring is not. Actual stresses are dependent on the pouring sequence and may be significantly different from those printed here.]

Tenth Point	Loc	Top Flange	Allow-able	Bottom Flange	Allow-able
0	0.0	0.0	50.0	0.0	50.0
1	8.6	-8.8	48.5	7.0	50.0
2	17.2	-15.8	48.5	12.5	50.0
3	25.8	-21.0	43.9	16.5	50.0
4	34.4	-24.1	43.9	19.0	50.0
5	43.0	-25.2	43.9	19.9	50.0
6	51.6	-24.3	43.8	19.2	50.0

7	60.2	-21.3	43.8	16.8	50.0
8	68.8	-16.2	48.5	12.8	50.0
9	77.3	-9.1	48.5	7.2	50.0
10	85.9	0.0	50.0	0.0	50.0

Construction load factor 1.25 being used for this table.

♀ Factored Strengths

Factored Strengths

Forces include ductility, redundancy, and operational factors

Tenth Point	Loc	Factored Shear (k)	Shear Strength (k)	Ratio	Factored Moment (k-ft)	Bending Strength (k-ft)	Ratio
		HL93			HL93		
0	0.00	207.0	325.2 (5)	0.636	0.0	4162.4 (20)	0.000
1	8.59	176.9	325.2 (5)	0.544	1444.6 I	4242.3 (20)	0.341
2	17.19	146.2	325.2 (5)	0.449	2551.6 I	4305.5 (20)	0.593
3	25.78	115.6	325.2 (5)	0.355	3343.8 I	4352.8 (20)	0.768
4	34.38	85.3	325.2 (5)	0.262	3824.4 I	4387.6 (20)	0.872

OUTPUT

5	42.97	55.5	325.2 (5)	0.171	3956.0 I	4406.2 (20) 0.898
6	51.56	84.5	325.2 (5)	0.260	3832.4 I	4411.5 (20) 0.869
7	60.16	115.1	325.2 (5)	0.354	3357.8 I	4402.3 (20) 0.763
8	68.75	146.2	325.2 (5)	0.450	2567.6 I	4379.6 (20) 0.586
9	77.35	177.8	325.2 (5)	0.547	1456.5 I	4343.5 (20) 0.335
10	85.94	208.9	325.2 (5)	0.642	0.0	4290.4 (20) 0.000

Absolute values of factored moment are shown with determining strength

(5) C 0.58 Fyw D t

(20) Compact section

♀ Lrfd Ratings

[This table uses the rating equation (6B.4.1-1) of the 2011 edition of the Manual for Bridge Evaluation where A1D is the sum of factored composite dead, noncomposite dead, and factored wearing surface loads. The denominator A2L(1+I) is the factored live load. If equation (6a.4.2.1-1) is to be used for LRFR ratings the condition LRFR RATINGS must be used in the girder input file.]

OUTPUT

HL93

Strength I

Span 1

Location	Compct Mom Cap/Noncmpt Allow Stress	Shear Capacity	Rating Factors Bending Inv.	Bending Oper.	Shear Inv.	Shear Oper.
0.00	4162. C	325.20	>999.00 B	>999.00	1.86	3.25
8.59	4242. C	325.20	4.08 B	7.14	2.23	3.90
17.19	4306. C	325.20	2.10 B	3.68	2.73	4.78
25.78	4353. C	325.20	1.49 B	2.60	3.42	5.99
34.38	4388. C	325.20	1.24 B	2.17	4.41	7.71
42.97	4406. C	325.20	1.19 B	2.07	5.90	10.33
51.56	4411. C	325.20	1.24 B	2.18	4.42	7.73
60.16	4402. C	325.20	1.50 B	2.63	3.43	6.00

OUTPUT

68.75	4380. C	325.20	2.14 B	3.75	2.73	4.78
77.35	4343. C	325.20	4.18 B	7.32	2.22	3.89
85.94	4290. C	325.20	>999.00 B	>999.00	1.84	3.22

Service II

Span 1

Location	Allowable Stress	Rating Factors Bending Inv.	Oper.
0.00	47500. S	> 999.00 B	>999.00
8.59	47500. S	4.57 B	5.95
17.19	47500. S	2.27 B	2.95
25.78	47500. S	1.55 B	2.01
34.38	47500. S	1.25 B	1.63
42.97	47500. S	1.19 B	1.54
51.56	47500. S	1.25 B	1.63
60.16	47500. S	1.55 B	2.01
68.75	47500. S	2.27 B	2.95
77.35	47500. S	4.60 B	5.97
85.94	47500. S	> 999.00 B	>999.00

OUTPUT

```
****************************************************

Minimum rating is  1.19 at location  42.97 in span 1.

****************************************************



     Rating Codes:

     T - Top steel governs

     B - Bottom steel governs

     C - Concrete governs

     R - Rebar governs

     V - Shear governs

     S - Serviceability governs


     Mom Strength Codes:

     C - Compact

     B - Braced non-compact

     U - Unbraced non-compact

     T - Transition between compact and braced non-compact

     S - Serviceability


Noncompact shapes ratings based on stress, as


             Fb  -  factored dead load stress

        IR = --------------------------------

                   factored LL+I stress
```

♀ Bearing Stiffeners

Bearing Stiffeners

OUTPUT

Location from Left End of Web Sect. (ft)	Factored Reaction (k)	Allowable Column Force (k)	Ratio	Allowable Bearing Force (k)	Ratio
0.00	207.78	676.26	0.307	708.75	0.293
85.94	209.71	676.26	0.310	708.75	0.296

♀ Fatigue

Fatigue Summary

Condition	Location from Left End of Girder	Category (Table 6.6.1.2.3-1)	Force Range (k-ft)	Actual Stress Range	Allowable Stress Range	Ratio
Base metal	8.59	B	184.28	2.42	20.62	0.118
Base metal	17.19	B	317.32	4.15	20.62	0.201
Base metal	25.78	B	399.13	5.20	20.62	0.252
Base metal	34.38	B	429.70	5.58	20.62	0.271
Base metal	42.97	B	423.28	5.49	20.62	0.266
Base metal	51.56	B	429.70	5.58	20.62	0.270
Base metal	60.16	B	399.13	5.18	20.62	0.251
Base metal	68.75	B	317.32	4.13	20.62	0.200

OUTPUT

Base metal	77.35	B	184.28	2.40	20.62	0.117
Near flg-web weld	8.59	B	184.28	2.33	20.62	0.113
Near flg-web weld	17.19	B	317.32	3.99	20.62	0.193
Near flg-web weld	25.78	B	399.13	5.00	20.62	0.242
Near flg-web weld	34.38	B	429.70	5.37	20.62	0.260
Near flg-web weld	42.97	B	423.28	5.28	20.62	0.256
Near flg-web weld	51.56	B	429.70	5.36	20.62	0.260
Near flg-web weld	60.16	B	399.13	4.98	20.62	0.242
Near flg-web weld	68.75	B	317.32	3.97	20.62	0.192
Near flg-web weld	77.35	B	184.28	2.31	20.62	0.112
Full depth conn PL	21.49	C'	358.27	4.49	14.76	0.304
Full depth conn PL	42.98	C'	423.28	5.28	14.76	0.358
Full depth conn PL	64.47	C'	358.08	4.48	14.76	0.303
Full depth conn PL	85.94	C'	0.00	0.00	14.76	0.000

♀ Max Performance Ratios

Maximum Performance Ratios

Criterion	Location ft	Max Performance Ratio
Design and tandem trucks		
Shear	85.94	0.642
Bending	42.97	0.908

OUTPUT

Fatigue truck

Fatigue	42.98	0.358
Bearing Stf.	85.94	0.310

♀ Shear Connectors

Girder 1 Shear Connectors

Maximum Pitch for 0.875 X 6.00 Stud Connectors for Fatigue

Tenth Point	Location	Shear Range (k)	Q	I	Studs in a Line	Max Pitch (in)
0	0.00	28.84	620.9	26378.	2	18.58
1	8.59	25.99	655.1	27250.	2	20.19
2	17.19	23.95	679.6	27877.	2	21.60
3	25.78	22.27	696.6	28314.	2	23.02
4	34.38	20.64	708.4	28620.	2	24.68 *
5	42.97	21.14	714.5	28779.	2	24.03 *
6	51.56	21.66	716.2	28823.	2	23.43
7	60.16	23.29	713.2	28745.	2	21.83

OUTPUT

8	68.75	24.92	705.7	28551.	2	20.48
9	77.35	26.80	693.3	28229.	2	19.16
10	85.94	28.84	673.9	27732.	2	18.00

HL93 fatigue truck shear range used for the above.

* Use 24 inches as per 6.10.10.1.2

Total Number Required for Strength

Span 1

57 between tenth points 0 and 5

57 between tenth points 5 and 10

* * * * * * * * * * * * * * * R A T E M O D E * * * * * * * * * * * * * * *

♀ Weight

OUTPUT

Girder 1 Weight

	Volume	Weight	
	cu ft	k	
Top Flange	7.76	3.80	
Web	8.88	4.35	
Bottom Flange	11.34	5.56	
Flange-Web Weld		0.06	
Bearing Stiffeners	0.46	0.22	
Steel in extensions		0.32	
Additional Steel		1.72	
Total Steel			16.03
Slab	380.43	57.06	
Flange Haunch	19.10	2.86	
Concrete in extensions		1.11	
Additional Concrete		7.65	
Total Concrete			68.69

OUTPUT

Total Steel and Concrete 84.71

Noncomposite weight data used in girder input is used for analysis loading.

♀ Deflections

Service Deflections - in

	Noncomp Dead Total	Noncomp Dead Steel Only	Noncomp Dead Slab Only	Comp Dead Excldg Wearng	Comp Dead Wearng Surfce Only	Live+I Max Up	Live+I Max Down
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	1.185	0.217	0.969	0.078	0.107	0.000	0.306
2	2.246	0.410	1.836	0.148	0.203	0.000	0.582
3	3.080	0.562	2.519	0.201	0.277	0.000	0.796
4	3.613	0.658	2.955	0.235	0.323	0.000	0.937
5	3.799	0.691	3.108	0.247	0.339	0.000	0.979
6	3.620	0.658	2.962	0.235	0.323	0.000	0.935
7	3.092	0.562	2.531	0.200	0.275	0.000	0.792
8	2.259	0.410	1.848	0.146	0.201	0.000	0.579
9	1.193	0.217	0.977	0.077	0.106	0.000	0.304
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Includes sidewalk deflections.

OUTPUT

Wearing surface deflections are for future wearing surface.

Loading for defl uses larger of des trk alone
or 0.25 des truck + full lane

	Loc							
Brace	21.49	2.701	0.493	2.208	0.177	0.243	0.000	0.698
Brace	42.98	3.799	0.691	3.108	0.247	0.339	0.000	0.979
Brace	64.47	2.703	0.491	2.212	0.175	0.241	0.000	0.693

Positive dead load deflection is downward.
Live load deflection as indicated in column heading.

♀ Support Rotations

Service Support Rotations

Clockwise Positive

Location	Total Dead Load		Live Load Range			
	Deg	Rad	Deg		Rad	
			Low	High	Low	High

OUTPUT

0.00	0.730	0.01274	0.000	0.297	0.00000	0.00518
85.94	-0.734	-0.01281	-0.277	0.000	-0.00484	0.00000

Factored Support Rotations - degrees

	Clockwise	Counterclockwise
0.00	1.432	0.000
85.94	0.000	-1.402

♀ Concurrent Max Live Reactions and Slopes

Concurrent Max Live Service Reactions and Slopes

Location	Max	Concurrent	Max	Concurrent
	Reaction	Slope	Slope	Reaction
ft	k	Deg	Deg	k

OUTPUT

0.00	79.00	0.013	0.297	52.47
85.94	79.00	-0.013	-0.277	42.70

♀ Status

run completed

Span 2 - Reactions

****

MDX Steel Highway Girder Design Program, Version 6.5.4074

Load and Resistance Factor Design - Composite Girder

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****

Apr 2, 2019 - 1:37 pm

Files: Span 2 - South Exterior.R1

R1.OUT

♀ Reactions

Girder 1 Factored Reactions - Strength I - k

Location		DC1	DC2	DW	LL+I Max	LL+I Min	Max Total	Min Total
0.00		45.03	6.61	10.89	124.43	0.00	186.96	41.90
	Steel	10.61						
	Conc	34.42						
85.94		47.05	6.61	10.89	124.43	0.00	188.98	43.35
	Steel	10.61						
	Conc	36.44						

Includes ductility, redundancy, and operational factors.

See 5th Ed.LRFD, commentary pg 3.11, for min total calcs

Span 2 - Reactions

Note: DC1 reactions at end supports in girder output account for the additional weight of the extension of the girder past the end bearing locations, based on GDREXT girder input (which defaults to 1.0 ft if not specified).

Unfactored Reactions - k

Location	DC1	DC2	DW	LL+I Max	LL+I Min	Max Total	Min Total
0.00	36.03	5.29	7.26	71.10	0.00	119.68	48.57
Steel	8.49						
Conc	27.54						
85.94	37.64	5.29	7.26	71.10	0.00	121.29	50.19
Steel	8.49						
Conc	29.15						

****

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****

Apr 2, 2019 - 1:43 pm

Files: SPAN 2 - TYPICAL INTERIOR.R1

R1.OUT

♀ Reactions

Span 2 - Reactions

Girder 1 Factored Reactions - Strength I - k

Location	DC1	DC2	DW	LL+I Max	LL+I Min	Max Total	Min Total

Includes ductility, redundancy, and operational factors.

Location	DC1	DC2	DW	LL+I Max	LL+I Min	Max Total	Min Total
0.00	54.68	6.61	10.89	161.96	0.00	234.14	48.85
Steel	11.69						
Conc	42.99						
85.94	54.68	6.61	10.89	161.96	0.00	234.14	48.85
Steel	11.69						
Conc	42.99						

See 5th Ed.LRFD, commentary pg 3.11, for min total calcs

Note: DC1 reactions at end supports in girder output account for the additional weight of the extension of the girder past the end bearing locations, based on GDREXT girder input (which defaults to 1.0 ft if not specified).

Unfactored Reactions - k

Location	DC1	DC2	DW	LL+I Max	LL+I Min	Max Total	Min Total

Span 2 - Reactions

```
    0.00    43.74     5.29     7.26    92.55      0.00     148.84    56.29
     Steel    9.35
     Conc    34.40



   85.94    43.74     5.29     7.26    92.55      0.00     148.84    56.29
     Steel    9.35
     Conc    34.40
```

****

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****

Apr 2, 2019 - 1:34 pm

Files: Span 2 - North Exterior.R1

R1.OUT

♀ Reactions

Girder 1 Factored Reactions - Strength I - k

```
 Location   DC1      DC2      DW       LL+I     LL+I       Max      Min
                                       Max      Min        Total    Total
         Includes ductility, redundancy, and operational factors.



    0.00    52.80     6.61    10.89   138.25      0.00     208.55    47.49
     Steel   10.61
     Conc    42.18
```

Span 2 - Reactions

85.94	54.86	6.61	10.89	138.25	0.00	210.62	48.98
Steel	10.61						
Conc	44.25						

See 5th Ed.LRFD, commentary pg 3.11, for min total calcs

Note: DC1 reactions at end supports in girder output account for the additional weight of the extension of the girder past the end bearing locations, based on GDREXT girder input (which defaults to 1.0 ft if not specified).

Unfactored Reactions - k

Location	DC1	DC2	DW	LL+I Max	LL+I Min	Max Total	Min Total
0.00	42.24	5.29	7.26	79.00	0.00	133.79	54.78
Steel	8.49						
Conc	33.75						
85.94	43.89	5.29	7.26	79.00	0.00	135.44	56.44
Steel	8.49						
Conc	35.40						

Span 1 - LL Deflection

****

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****

Mar 27, 2019 - 0:25 am

Files: Span 1 - South Exterior.R1

R1.OUT

♀ Deflections

Service Deflections - in

	Noncomp Dead Total	Noncomp Dead Steel Only	Noncomp Dead Slab Only	Comp Dead Excldg Wearng	Comp Dead Wearng Surfce Only	Live+I Max Up	Live+I Max Down
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	0.902	0.197	0.705	0.075	0.104	0.000	0.299
2	1.706	0.373	1.333	0.143	0.197	0.000	0.571
3	2.336	0.511	1.825	0.196	0.270	0.000	0.782
4	2.736	0.599	2.137	0.230	0.316	0.000	0.925
5	2.874	0.629	2.245	0.242	0.332	0.000	0.966
6	2.738	0.599	2.139	0.230	0.316	0.000	0.924
7	2.340	0.511	1.828	0.196	0.269	0.000	0.781
8	1.710	0.373	1.337	0.143	0.196	0.000	0.569
9	0.904	0.197	0.707	0.075	0.103	0.000	0.298
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Span 1 - LL Deflection

Includes sidewalk deflections.

Wearing surface deflections are for future wearing surface.

Loading for defl uses larger of des trk alone
or 0.25 des truck + full lane

****

MDX Steel Highway Girder Design Program, Version 6.5.4113

Load and Resistance Factor Design - Composite Girder

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****

Apr 26, 2019 - 10:11 am

Files: SPAN 1 - TYPICAL INTERIOR.R1
R1.OUT

♀ Deflections

Service Deflections - in

	Noncomp Dead Total	Noncomp Dead Steel Only	Noncomp Dead Slab Only	Comp Dead Excldg Wearng	Comp Dead Wearng Surfce Only	Live+I Max Up	Live+I Max Down
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	1.072	0.219	0.853	0.069	0.095	0.000	0.278

Span 1 - LL Deflection

2	2.029	0.415	1.614	0.131	0.180	0.000	0.530
3	2.779	0.568	2.211	0.179	0.246	0.000	0.725
4	3.255	0.666	2.590	0.210	0.288	0.000	0.857
5	3.418	0.699	2.719	0.220	0.303	0.000	0.895
6	3.255	0.666	2.590	0.210	0.288	0.000	0.857
7	2.779	0.568	2.211	0.179	0.246	0.000	0.725
8	2.029	0.415	1.614	0.131	0.180	0.000	0.530
9	1.072	0.219	0.853	0.069	0.095	0.000	0.278
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Includes sidewalk deflections.

Wearing surface deflections are for future wearing surface.

Loading for defl uses larger of des trk alone
or 0.25 des truck + full lane

****

MDX Steel Highway Girder Design Program, Version 6.5.4113

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****

Apr 26, 2019 - 10:10 am

Files: Span 1 - North Exterior.R1

R1.OUT

♀ Deflections

Service Deflections - in

Span 1 - LL Deflection

	Noncomp Dead Total	Noncomp Dead Steel Only	Noncomp Dead Slab Only	Comp Dead Excldg Wearng	Comp Dead Wearng Surfce Only	Live+I Max Up	Live+I Max Down
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	1.050	0.197	0.852	0.072	0.098	0.000	0.285
2	1.988	0.373	1.615	0.135	0.186	0.000	0.544
3	2.725	0.511	2.214	0.185	0.254	0.000	0.743
4	3.194	0.599	2.596	0.216	0.297	0.000	0.877
5	3.356	0.629	2.727	0.227	0.311	0.000	0.915
6	3.197	0.599	2.598	0.216	0.297	0.000	0.876
7	2.729	0.511	2.217	0.184	0.253	0.000	0.742
8	1.992	0.373	1.619	0.135	0.185	0.000	0.542
9	1.052	0.197	0.855	0.071	0.098	0.000	0.285
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Includes sidewalk deflections.

Wearing surface deflections are for future wearing surface.

Loading for defl uses larger of des trk alone
or 0.25 des truck + full lane

Span 2 - LL Deflection

****

MDX Steel Highway Girder Design Program, Version 6.5.4113

Load and Resistance Factor Design - Composite Girder

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Apr 26, 2019 - 10:13 am

Files: Span 2 - South Exterior.R1

R1.OUT

♀ Deflections

Service Deflections - in

	Noncomp Dead Total	Noncomp Dead Steel Only	Noncomp Dead Slab Only	Comp Dead Excldg Wearng	Comp Dead Wearng Surfce Only	Live+I Max Up	Live+I Max Down
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	0.957	0.217	0.740	0.084	0.116	0.000	0.326
2	1.811	0.410	1.401	0.159	0.219	0.000	0.622
3	2.481	0.562	1.919	0.218	0.300	0.000	0.853
4	2.908	0.658	2.250	0.256	0.352	0.000	1.006
5	3.057	0.691	2.366	0.268	0.369	0.000	1.051
6	2.914	0.658	2.257	0.255	0.350	0.000	1.003
7	2.492	0.562	1.930	0.217	0.298	0.000	0.847
8	1.823	0.410	1.412	0.158	0.217	0.000	0.617
9	0.964	0.217	0.747	0.083	0.114	0.000	0.323
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Span 2 - LL Deflection

Includes sidewalk deflections.

Wearing surface deflections are for future wearing surface.

Loading for defl uses larger of des trk alone
or 0.25 des truck + full lane

****
MDX Steel Highway Girder Design Program, Version 6.5.4113
Load and Resistance Factor Design - Composite Girder

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****

Apr 26, 2019 - 10:14 am
Files: SPAN 2 - TYPICAL INTERIOR.R1
R1.OUT

♀ Deflections

Service Deflections - in

	Noncomp Dead Total	Noncomp Dead Steel Only	Noncomp Dead Slab Only	Comp Dead Excldg Wearng	Comp Dead Wearng Surfce Only	Live+I Max Up	Live+I Max Down
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	1.178	0.241	0.937	0.076	0.104	0.000	0.299

Span 2 - LL Deflection

2	2.230	0.456	1.774	0.144	0.197	0.000	0.570
3	3.053	0.624	2.429	0.197	0.270	0.000	0.781
4	3.577	0.731	2.845	0.230	0.317	0.000	0.922
5	3.756	0.768	2.988	0.242	0.332	0.000	0.964
6	3.577	0.731	2.845	0.230	0.317	0.000	0.922
7	3.053	0.624	2.429	0.197	0.270	0.000	0.781
8	2.230	0.456	1.774	0.144	0.197	0.000	0.570
9	1.178	0.241	0.937	0.076	0.104	0.000	0.299
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Includes sidewalk deflections.

Wearing surface deflections are for future wearing surface.

Loading for defl uses larger of des trk alone

or 0.25 des truck + full lane

****

MDX Steel Highway Girder Design Program, Version 6.5.4074

Load and Resistance Factor Design - Composite Girder

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Files: Span 2 - North Exterior.R1

R1.OUT

♀ Deflections

Service Deflections - in

Span 2 - LL Deflection

	Noncomp Dead Total	Noncomp Dead Steel Only	Noncomp Dead Slab Only	Comp Dead Excldg Wearng	Comp Dead Wearng Surfce Only	Live+I Max Up	Live+I Max Down
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000
1	1.185	0.217	0.969	0.078	0.107	0.000	0.306
2	2.246	0.410	1.836	0.148	0.203	0.000	0.582
3	3.080	0.562	2.519	0.201	0.277	0.000	0.796
4	3.613	0.658	2.955	0.235	0.323	0.000	0.937
5	3.799	0.691	3.108	0.247	0.339	0.000	0.979
6	3.620	0.658	2.962	0.235	0.323	0.000	0.935
7	3.092	0.562	2.531	0.200	0.275	0.000	0.792
8	2.259	0.410	1.848	0.146	0.201	0.000	0.579
9	1.193	0.217	0.977	0.077	0.106	0.000	0.304
SUPPORT	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Includes sidewalk deflections.

Wearing surface deflections are for future wearing surface.

Loading for defl uses larger of des trk alone
or 0.25 des truck + full lane

Span 1 - Shear Studs

****

MDX Steel Highway Girder Design Program, Version 6.5.4113

Load and Resistance Factor Design - Composite Girder

****

Apr 26, 2019 - 10:11 am

Files: Span 1 - South Exterior.R1

R1.OUT

♀ Shear Connectors

Girder 1 Shear Connectors

Maximum Pitch for 0.875 X 6.00 Stud Connectors for Fatigue

Tenth Point	Location	Shear Range (k)	Q	I	Studs in a Line	Max Pitch (in)
0	0.00	23.54	695.6	28290.	2	21.79
1	8.39	21.09	673.9	27732.	2	24.61 *
2	16.79	19.56	655.6	27263.	2	26.81 *
3	25.18	18.09	642.5	26927.	2	29.22 *
4	33.58	16.68	634.6	26728.	2	31.85 *
5	41.97	16.12	633.4	26697.	2	32.98 *

Span 1 - Shear Studs

6	50.36	17.30	638.3	26820.	2	30.64 *
7	58.76	18.71	649.4	27105.	2	28.13 *
8	67.15	20.12	665.5	27516.	2	25.91 *
9	75.55	21.77	686.0	28042.	2	23.68
10	83.94	23.54	709.3	28643.	2	21.64

HL93 fatigue truck shear range used for the above.

* Use 24 inches as per 6.10.10.1.2

Total Number Required for Strength

Span 1

61 between tenth points 0 and 5

61 between tenth points 5 and 10

****

MDX Steel Highway Girder Design Program, Version 6.5.4113

Load and Resistance Factor Design - Composite Girder

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****

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Files: SPAN 1 - TYPICAL INTERIOR.R1

R1.OUT

♀ Shear Connectors

Girder 1 Shear Connectors

Maximum Pitch for 0.875 X 6.00 Stud Connectors for Fatigue

Span 1 - Shear Studs

Tenth Point	Location	Shear Range (k)	Q	I	Studs in a Line	Max Pitch (in)
0	0.00	26.97	724.6	29041.	2	18.74
1	8.39	24.17	724.6	29041.	2	20.91
2	16.79	22.42	724.6	29041.	2	22.55
3	25.18	20.73	724.6	29041.	2	24.38 *
4	33.58	19.11	724.6	29041.	2	26.45 *
5	41.97	18.47	724.6	29041.	2	27.36 *
6	50.36	19.82	724.6	29041.	2	25.50 *
7	58.76	21.44	724.6	29041.	2	23.58
8	67.15	23.06	724.6	29041.	2	21.92
9	75.55	24.95	724.6	29041.	2	20.26
10	83.94	26.97	724.6	29041.	2	18.74

HL93 fatigue truck shear range used for the above.

* Use 24 inches as per 6.10.10.1.2

Total Number Required for Strength

Span 1

74 between tenth points 0 and 5

74 between tenth points 5 and 10

****

MDX Steel Highway Girder Design Program, Version 6.5.4113

Page 3

Span 1 - Shear Studs
Load and Resistance Factor Design - Composite Girder

****

Apr 26, 2019 - 10:10 am

Files: Span 1 - North Exterior.R1

R1.OUT

♀ Shear Connectors

Girder 1 Shear Connectors

Maximum Pitch for 0.875 X 6.00 Stud Connectors for Fatigue

Tenth Point	Location	Shear Range (k)	Q	I	Studs in a Line	Max Pitch (in)
0	0.00	24.52	632.8	26682.	2	21.69
1	8.39	21.97	660.6	27391.	2	23.80
2	16.79	20.38	680.6	27902.	2	25.38 *
3	25.18	18.84	694.7	28266.	2	27.24 *
4	33.58	17.37	703.0	28481.	2	29.42 *
5	41.97	16.79	706.6	28574.	2	30.38 *
6	50.36	18.02	705.7	28551.	2	28.32 *
7	58.76	19.49	700.3	28410.	2	26.26 *

Span 1 - Shear Studs

8	67.15	20.96	689.9	28143.	2	24.55 *
9	75.55	22.68	673.9	27732.	2	22.89
10	83.94	24.52	651.1	27148.	2	21.45

HL93 fatigue truck shear range used for the above.

* Use 24 inches as per 6.10.10.1.2

Total Number Required for Strength

Span 1

58 between tenth points 0 and 5

58 between tenth points 5 and 10

Span 2 - Shear Studs

****

MDX Steel Highway Girder Design Program, Version 6.5.4113

Load and Resistance Factor Design - Composite Girder

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Apr 26, 2019 - 10:13 am

Files: Span 2 - South Exterior.R1

R1.OUT

♀ Shear Connectors

Girder 1 Shear Connectors

Maximum Pitch for 0.875 X 6.00 Stud Connectors for Fatigue

Tenth Point	Location	Shear Range (k)	Q	I	Studs in a Line	Max Pitch (in)
0	0.00	27.37	675.0	27758.	2	18.95
1	8.59	24.67	652.9	27192.	2	21.30
2	17.19	22.73	635.8	26759.	2	23.35
3	25.78	21.14	624.7	26475.	2	25.29 *
4	34.38	19.59	619.6	26345.	2	27.37 *
5	42.97	20.06	622.1	26411.	2	26.68 *

Span 2 - Shear Studs

6	51.56	20.56	631.0	26634.	2	25.90 *
7	60.16	22.10	646.5	27031.	2	23.86
8	68.75	23.65	666.6	27543.	2	22.04
9	77.35	25.44	690.4	28155.	2	20.22
10	85.94	27.37	717.1	28845.	2	18.54

HL93 fatigue truck shear range used for the above.

* Use 24 inches as per 6.10.10.1.2

Total Number Required for Strength

Span 1

57 between tenth points 0 and 5

57 between tenth points 5 and 10

****

MDX Steel Highway Girder Design Program, Version 6.5.4113

Load and Resistance Factor Design - Composite Girder

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Files: SPAN 2 - TYPICAL INTERIOR.R1

R1.OUT

♀ Shear Connectors

Girder 1 Shear Connectors

Maximum Pitch for 0.875 X 6.00 Stud Connectors for Fatigue

Span 2 - Shear Studs

Tenth Point	Location	Shear Range (k)	Q	I	Studs in a Line	Max Pitch (in)
0	0.00	31.28	724.6	29041.	2	16.16
1	8.59	28.19	724.6	29041.	2	17.93
2	17.19	25.98	724.6	29041.	2	19.46
3	25.78	24.16	724.6	29041.	2	20.92
4	34.38	22.39	724.6	29041.	2	22.57
5	42.97	22.93	724.6	29041.	2	22.04
6	51.56	23.50	724.6	29041.	2	21.51
7	60.16	25.26	724.6	29041.	2	20.01
8	68.75	27.03	724.6	29041.	2	18.70
9	77.35	29.07	724.6	29041.	2	17.39
10	85.94	31.28	724.6	29041.	2	16.16

HL93 fatigue truck shear range used for the above.

Total Number Required for Strength

Span 1

74 between tenth points 0 and 5

74 between tenth points 5 and 10

****

MDX Steel Highway Girder Design Program, Version 6.5.4074

Load and Resistance Factor Design - Composite Girder

Span 2 - Shear Studs

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Files: Span 2 - North Exterior.R1

R1.OUT

♀ Shear Connectors

Girder 1 Shear Connectors

Maximum Pitch for 0.875 X 6.00 Stud Connectors for Fatigue

Tenth Point	Location	Shear Range (k)	Q	I	Studs in a Line	Max Pitch (in)
0	0.00	28.84	620.9	26378.	2	18.58
1	8.59	25.99	655.1	27250.	2	20.19
2	17.19	23.95	679.6	27877.	2	21.60
3	25.78	22.27	696.6	28314.	2	23.02
4	34.38	20.64	708.4	28620.	2	24.68 *
5	42.97	21.14	714.5	28779.	2	24.03 *
6	51.56	21.66	716.2	28823.	2	23.43
7	60.16	23.29	713.2	28745.	2	21.83
8	68.75	24.92	705.7	28551.	2	20.48

Span 2 - Shear Studs

9	77.35	26.80	693.3	28229.	2	19.16
10	85.94	28.84	673.9	27732.	2	18.00

HL93 fatigue truck shear range used for the above.

* Use 24 inches as per 6.10.10.1.2

Total Number Required for Strength

Span 1

57 between tenth points 0 and 5

57 between tenth points 5 and 10

Span 1 - Bearing Stiffeners
****

MDX Steel Highway Girder Design Program, Version 6.5.4113

Load and Resistance Factor Design - Composite Girder

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****

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Files: Span 1 - South Exterior.R1

R1.OUT

♀ Dimensions

Bearing Stiffeners

Location	Width	Thickness
0.00	7.75	0.750
83.94	7.75	0.750

♀ Bearing Stiffeners

Bearing Stiffeners

Location from Left End of Web Sect. (ft)	Factored Reaction (k)	Allowable Column Force (k)	Ratio	Allowable Bearing Force (k)	Ratio
0.00	175.99	676.26	0.260	708.75	0.248
83.94	176.60	676.26	0.261	708.75	0.249

****

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Span 1 - Bearing Stiffeners

Load and Resistance Factor Design - Composite Girder

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R1.OUT

♀ Dimensions

Bearing Stiffeners

Location	Width	Thickness
0.00	7.75	0.750
83.94	7.75	0.750

♀ Bearing Stiffeners

Bearing Stiffeners

Location from Left End of Web Sect. (ft)	Factored Reaction (k)	Allowable Column Force (k)	Ratio	Allowable Bearing Force (k)	Ratio
0.00	216.74	676.26	0.320	708.75	0.306
83.94	216.74	676.26	0.320	708.75	0.306

****

MDX Steel Highway Girder Design Program, Version 6.5.4113

Load and Resistance Factor Design - Composite Girder

Span 1 - Bearing Stiffeners

****

Apr 26, 2019 - 10:10 am

Files: Span 1 - North Exterior.R1

R1.OUT

♀ Dimensions

Bearing Stiffeners

Location	Width	Thickness
0.00	7.75	0.750
83.94	7.75	0.750

♀ Bearing Stiffeners

Bearing Stiffeners

Location from Left End of Web Sect. (ft)	Factored Reaction (k)	Allowable Column Force (k)	Ratio	Allowable Bearing Force (k)	Ratio
0.00	188.62	676.26	0.279	708.75	0.266
83.94	189.25	676.26	0.280	708.75	0.267

Span 2 - Bearing Stiffeners

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MDX Steel Highway Girder Design Program, Version 6.5.4113

Load and Resistance Factor Design - Composite Girder

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Apr 26, 2019 - 10:13 am

Files: Span 2 - South Exterior.R1

R1.OUT

♀ Dimensions

Dimensions Bearing Stiffeners

Location	Width	Thickness
0.00	7.75	0.750
85.94	7.75	0.750

♀ Bearing Stiffeners

Bearing Stiffeners

Location from Left End of Web Sect. (ft)	Factored Reaction (k)	Allowable Column Force (k)	Ratio	Allowable Bearing Force (k)	Ratio
0.00	186.19	676.26	0.275	708.75	0.263
85.94	188.08	676.26	0.278	708.75	0.265

****

Span 2 - Bearing Stiffeners

MDX Steel Highway Girder Design Program, Version 6.5.4113

Load and Resistance Factor Design - Composite Girder

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Files: SPAN 2 - TYPICAL INTERIOR.R1

R1.OUT

♀ Dimensions

Dimensions Bearing Stiffeners

Location	Width	Thickness
0.00	7.75	0.750
85.94	7.75	0.750

♀ Bearing Stiffeners

Bearing Stiffeners

Location from Left End of Web Sect. (ft)	Factored Reaction (k)	Allowable Column Force (k)	Ratio	Allowable Bearing Force (k)	Ratio
0.00	233.37	676.26	0.345	708.75	0.329
85.94	233.37	676.26	0.345	708.75	0.329

****

Span 2 - Bearing Stiffeners

MDX Steel Highway Girder Design Program, Version 6.5.4074

Load and Resistance Factor Design - Composite Girder

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Files: Span 2 - North Exterior.R1

R1.OUT

♀ Dimensions

Dimensions Bearing Stiffeners

Location	Width	Thickness
0.00	7.75	0.750
85.94	7.75	0.750

♀ Bearing Stiffeners

Bearing Stiffeners

Location from Left End of Web Sect. (ft)	Factored Reaction (k)	Allowable Column Force (k)	Ratio	Allowable Bearing Force (k)	Ratio
0.00	207.78	676.26	0.307	708.75	0.293
85.94	209.71	676.26	0.310	708.75	0.296

CAMBER CALCULATIONS - SPAN 1 VTRANS D37 IM 091-1(68)

GIRDER 1	S. ABUT.	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	N. ABUT.
STEEL DEFLECTION	0.00	0.20	0.37	0.51	0.60	0.63	0.60	0.51	0.37	0.20	0.00
CONC. DEFLECTION	0.00	0.71	1.33	1.83	2.14	2.25	2.14	1.83	1.34	0.71	0.00
S.D.L. DEFLECTION	0.00	0.08	0.14	0.20	0.23	0.24	0.23	0.20	0.14	0.08	0.00
VERTICAL CURVE	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
RESIDUAL CAMBER	0.00	0.33	0.62	0.85	1.00	1.05	1.00	0.85	0.62	0.33	0.00
Total Camber	0.00	1.30	2.47	3.38	3.96	4.17	3.97	3.38	2.47	1.30	0.00

GIRDERS 2-4	S. ABUT.	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	N. ABUT.
STEEL DEFLECTION	0.00	0.22	0.42	0.57	0.67	0.70	0.67	0.57	0.42	0.22	0.00
CONC. DEFLECTION	0.00	0.85	1.61	2.21	2.59	2.72	2.59	2.21	1.61	0.85	0.00
S.D.L. DEFLECTION	0.00	0.07	0.13	0.18	0.21	0.22	0.21	0.18	0.13	0.07	0.00
VERTICAL CURVE	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
RESIDUAL CAMBER	0.00	0.33	0.62	0.85	1.00	1.05	1.00	0.85	0.62	0.33	0.00
Total Camber	0.00	1.47	2.78	3.81	4.47	4.69	4.47	3.81	2.78	1.47	0.00

GIRDER 5	S. ABUT.	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	N. ABUT.
STEEL DEFLECTION	0.00	0.20	0.37	0.51	0.60	0.63	0.60	0.51	0.37	0.20	0.00
CONC. DEFLECTION	0.00	0.85	1.62	2.21	2.60	2.73	2.60	2.22	1.62	0.86	0.00
S.D.L. DEFLECTION	0.00	0.07	0.14	0.19	0.22	0.23	0.22	0.18	0.14	0.07	0.00
VERTICAL CURVE	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
RESIDUAL CAMBER	0.00	0.33	0.62	0.86	1.00	1.05	1.00	0.86	0.62	0.33	0.00
Total Camber	0.00	1.45	2.75	3.77	4.41	4.63	4.41	3.77	2.75	1.46	0.00

CAMBER CALCULATIONS - SPAN 2 VTRANS D37 IM 091-1(68)

GIRDER 1	S. ABUT.	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	N. ABUT.
STEEL DEFLECTION	0.00	0.22	0.41	0.56	0.66	0.69	0.66	0.56	0.41	0.22	0.00
CONC. DEFLECTION	0.00	0.74	1.40	1.92	2.25	2.37	2.26	1.93	1.41	0.75	0.00
S.D.L. DEFLECTION	0.00	0.08	0.16	0.22	0.26	0.27	0.26	0.22	0.16	0.08	0.00
VERTICAL CURVE	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
RESIDUAL CAMBER	0.00	0.34	0.64	0.87	1.03	1.07	1.03	0.87	0.64	0.34	0.00
Total Camber	0.00	1.38	2.61	3.57	4.19	4.40	4.20	3.58	2.62	1.38	0.00

GIRDERS 2-4	S. ABUT.	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	N. ABUT.
STEEL DEFLECTION	0.00	0.24	0.46	0.62	0.73	0.77	0.73	0.62	0.46	0.24	0.00
CONC. DEFLECTION	0.00	0.94	1.77	2.43	2.85	2.99	2.85	2.43	1.77	0.94	0.00
S.D.L. DEFLECTION	0.00	0.08	0.14	0.20	0.23	0.24	0.23	0.20	0.14	0.08	0.00
VERTICAL CURVE	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
RESIDUAL CAMBER	0.00	0.34	0.64	0.87	1.02	1.07	1.02	0.87	0.64	0.34	0.00
Total Camber	0.00	1.59	3.01	4.12	4.83	5.07	4.83	4.12	3.01	1.59	0.00

GIRDER 5	S. ABUT.	0.1L	0.2L	0.3L	0.4L	0.5L	0.6L	0.7L	0.8L	0.9L	N. ABUT.
STEEL DEFLECTION	0.00	0.22	0.41	0.56	0.66	0.69	0.66	0.56	0.41	0.22	0.00
CONC. DEFLECTION	0.00	0.97	1.84	2.52	2.96	3.11	2.96	2.53	1.85	0.98	0.00
S.D.L. DEFLECTION	0.00	0.08	0.15	0.20	0.24	0.25	0.24	0.20	0.15	0.08	0.00
VERTICAL CURVE	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
RESIDUAL CAMBER	0.00	0.34	0.64	0.87	1.02	1.07	1.02	0.87	0.64	0.34	0.00
Total Camber	0.00	1.60	3.04	4.16	4.87	5.12	4.88	4.17	3.05	1.61	0.00

BEARING DESIGN - LOAD TABULATION - Span 1 VTRANS - D37 IM 091-1(68)

References:
1) AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017 with Interims thru 2018

Unfactored Reaction @ Abutment

Girder	Total DL (k)	End Block DL (k)	LL (no impact) (k)
S. Exterior	49.58	3.73	52.70
Typ. Interior	55.00	4.68	68.14
N. Exterior	53.43	3.73	56.33

Steel Girder Extension

Steel Girder Area = 0.35 SF
Girder Extension = 2.00 ft
Girder Extension Weight = 0.347 k = 2.00 ft x 0.35 SF x 0.49 kcf

Concrete End Block Additional Load

Assume a concrete end block integral with the steel beam that is 3'-1" high x 12" wide x full width of abutment. Include additional 1.60 SF of deck concrete above centerline of bearing.

End Block Load = 0.70 klf
S. Exterior Trib. Width = 5.316 ft
Interior Trib. Width = 6.667 ft
N. Exterior Trib. Width = 5.705 ft

BEARING DESIGN - LOAD TABULATION - Span 1 VTRANS - D37 IM 091-1(68)

Breaking Force

Span Length =	83.95	ft
Number of Bearings =	10	
HL-93 Truck Weight =	72	kips
HL-93 Lane Load =	0.64	kips/ft

MPF

1 Lane =	1.20
2 Lanes =	1.00
3 Lanes =	0.85

Per (1) 3.6.4, the braking force shall be taken as the greater of 25% of the design truck or tandem, or 5% of the design truck or tandem plus lane load.

25% of Truck Load =	18.00	kips
5% of Truck + Lane load =	6.29	kips
Controlling Braking Force =	18.00	kips

Apply braking force in all lanes and apply appropriate multiple presence factors.

Roadway Width =	27.00 ft	
No. of Lanes =	2	
MPF =	1.00	
Braking Force =	3.60	kips

Bearing Design - Method A - Typical Interior - Span 1 VTRANS D37 IM 091-1(68)

References:
1) AASHTO LRFD Bridge Design, 8th Edition, with 2018 Errata

Note that bearings are designed for the Service Limit State.

DEAD LOAD REACTION:

V_{DL} = 55.00 k
$V_{DL\ END\ BLOCK}$ = 4.68 k

LIVE LOAD REACTION:

$V_{LL\ MAX}$ = 68.14 k
$P_{mSERVICE}$ = 127.82 k = 55.00 k + 4.68 k + 68.14 k

σ_s = average compressive stress due to total load from applicable service load combinations
σ_L = average compressive stress due to live load from the service limit state

Check	OK?
Shape Factor	OK
Cover = 0.25" min	OK
Cover < 70% h_{ri}	OK
σ_S < 1.25 ksi	OK
σ_S < 1.25*G*S	OK
Total Deflection	OK
Long Term Deflection	OK
Shear Deformation	OK
Slippage	OK
Stability	OK
Reinforcement	OK

Calculate Compressive Stress

<u>Bearing Properties</u>

Hardness :	60	Durometer	
G_{min} =	0.13 ksi		1) Table 14.7.6.2-1
G_{max} =	0.2 ksi		1) Table 14.7.6.2-1
Bearing Diam. =	13 in		
Bearing Area, A =	132.73 in²		
Bearing Total Thickness =	3 in	(1/4" increments)	
Laminate Thickness =	0.1196 in	(11 Gauge Laminate)	
Cover =	0.25 in		
Number of Internal Layers, n=	4		
Additional n =	1		1) 14.7.6.1
Design n =	5		

Number of Laminates = 5 = 4 + 1
Elastomer Layer Thickness, h_{ri} = 0.476 in = (3.00 in - (2 x 0.25 in) - (5 x 0.1196 in))/ 4.00

Bearing Design - Method A - Typical Interior - Span 1 — VTRANS D37 IM 091-1(68)

Check Shape Factor Requirments

Shape Factor, S = D / ($4h_n$)
S = 6.83
Elastomer Layer Thickness, h_n = 0.476 in
Design n = 5
S^2/n = 9.34 = (6.83 ^ 2)/ 5

Per (1) C14.7.6.1, S^2/n shall be limited to 16 when bearing is circular.

S^2/n = 9.34 < 16

OK

Calculate h_{rt}

Cover = 0.25 in
Lamination Thickenss = 0.4755 in
Design n = 5

h_{rt} = 2.878 in = (2 x 0.25 in) + (5 x 0.5 in)

Check Cover Requirments

Per (1) 14.7.5.1, cover elastomer layer shall be no thicker than 70% of the internal layers

70% of Internal Layer = 0.333 in = 0.48 in x 0.7
Cover = 0.250 in

OK

Calculate Compressive Stresses

For Steel Reinforced elastomeric bearings,

$\sigma_S \leq 1.25$ ksi, and $\sigma_S \leq 1.25*G*S$ — 1) 14.7.6.3.2-7,8

σ_S = ($Pm_{SERVICE}$)/A = 0.963 = 127.82 k/ 132.73 in²
Allowable Stress = 1.25 ksi

OK

σ_S = ($Pm_{SERVICE}$)/A = 0.963 ksi
1.25*G*S = 1.111 ksi = 1.25 x 0.13 ksi x 6.83

OK

Bearing Design - Method A - Typical Interior - Span 1 VTRANS D37 IM 091-1(68)

Calculate Compressive Deflection

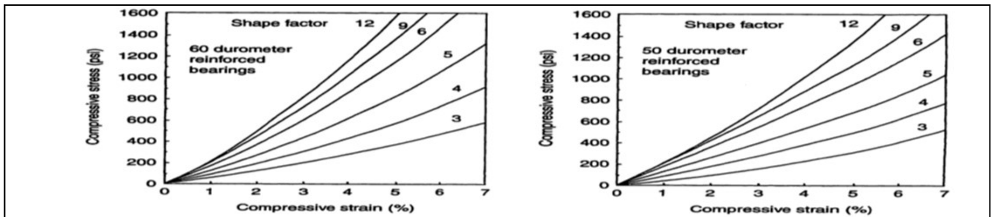


FIGURE 14.6.5.3.3-1 Load Deflection Behavior of Elastomeric Bearings

where ε DL= 0.025
where ε LL= 0.028

σ_{DL} = 0.450 ksi = (55 k + 4.6833333 k)/ 132.73 in²
σ_{LL} = 0.513 ksi = 68.14 k/ 132.73 in²

Calculate Instantaneous Live Load Deflection

Per (1) Eq. 14.7.5.3.6-1,

$\delta_L = \sum \varepsilon_{Li} h_{ri}$

h_{ri} = 0.4755 in
δ_L = 0.013 in = 0.028 0.4755 in

Calculate Initial Dead Load Deflection

Per (1) Eq. 14.7.5.3.6-2,

$\delta_d = \sum \varepsilon_{di} h_{ri}$

δ_d= 0.012 = 0.025 x 0.4755 in

Check Total Deflection

Per (1) 14.7.6.3.3,

The compressive deflection under instantaneous live load and initial dead load of a PEP or an internal layer of a steel-reinforced elastomeric bearing at the service limit state without impact shall not exceed 0.09hri, where h_{ri} is the thickness of a PEP, or the thickness of an internal layer of a steel-reinforced elastomeric bearing (in.).

δ_{all} = 0.043 in = 0.09 x 0.4755 in
$\delta_d + \delta_L$ = 0.025 in = 0.013 + + 0.012 in

OK

Bearing Design - Method A - Typical Interior - Span 1 VTRANS D37 IM 091-1(68)

Calculate Live Load Deflection + Long Term Creep

Per (1) Table 14.7.6.2-1,

a_{cr} = 0.35

$$\delta_{lt} = \delta_d + a_{cr}\delta_d \qquad (14.7.5.3.6\text{-}3)$$

$\delta_{LL + CREEP}$ = 0.016 0.012 in + (0.35 x 0.012 in)
δ_{max} = 0.125 in
OK

Check Shear

Deck Span = 85.00 ft = 1020.00 in Assume 12" deck extension at each CL Brg.

Check Temperature Effects

$\Delta_{temp} = L\alpha\, \Delta T$

Per (1) Table 3.12.2.1-1, T_{max} = 120 degress, and T_{min} = -30 degrees for cold climate and steel. Per (1) 14.7.5.3.2, use 65% of the design therman movement range.

T_{max} = 120 degrees
T_{min} = -30 degrees
65% of design thermal range = 97.50 = 0.65 x (120 - -30)

For construction purposes, say that design temperature range is 120 degrees
Design Temperature Range = 120 degrees

Max. Set Temp. = 90 = -30 + 120
Min. Set Temp. = 0 = 120 - 120

Where:

α (structural steel) = 0.0000065 /°F
temp. = 120 ° (120° for Construction Purposes)
L = 1020.00 in (length to be used for thermal movment)
$\Delta_{TEMP\ long}$ = 0.398 in = (1020.00 in/ 2) x 7E-06 x 120 °
$\Delta_{S\ long} = \Delta_{TEMP}$ = 0.398 in

Per (1) 14.7.6.3.4-1,

$h_{rt} > 2^*\Delta s$

Δs_{long} = 0.398 in
$2\Delta s_{long}$ = 0.796 in
h_{rt} = 2.878 in
OK

Check Slippage

Horizontal Force

Per (1) 14.6.3.1-2,

$$H_{bu} = GA\frac{\Delta_u}{h_{rt}} \qquad (14.6.3.1\text{-}2)$$

where:

G = shear modulus of the elastomer (ksi)
A = plan area of elastomeric element or bearing (in.^{2})
Δ_u = shear deformation from applicable strength and extreme event load combinations in Table 3.4.1-1 (in.)
h_{rt} = total elastomer thickness (in.)

G_{max} =	0.20 ksi				
A =	132.73 in²				
Δ_u =	0.40 in				
h_{rt} =	2.88 in				
H_{bu} =	3.67 k =	0.20 ksi x	132.73 in² x	(0.40 in/	2.88 in)

Calculate Minimum Vertical Force

Assume a coefficient of friction, μ, equal 0.20.
The force resisting sliding is equal to μ*Stringer DL Rxn.

μ =	0.20		
Stringer DL Rxn =	59.68 k =	55.00 k +	4.68 k
Resisting Force =	11.94 k =	0.2 x	59.68 k
H_{bu} =	3.67 k		

OK

Check Stability

Per (1) 14.7.6.3.6, the total thickness of pad shall not exceed L/3, W/3, or D/4

D/4 =	3.25 in =	13 in/	4

Calculate Bearing Thickness

Total Laminate Thickness =	1.902 in =	4 x	0.476 in
Total Cover Thickness =	0.500 in =	2 x	0.250 in
Total Reinforcement Thickness =	0.598 in =	5 x	#########
Total =	3.00 in		

OK

Bearing Design - Method A - Typical Interior - Span 1 VTRANS D37 IM 091-1(68)

Check Reinforcement

Per (1) 14.7.5.3.5 the minimum thickness of steel reinforcement, h_s, shall be,

h_S min = 0.0625 in

Per (1) 14.7.5.3.5 the thickness of the steel reinforcement at the strength limit state shall satisfy,

$h_s > (3.0 * h_{ri} * \sigma_S)/F_y$

F_y = 36 ksi

ΔF_{TH} = 24 ksi 1) Table 6.631.2.3-1, Category A

σ_s = 0.963 ksi

h_s = 0.038 in = (3 x 0.476 in x 0.963 ksi)/ 36 ksi

Per (1) 14.7.5.3.5 the thickness of the steel reinforcement at the fatigue limit state shall satisfy,

$h_s > (2.0 * h_{rmax} * \sigma_L)/\Delta F_{TH}$

σ_L = 0.513 ksi = 68.14 k/ 132.73 in²

h_s = 0.038 in = (2 x 0.476 in x 0.513 ksi)/ 24 ksi

Controlling h_s = 0.063

Use 11 gauge steel plate.

t_{plate} = 0.1196 in

OK

Bearing Design - Method A - N. Exterior - Span 1 VTRANS D37 IM 091-1(68)

References:
1) AASHTO LRFD Bridge Design, 8th Edition, with 2018 Errata

Note that bearings are designed for the Service Limit State.

DEAD LOAD REACTION:

V_{DL} = 53.43 k

$V_{DL\ END\ BLOCK}$ = 3.73 k

LIVE LOAD REACTION:

$V_{LL\ MAX}$ = 56.33 k

$P_{mSERVICE}$= 113.49 k = 53.43 k + 3.73 k + 56.33 k

σ_s = average compressive stress due to total load from applicable service load combinations

σ_L = average compressive stress due to live load from the service limit state

Check	OK?
Shape Factor	OK
Cover = 0.25" min	OK
Cover < 70% h_n	OK
σ_S < 1.25 ksi	OK
σ_S < 1.25*G*S	OK
Total Deflection	OK
Long Term Deflection	OK
Shear Deformation	OK
Slippage	OK
Stability	OK
Reinforcement	OK

Bearing Design - Method A - N. Exterior - Span 1 VTRANS D37 IM 091-1(68)

Calculate Compressive Stress

Bearing Properties

Hardness :	60	Durometer				
G_{min} =	0.13 ksi	1) Table 14.7.6.2-1				
G_{max} =	0.2 ksi	1) Table 14.7.6.2-1				
Bearing Diam. =	13 in					
Bearing Area, A =	132.73 in²					
Bearing Total Thickness =	3 in	(1/4" increments)				
Laminate Thickness =	0.1196 in	(11 Gauge Laminate)				
Cover =	0.25 in					
Number of Internal Layers, n=	4					
Additional n =	1	1) 14.7.6.1				
Design n =	5					
Number of Laminates =	5 =	4 +	1			
Elastomer Layer Thickness, h_{ri} =	0.476 in =	(3.00 in -	(2 x	0.25 in) -	(5 x 0.1196 in))/	4.00

Check Shape Factor Requirments

Shape Factor, S =	D / (4h_{ri})			
S =	6.83			
Elastomer Layer Thickness, h_{ri}=	0.476 in			
Design n =	5			
S^2/n =	9.34 =	(6.83 ^	2)/	5

Per (1) C14.7.6.1, S^2/n shall be limited to 16 when bearing is circular.

S^2/n =	9.34	<	16
	OK		

Bearing Design - Method A - N. Exterior - Span 1 VTRANS D37 IM 091-1(68)

Calculate h_{rt}

Cover =	0.25 in					
Lamination Thickenss =	0.4755 in					
Design n =	5					
h_{rt} =	2.878 in =	(2 x	0.25 in) +	(5 x	0.48 in)	

Check Cover Requirments

Per (1) 14.7.5.1, cover elastomer layer shall be no thicker than 70% of the internal layers

70% of Internal Layer =	0.333 in =	0.48 in x	0.7
Cover =	0.250 in		

OK

Calculate Compressive Stresses

For Steel Reinforced elastomeric bearings,

$\sigma_S \leq 1.25$ ksi, and $\sigma_S \leq 1.25^*G^*S$ 1) 14.7.6.3.2-7,8

σ_S = $(Pm_{SERVICE})/A$ =	0.855 =	113.49 k/	132.73 in²
Allowable Stress =	1.25 ksi		

OK

σ_S = $(Pm_{SERVICE})/A$ =	0.855 ksi			
1.25*G*S =	1.111 ksi =	1.25 x	0.13 ksi x	6.83

OK

Bearing Design - Method A - N. Exterior - Span 1 VTRANS D37 IM 091-1(68)

Calculate Compressive Deflection

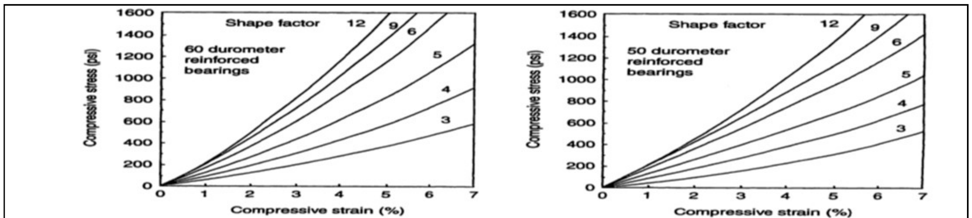


FIGURE 14.6.5.3.3-1 Load Deflection Behavior of Elastomeric Bearings

where ε DL= 0.028
where ε LL= 0.028

σ_{DL} = 0.431 ksi = (53.43 k + 3.7346922 k)/ 132.73 in²
σ_{LL} = 0.424 ksi = 56.33 k/ 132.73 in²

Calculate Instantaneous Live Load Deflection

Per (1) Eq. 14.7.5.3.6-1,

$\delta_L = \sum \varepsilon_{Li} h_{ri}$

h_{ri} = 0.4755 in

δ_L = 0.013 in = 0.028 0.4755 in

Calculate Initial Dead Load Deflection

Per (1) Eq. 14.7.5.3.6-2,

$\delta_d = \sum \varepsilon_{di} h_{ri}$

δ_d= 0.013 = 0.028 x 0.4755 in

Check Total Deflection

Per (1) 14.7.6.3.3,

The compressive deflection under instantaneous live load and initial dead load of a PEP or an internal layer of a steel-reinforced elastomeric bearing at the service limit state without impact shall not exceed 0.09hri, where h_{ri} is the thickness of a PEP, or the thickness of an internal layer of a steel-reinforced elastomeric bearing (in.).

δ_{all} = 0.043 in = 0.09 x 0.4755 in
$\delta_d + \delta_L$ = 0.027 in = 0.013 + + 0.013 in

OK

Bearing Design - Method A - N. Exterior - Span 1 VTRANS D37 IM 091-1(68)

Calculate Live Load Deflection + Long Term Creep

Per (1) Table 14.7.6.2-1,

a_{cr} = 0.35

$$\delta_{lt} = \delta_d + a_{cr}\delta_d \qquad (14.7.5.3.6\text{-}3)$$

$\delta_{LL\ +\ CREEP}$ = 0.018 0.013 in + (0.35 x 0.013 in)
δ_{max} = 0.125 in

OK

Check Shear

Assume 12" deck extension beyond CL Brg.

Deck Span = 85.00 ft = 1020.00 in

Check Temperature Effects

Δ_{temp} = Lα ΔT

Per (1) Table 3.12.2.1-1, T_{max} = 120 degress, and T_{min} = -30 degrees for cold climate and steel. Per (1) 14.7.5.3.2, use 65% of the design therman movement range.

T_{max} = 120 degrees
T_{min} = -30 degrees
65% of design thermal range = 97.50 = 0.65 x (120 - -30)

For construction purposes, say that design temperature range is 120 degress
Design Temperature Range = 120 degrees

Max. Set Temp. = 90 = -30 + 120
Min. Set Temp. = 0 = 120 - 120

Where:

α (structural steel) = 0.0000065 /°F
temp. = 120 ° (100° for Construction Purposes)
L = 1020.00 in (length to be used for thermal movment)
$\Delta_{TEMP\ long}$ = 0.398 in = (1020.00 in/ 2) x 6.5E-06 x 120 °
$\Delta_{S\ long} = \Delta_{TEMP}$ = 0.398 in

Per (1) 14.7.6.3.4-1,

$h_{rt} > 2^*\Delta s$

Δs_{long} = 0.398 in
$2\Delta s_{long}$ = 0.796 in
h_{rt} = 2.878 in

OK

Check Slippage

Horizontal Force

Per (1) 14.6.3.1-2,

$$H_{bu} = GA\frac{\Delta_u}{h_{rt}} \qquad (14.6.3.1\text{-}2)$$

where:

G = shear modulus of the elastomer (ksi)
A = plan area of elastomeric element or bearing (in.^{2})
Δ_u = shear deformation from applicable strength and extreme event load combinations in Table 3.4.1-1 (in.)
h_{rt} = total elastomer thickness (in.)

G_{max} =	0.20 ksi			
A =	132.73 in²			
Δ_u =	0.40 in			
h_{rt} =	2.88 in			
H_{bu} =	3.67 k =	0.20 ksi x	132.73 in² x	(0.40 in/ 2.88 in)

Calculate Minimum Vertical Force

Assume a coefficient of friction, μ, equal 0.20.
The force resisting sliding is equal to μ*Stringer DL Rxn.

μ =	0.20		
Stringer DL Rxn =	57.16 k =	53.43 k +	3.73 k
Resisting Force =	11.43 k =	0.2 x	57.16 k
H_{bu} =	3.67 k		

OK

Check Stability

Per (1) 14.7.6.3.6, the total thickness of pad shall not exceed L/3, W/3, or D/4

D/4 =	3.25 in =	13 in/	4

Calculate Bearing Thickness

Total Laminate Thickness =	1.902 in =	4 x	0.476 in
Total Cover Thickness =	0.500 in =	2 x	0.250 in
Total Reinforcement Thickness =	0.598 in =	5 x	0.11960 in
Total =	3.00 in		

OK

Check Reinforcement

Per (1) 14.7.5.3.5 the minimum thickness of steel reinforcement, h_s, shall be,

h_s min = 0.0625 in

Per (1) 14.7.5.3.5 the thickness of the steel reinforcement at the strength limit state shall satisfy,

$h_s > (3.0 * h_n * \sigma_s)/F_y$

F_y = 36 ksi

ΔF_{TH} = 24 ksi 1) Table 6.631.2.3-1, Category A

σ_s = 0.855 ksi

h_s = 0.034 in = (3 x 0.476 in x 0.855 ksi)/ 36 ksi

Per (1) 14.7.5.3.5 the thickness of the steel reinforcement at the fatigue limit state shall satisfy,

$h_s > (2.0 * h_{rmax} * \sigma_L)/\Delta F_{TH}$

σ_L = 0.424 ksi = 56.33 k/ 132.73 in²

h_s = 0.034 in = (2 x 0.476 in x 0.424 ksi)/ 24 ksi

Controlling h_s = 0.063

Use 11 gauge steel plate.

t_{plate} = 0.1196 in

OK

BEARING DESIGN - LOAD TABULATION - Span 2 VTRANS - D37 IM 091-1(68)

References:
1) AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017 with Interims thru 2018

Unfactored Reaction @ Abutment

Girder	Total DL (k)	End Block DL (k)	LL (no impact) (k)
S. Exterior	48.58	3.53	57.76
Typ. Interior	56.30	4.68	75.18
N. Exterior	54.79	3.53	64.18

Steel Girder Extension

Steel Girder Area = 0.35 SF
Girder Extension = 2.00 ft
Girder Extension Weight = 0.347 k = 2.00 ft x 0.35 SF x 0.49 kcf

Concrete End Block Additional Load

Assume a concrete end block integral with the steel beam that is 3'-1" high x 12" wide x full width of abutment. Include additional 1.60 SF of deck concrete above centerline of bearing.

End Block Load = 0.70 klf
S. Exterior Trib. Width = 5.019 ft
Interior Trib. Width = 6.667 ft
N. Exterior Trib. Width = 5.867 ft

BEARING DESIGN - LOAD TABULATION - Span 2 VTRANS - D37 IM 091-1(68)

Breaking Force

Span Length =	85.94	ft
Number of Bearings =	12	
HL-93 Truck Weight =	72	kips
HL-93 Lane Load =	0.64	kips/ft

MPF

1 Lane =	1.20
2 Lanes =	1.00
3 Lanes =	0.85

Per (1) 3.6.4, the braking force shall be taken as the greater of 25% of the design truck or tandem, or 5% of the design truck or tandem plus lane load.

25% of Truck Load =	18.00	kips
5% of Truck + Lane load =	6.35	kips
Controlling Braking Force =	18.00	kips

Apply braking force in all lanes and apply appropriate multiple presence factors.

No. of Lanes =	2	
MPF =	1.00	
Braking Force =	3.00	kips

Bearing Design - Method A - Typical Interior - Span 2 VTRANS D37 IM 091-1(68)

References:
1) AASHTO LRFD Bridge Design, 8th Edition, with 2018 Errata

Note that bearings are designed for the Service Limit State.

DEAD LOAD REACTION:

V_{DL} = 56.30 k

$V_{DL\ END\ BLOCK}$ = 4.68 k

LIVE LOAD REACTION:

$V_{LL\ MAX}$ = 75.18 k

$P_{mSERVICE}$ = 136.16 k = 56.30 k + 4.68 k + 75.18 k

σ_s = average compressive stress due to total load from applicable service load combinations

σ_L = average compressive stress due to live load from the service limit state

Check	OK?
Shape Factor	OK
Cover = 0.25" min	OK
Cover < 70% h_{ri}	OK
σ_S < 1.25 ksi	OK
σ_S < 1.25*G*S	OK
Total Deflection	OK
Long Term Deflection	OK
Shear Deformation	OK
Slippage	OK
Stability	OK
Reinforcement	OK

Calculate Compressive Stress

Bearing Properties

Hardness :	60	Durometer	
G_{min} =	0.13 ksi		1) Table 14.7.6.2-1
G_{max} =	0.2 ksi		1) Table 14.7.6.2-1
Bearing Diam. =	13 in		
Bearing Area, A =	132.73 in²		
Bearing Total Thickness =	3 in	(1/4" increments)	
Laminate Thickness =	0.1196 in	(11 Gauge Laminate)	
Cover =	0.25 in		
Number of Internal Layers, n=	4		
Additional n =	1		1) 14.7.6.1
Design n =	5		

Number of Laminates = 5 = 4 + 1

Elastomer Layer Thickness, h_{ri} = 0.476 in = (3.00 in - (2 x 0.25 in) - (5 x 0.1196 in))/ 4.00

Bearing Design - Method A - Typical Interior - Span 2 VTRANS D37 IM 091-1(68)

Check Shape Factor Requirments

Shape Factor, S = D / ($4h_n$)
S = 6.83
Elastomer Layer Thickness, h_n = 0.476 in
Design n = 5
S^2/n = 9.34 = (6.83 ^ 2)/ 5

Per (1) C14.7.6.1, S^2/n shall be limited to 16 when bearing is circular.
S^2/n = 9.34 < 16
OK

Calculate h_{rt}

Cover = 0.25 in
Lamination Thickenss = 0.4755 in
Design n = 5

h_{rt} = 2.878 in = (2 x 0.25 in) + (5 x 0.48 in)

Check Cover Requirments

Per (1) 14.7.5.1, cover elastomer layer shall be no thicker than 70% of the internal layers
70% of Internal Layer = 0.333 in = 0.48 in x 0.7
Cover = 0.250 in
OK

Calculate Compressive Stresses

For Steel Reinforced elastomeric bearings,

$\sigma_S \leq 1.25$ ksi, and $\sigma_S \leq 1.25*G*S$ 1) 14.7.6.3.2-7,8

σ_S = ($Pm_{SERVICE}$)/A = 1.026 = 136.16 k/ 132.73 in²
Allowable Stress = 1.25 ksi
OK

σ_S = ($Pm_{SERVICE}$)/A = 1.026 ksi
1.25*G*S = 1.111 ksi = 1.25 x 0.13 ksi x 6.83
OK

Calculate Compressive Deflection

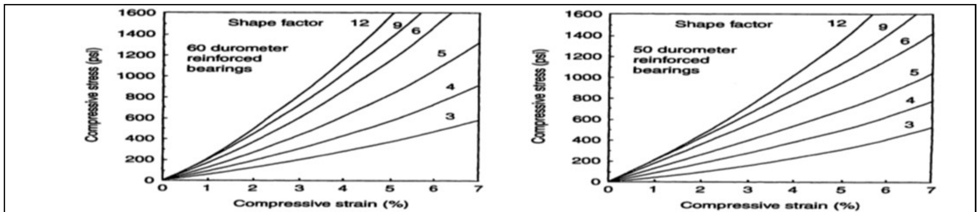


FIGURE 14.6.5.3.3-1 Load Deflection Behavior of Elastomeric Bearings

where ε DL= 0.026
where ε LL= 0.030

σ_{DL} = 0.459 ksi = (56.3 k + 4.6833333 k)/ 132.73 in²
σ_{LL} = 0.566 ksi = 75.18 k/ 132.73 in²

Calculate Instantaneous Live Load Deflection

Per (1) Eq. 14.7.5.3.6-1,

$\delta_L = \sum \varepsilon_{Li} h_{ri}$

h_{ri} = 0.4755 in
δ_L = 0.014 in = 0.030 0.4755 in

Calculate Initial Dead Load Deflection

Per (1) Eq. 14.7.5.3.6-2,

$\delta_d = \sum \varepsilon_{di} h_{ri}$

δ_d = 0.012 = 0.026 x 0.4755 in

Check Total Deflection

Per (1) 14.7.6.3.3,

The compressive deflection under instantaneous live load and initial dead load of a PEP or an internal layer of a steel-reinforced elastomeric bearing at the service limit state without impact shall not exceed 0.09hri, where h_{ri} is the thickness of a PEP, or the thickness of an internal layer of a steel-reinforced elastomeric bearing (in.).

δ_{all} = 0.043 in = 0.09 x 0.4755 in
$\delta_d + \delta_L$ = 0.027 in = 0.014 + + 0.012 in

OK

Calculate Live Load Deflection + Long Term Creep

Per (1) Table 14.7.6.2-1,

a_{cr} = 0.35

$$\delta_{lt} = \delta_d + a_{cr}\delta_d \qquad (14.7.5.3.6\text{-}3)$$

$\delta_{LL + CREEP}$ = 0.017 0.012 in + (0.35 x 0.012 in)
δ_{max} = 0.125 in

OK

Check Shear

Acute corner to acute corner length

Deck Span = 103.71 ft = 1244.50 in

Check Temperature Effects

Δ_{temp} = Lα ΔT

Per (1) Table 3.12.2.1-1, T_{max} = 120 degress, and T_{min} = -30 degrees for cold climate and steel. Per (1) 14.7.5.3.2, use 65% of the design therman movement range.

T_{max} =	120	degrees			
T_{min} =	-30	degrees			
65% of design thermal range =	97.50 =		0.65 x	(120 -	-30)

For construction purposes, say that design temperature range is 120 degress

Design Temperature Range =	120	degrees		
Max. Set Temp. =	90 =		-30 +	120
Min. Set Temp. =	0 =		120 -	120

Where:

α (structural steel) =	0.0000065	/°F				
temp. =	120	°	(120° for Construction Purposes)			
L =	1244.50 in		(length to be used for thermal movment)			
$\Delta_{TEMP\ long}$ =	0.485 in =	(1244.50 in/	2) x	6.5E-06 x	120 °	
$\Delta_{S\ long}$ = Δ_{TEMP} =	0.485 in					

Per (1) 14.7.6.3.4-1,

$h_{rt} > 2^*\Delta s$

Δs_{long} =	0.485	in
$2\Delta s_{long}$ =	0.971 in	
h_{rt} =	2.878 in	

OK

Check Slippage

Horizontal Force

Per (1) 14.6.3.1-2,

$$H_{bu} = GA\frac{\Delta_u}{h_{rt}} \quad (14.6.3.1\text{-}2)$$

where:

G = shear modulus of the elastomer (ksi)
A = plan area of elastomeric element or bearing (in.^{2})
Δ_u = shear deformation from applicable strength and extreme event load combinations in Table 3.4.1-1 (in.)
h_{rt} = total elastomer thickness (in.)

G_{max} =	0.20 ksi			
A =	132.73 in²			
Δ_u =	0.49 in			
h_{rt} =	2.88 in			
H_{bu} =	4.48 k =	0.20 ksi x	132.73 in² x	(0.49 in/ 2.88 in)

Calculate Minimum Vertical Force

Assume a coefficient of friction, μ, equal 0.20.
The force resisting sliding is equal to μ*Stringer DL Rxn.

μ =	0.20		
Stringer DL Rxn =	60.98 k =	56.30 k +	4.68 k
Resisting Force =	12.20 k =	0.2 x	60.98 k
H_{bu} =	4.48 k		

OK

Check Stability

Per (1) 14.7.6.3.6, the total thickness of pad shall not exceed L/3, W/3, or D/4

D/4 = 3.25 in = 13 in/ 4

Calculate Bearing Thickness

Total Laminate Thickness =	1.902 in =	4 x	0.476 in
Total Cover Thickness =	0.500 in =	2 x	0.250 in
Total Reinforcement Thickness =	0.598 in =	5 x	0.11960 in
Total =	3.00 in		

OK

Bearing Design - Method A - Typical Interior - Span 2 VTRANS D37 IM 091-1(68)

Check Reinforcement

Per (1) 14.7.5.3.5 the minimum thickness of steel reinforcement, h_s, shall be,

h_S min = 0.0625 in

Per (1) 14.7.5.3.5 the thickness of the steel reinforcement at the strength limit state shall satisfy,

$h_s > (3.0 * h_{ri} * \sigma_S)/F_y$

F_y = 36 ksi

ΔF_{TH} = 24 ksi 1) Table 6.631.2.3-1, Category A

σ_s = 1.026 ksi

h_s = 0.041 in = (3 x 0.476 in x 1.026 ksi)/ 36 ksi

Per (1) 14.7.5.3.5 the thickness of the steel reinforcement at the fatigue limit state shall satisfy,

$h_s > (2.0 * h_{rmax} * \sigma_L)/\Delta F_{TH}$

σ_L = 0.566 ksi = 75.18 k/ 132.73 in²

h_s = 0.041 in = (2 x 0.476 in x 0.566 ksi)/ 24 ksi

Controlling h_s = 0.063

Use 11 gauge steel plate.

t_{plate} = 0.1196 in

OK

Bearing Design - Method A - N. Exterior - Span 2 VTRANS D37 IM 091-1(68)

References:
1) AASHTO LRFD Bridge Design, 8th Edition, with 2018 Errata

Note that bearings are designed for the Service Limit State.

DEAD LOAD REACTION:

V_{DL} = 54.79 k
$V_{DL\ END\ BLOCK}$ = 3.53 k

LIVE LOAD REACTION:

$V_{LL\ MAX}$ = 64.18 k
$P_{mSERVICE}$= 122.50 k = 54.79 k + 3.53 k + 64.18 k

σ_s = average compressive stress due to total load from applicable service load combinations
σ_L = average compressive stress due to live load from the service limit state

Check	OK?
Shape Factor	OK
Cover = 0.25" min	OK
Cover < 70% h_{ri}	OK
σ_S < 1.25 ksi	OK
σ_S < 1.25*G*S	OK
Total Deflection	OK
Long Term Deflection	OK
Shear Deformation	OK
Slippage	OK
Stability	OK
Reinforcement	OK

Calculate Compressive Stress

Bearing Properties

Hardness : 60 Durometer
G_{min} = 0.13 ksi 1) Table 14.7.6.2-1
G_{max} = 0.2 ksi 1) Table 14.7.6.2-1
Bearing Diam. = 13 in
Bearing Area, A = 132.73 in²
Bearing Total Thickness = 3 in (1/4" increments)
Laminate Thickness = 0.1196 in (11 Gauge Laminate)
Cover = 0.25 in
Number of Internal Layers, n= 4
Additional n = 1
Design n = 5
Number of Laminates = 5 = 4 + 1
Elastomer Layer Thickness, h_{ri} = 0.476 in = (3.00 in - (2 x 0.25 in) - (5 x 0.1196 in))/ 4.00

Check Shape Factor Requirments

Shape Factor, S = D / ($4h_{ri}$)
S = 6.83
Elastomer Layer Thickness, h_{ri}= 0.476 in
Design n = 5
S^2/n = 9.34 = (6.83 ^ 2)/ 5
Per (1) C14.7.6.1, S^2/n shall be limited to 16 when bearing is circular.
S^2/n = 9.34 < 16
OK

Bearing Design - Method A - N. Exterior - Span 2 VTRANS D37 IM 091-1(68)

Calculate h_{rt}

Cover =	0.25 in					
Lamination Thickenss =	0.4755 in					
Design n =	5					
h_{rt} =	2.878 in =	(2 x	0.25 in) +	(5 x	0.4755 in)	

Check Cover Requirments

Per (1) 14.7.5.1, cover elastomer layer shall be no thicker than 70% of the internal layers

70% of Internal Layer =	0.333 in =	0.48 in x	0.7
Cover =	0.250 in		

OK

Calculate Compressive Stresses

For Steel Reinforced elastomeric bearings,

$\sigma_S \leq 1.25$ ksi, and $\sigma_S \leq 1.25*G*S$ 1) 14.7.6.3.2-7,8

$\sigma_S = (Pm_{SERVICE})/A$ =	0.923 =	122.50 k/	132.73 in²
Allowable Stress =	1.25 ksi		

OK

$\sigma_S = (Pm_{SERVICE})/A$ =	0.923 ksi			
1.25*G*S =	1.111 ksi =	1.25 x	0.13 ksi x	6.83

OK

Calculate Compressive Deflection

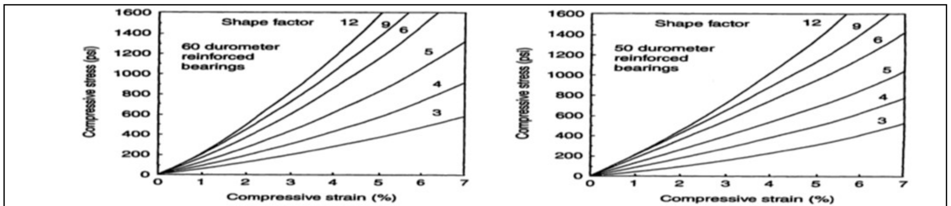


FIGURE 14.6.5.3.3-1 Load Deflection Behavior of Elastomeric Bearings

where ε DL= 0.025
where ε LL= 0.025

σ_{DL} = 0.439 ksi = (54.79 k + 3.525672 k)/ 132.73 in²
σ_{LL} = 0.484 ksi = 64.18 k/ 132.73 in²

Calculate Instantaneous Live Load Deflection

Per (1) Eq. 14.7.5.3.6-1,

$\delta_L = \sum \varepsilon_{Li} h_{ri}$

h_{ri} = 0.4755 in
δ_L = 0.012 in = 0.025 0.4755 in

Calculate Initial Dead Load Deflection

Per (1) Eq. 14.7.5.3.6-2,

$\delta_d = \sum \varepsilon_{di} h_{ri}$

δ_d= 0.012 = 0.025 x 0.4755 in

Check Total Deflection

Per (1) 14.7.6.3.3,

The compressive deflection under instantaneous live load and initial dead load of a PEP or an internal layer of a steel-reinforced elastomeric bearing at the service limit state without impact shall not exceed 0.09hri, where h_{ri} is the thickness of a PEP, or the thickness of an internal layer of a steel-reinforced elastomeric bearing (in.).

δ_{all} = 0.043 in = 0.09 x 0.4755 in
$\delta_d + \delta_L$ = 0.024 in = 0.012 + + 0.012 in

OK

Calculate Live Load Deflection + Long Term Creep

Per (1) Table 14.7.6.2-1,

a_{cr} = 0.35

$$\delta_{lt} = \delta_d + a_{cr}\delta_d \qquad (14.7.5.3.6\text{-}3)$$

$\delta_{LL + CREEP}$ = 0.016 0.012 in + (0.35 x 0.012 in)

δ_{max} = 0.125 in

OK

Check Shear

Acute corner to acute corner length

Deck Span = 103.71 ft = 1244.50 in

Check Temperature Effects

$\Delta_{temp} = L\alpha\ \Delta T$

Per (1) Table 3.12.2.1-1, T_{max} = 120 degress, and T_{min} = -30 degrees for cold climate and steel. Per (1) 14.7.5.3.2, use 65% of the design therman movement range.

T_{max} = 120 degrees
T_{min} = -30 degrees
65% of design thermal range = 97.50 = 0.65 x (120 - -30)

For construction purposes, say that design temperature range is 120 degress
Design Temperature Range = 120 degrees

Max. Set Temp. = 90 = -30 + 120
Min. Set Temp. = 0 = 120 - 120

Where:

α (structural steel) = 0.0000065 /°F
temp. = 120 ° (120° for Construction)
L = 1244.50 in (length to be used for thermal movment)
$\Delta_{TEMP\ long}$ = 0.485 in = (1244.50 in/ 2) x 6.5E-06 x 120 °
$\Delta_{S\ long} = \Delta_{TEMP}$ = 0.485 in

Per (1) 14.7.6.3.4-1,

$h_{rt} > 2^*\Delta s$

$\Delta s_{\ long}$ = 0.485 in
$2\Delta s_{\ long}$ = 0.971 in
h_{rt} = 2.878 in

OK

Check Slippage

Horizontal Force

Per (1) 14.6.3.1-2,

$$H_{bu} = GA\frac{\Delta_u}{h_{rt}} \quad (14.6.3.1\text{-}2)$$

where:

G = shear modulus of the elastomer (ksi)
A = plan area of elastomeric element or bearing (in.²)
Δ_u = shear deformation from applicable strength and extreme event load combinations in Table 3.4.1-1 (in.)
h_{rt} = total elastomer thickness (in.)

G_{max} =	0.20 ksi				
A =	132.73 in²				
Δ_u =	0.49 in				
h_{rt} =	2.88 in				
H_{bu} =	4.48 k =	0.20 ksi x	132.73 in² x	(0.49 in/	2.88 in)

Calculate Minimum Vertical Force

Assume a coefficient of friction, μ, equal 0.20.
The force resisting sliding is equal to μ*Stringer DL Rxn.

μ =	0.20		
Stringer DL Rxn =	58.32 k =	54.79 k +	3.53 k
Resisting Force =	11.66 k =	0.2 x	58.32 k
H_{bu} =	4.48 k		

OK

Check Stability

Per (1) 14.7.6.3.6, the total thickness of pad shall not exceed L/3, W/3, or D/4

D/4 = 3.25 in = 13 in/ 4

Calculate Bearing Thickness

Total Laminate Thickness =	1.902 in =	4 x	0.476 in
Total Cover Thickness =	0.500 in =	2 x	0.250 in
Total Reinforcement Thickness =	0.598 in =	5 x	0.11960 in
Total =	3.00 in		

OK

Check Reinforcement

Per (1) 14.7.5.3.5 the minimum thickness of steel reinforcement, h_s, shall be,

h_S min = 0.0625 in

Per (1) 14.7.5.3.5 the thickness of the steel reinforcement at the strength limit state shall satisfy,

$h_s > (3.0 * h_n * \sigma_S)/F_y$

F_y = 36 ksi

ΔF_{TH} = 24 ksi 1) Table 6.631.2.3-1, Category A

σ_s = 0.923 ksi

h_s = 0.037 in = (3 x 0.476 in x 0.923 ksi)/ 36 ksi

Per (1) 14.7.5.3.5 the thickness of the steel reinforcement at the fatigue limit state shall satisfy,

$h_s > (2.0 * h_{rmax} * \sigma_L)/\Delta F_{TH}$

σ_L = 0.484 ksi = 64.18 k/ 132.73 in²

h_s = 0.037 in = (2 x 0.476 in x 0.484 ksi)/ 24 ksi

Controlling h_s = 0.063

Use 11 gauge steel plate.

t_{plate} = 0.1196 in

OK

FLANGE TO WELD CALCULATION - SPAN 1 VTRANS D37 IM 091-1(68)

References:
1) AASHTO LRFD Bridge Design, 8th Edition, with 2018 Errata

R_r = $0.6\Phi_{e2}F_{exx}$ *(1) 6.13.3.2.4b-1*
R_r = 33.60

Φ_{e2} = 0.8 *(1) 6.5.4.2*
F_{exx} = 70 ksi

$$V_{ALLOWABLE} = \boxed{\frac{2R_r \frac{t_{WELD}}{\sqrt{2}} I}{Q}}$$

Span 1 controls for shear, forces taken from MDX. Max shear is summation of factored max shears for DC, DW and LL.

Span	Girder	Moment of Inertia	Q_{TOP}	Q_{BOT}	Max Shear	$t_{WELD\ TOP}$	$t_{WELD\ BOTTOM}$	$V_{ALLOWABLE\ TOP}$	$V_{ALLOWABLE\ BOTTOM}$	CHECK
1	G1	30,289	805	597	206	5/16	5/16	559	754	OK
1	G2-G4	32,057	853	623	231	5/16	5/16	558	764	OK
1	G5	32,057	853	623	187	5/16	5/16	558	764	OK

FLANGE TO WELD CALCULATION - SPAN 2 VTRANS D37 IM 091-1(68)

References:
1) AASHTO LRFD Bridge Design, 8th Edition, with 2018 Errata

$R_r = 0.6\Phi_{e2}F_{exx}$ *(1) 6.13.3.2.4b-1*

$R_r = 33.60$

$\Phi_{e2} = 0.8$ *(1) 6.5.4.2*

$F_{exx} = 70$ ksi

$$V_{ALLOWABLE} = \boxed{\frac{2R_r \frac{t_{WELD}}{\sqrt{2}} I}{Q}}$$

Span 1 controls for shear, forces taken from MDX. Max shear is summation of factored max shears for DC, DW and LL.

Span	Girder	Moment of Inertia	Q_{TOP}	Q_{BOT}	Max Shear	$t_{WELD\ TOP}$	$t_{WELD\ BOTTOM}$	$V_{ALLOWABLE\ TOP}$	$V_{ALLOWABLE\ BOTTOM}$	CHECK
1	G1	30,289	805	597	210	5/16	5/16	559	754	OK
1	G2-G4	32,057	853	623	233	5/16	5/16	558	764	OK
1	G5	32,057	853	623	188	5/16	5/16	558	764	OK

FINAL DESIGN - INTERIOR END DIAPHRAGM DESIGN - SPAN 1 VTRANS D37 IM 091-1(68)

References:
1) AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017 with Interims thru 2018
2) VTrans Structures Design Manual, 5th Edition

Skew =	10.47	°
Dia. Sp. =	6.67	ft
Length of Dia., L =	6.78	ft

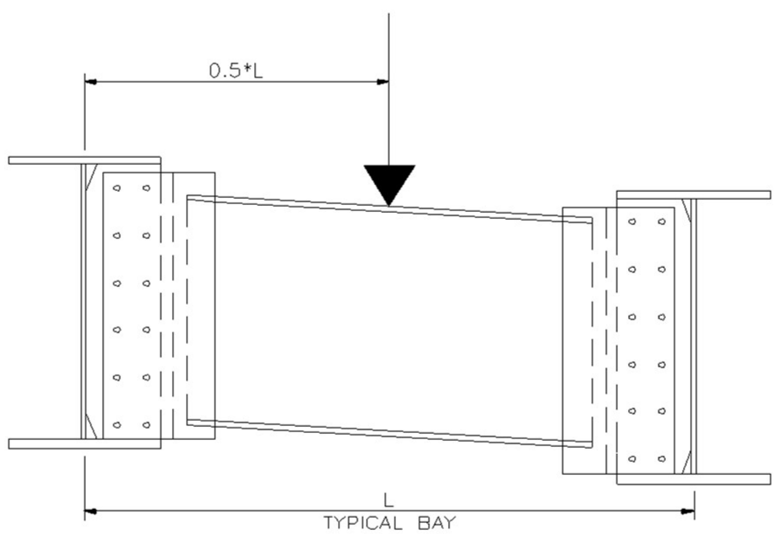
W Shape Self Weight: W27X84

Self Wt =	84	lb/ft
L =	6.78	ft
M_{DL} =	0.48	k-ft
V_{DL} =	0.28	kips

Live Load:

HL-93 wheel load is applied as shown below.

P =	16	kips	
Dyn. Load allowance, IM =	1.33		(1) Table 3.6.2.1-1
P' = P x IM =	21.28		
V_{LL} = 0.5*P x IM =	10.64	kips	
M_{LL+I} =	36.07	Kips-ft	



FINAL DESIGN - INTERIOR END DIAPHRAGM DESIGN - SPAN 1 VTRANS D37 IM 091-1(68)

Determine Governing Loads: (1) Table 3.4.1-1

For Strength I, γ_p (DL) = 1.25
Live Load Factor = 1.75

Dead + Live Load:

M_u = 63.72 k-ft = 1.25 x 0.48 k-ft + 1.75 x 36.07 k-ft
V_u = 18.98 kips = 1.25 x 0.28 k + 1.75 x 10.64 k

Mu =	63.72	ft kips
Vu =	18.98	kips

STEEL W BEAM

Flexure: (1) 6.12.2.2.5-1

L_b = 6.78 ft
Fy = 50 ksi
E = 29000 ksi
D = 26.7 in
D_c = 13.4 in^3
t_f = 0.64 in
b_f = 10.00 in
t_w = 0.46 in
$b_f/2*t_f$ = 7.81 = 10.00 in/ (2 x 0.64 in)

For a W-Shaped beam that has a compression flange braced discretetly, the following shall be satisfied:

$$f_{bu} + \frac{1}{3} f_\ell \le \phi_f F_{nc} \qquad (6.10.8.1.1\text{-}1)$$

where:

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2
f_{bu} = flange stress calculated without consideration of flange lateral bending determined as specified in Article 6.10.1.6 (ksi)
f_ℓ = flange lateral bending stress determined as specified in Article 6.10.1.6 (ksi)
F_{nc} = nominal flexural resistance of the flange determined as specified in Article 6.10.8.2 (ksi)

Check Local Buckling Resistance

Calculate λpf =

Lambda PF =	9.15	0.38 x	(29000/	50.00)^ 0.5
Lambda F =	7.81			

Since Lambda f < Lambda PF, Fyc for Local Buckling = Fy

Check Lateral Torsional Buckling (LTB)

Lb = 6.78 ft

L_p = limiting unbraced length to achieve the nominal flexural resistance of $R_b R_h F_{yc}$ under uniform bending (in.)

$$= 1.0 r_t \sqrt{\frac{E}{F_{yc}}} \qquad (6.10.8.2.3\text{-}4)$$

r_t = effective radius of gyration for lateral torsional buckling (in.)

$$= \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}} \qquad (6.10.8.2.3\text{-}9)$$

A =	1.32 =	1 +	13.35 in x / 3 x	0.46 in / 10.00 in x	0.64	
B = 12*A =	15.84 =	12 x	1.32			
C = $B^{0.5}$ =	3.98 =	15.84^	0.5			
r_t =	2.51 in =	10.00 in/	3.98			
Lp =	60.5 in =	1 x	2.51 x	(29000 ksi/	50 ksi)^	0.5
Lp =	5.04 ft =	60.51 in/	12			

$$Lr = \pi r_t \sqrt{\frac{E}{F_{yr}}} \qquad (6.10.8.2.3\text{-}5)$$

Fyr = min of 0.7*Fyc or Fyw. Since Fyc = Fyw, Fyr = 0.7*Fyc
Fyr = 35 ksi

Lr =	227.2 in =	3.14 x	2.51 x	(29000 ksi/	35 ksi)^	0.5
Lr =	18.94 ft =	227.23 in/	12			

Therefore Lp<Lb<Lr

- If $L_p < L_b \leq L_r$, then:

$$F_{nc} = C_b\left[1-\left(1-\frac{F_{yr}}{R_h F_{yc}}\right)\left(\frac{L_b-L_p}{L_r-L_p}\right)\right]R_b R_h F_{yc} \leq R_b R_h F_{yc}$$

(6.10.8.2.3-2)

C_b =	1.00	(Conservatively Assume)			
R_h =	1.00	(Not a hybrid member)			
$2^*D_c/t_w$ =	58.04 =	2 x	13.35 in/	0.46 in	
λ_{rw} =	137.27 =	5.7 x	(29000 ksi/	50.00 ksi)^	0.5
R_b =	1.00				

A = 0.13 = $\dfrac{81.36\text{ in} - 60.51\text{ in}}{227.23\text{ in} - 60.51\text{ in}}$

B = 0.30 = $1 - \dfrac{35\text{ ksi}}{1.00 \times 50.00}$

C = 1-(A*B) = 0.96 = 1 - 0.13 x 0.30

FNC = Cb*C*Rb*Rh*Fyc = 48.12 ksi = 1.00 0.96 x 1.00 1.00 50 ksi

By inspection, LTB Controls, and FNC = 48.12 ksi

Check Bending Stress:

S_x = 213 in³

f_{bu} = 3.59 ksi = 764.65 k-in/ 213 in³

OK

Check Serviceability Limit State:

Service II for Flexure

$$F_{SII} = F_{DCI} + F_{DC2+DW} + 1.3F_{LL+I} \leq 0.95F_y$$

0.95 Fy = 47.5 ksi

Sx = 213 in³

F_{DC} = $\dfrac{M_{DC}}{S_{NCDL}}$ $\dfrac{0.48}{213.00}$ = 0.03

F_{LL} = $\dfrac{M_{DC}}{S_{NCDL}}$ $\dfrac{36.07}{0.00}$ = 2.03

F_{SII} = 2.06 ksi OK

Fatigue: CATEGORY A:

$$\Delta F_n = \left(\frac{A}{N}\right)^{1/3} \geq \frac{1}{2}\Delta F_{TH}$$

ADTT =	8000	trucks/day	
p =	0.65	>3 lanes	
n =	1		
ΔF_{th} =	24	ksi	(1) Table 6.6.1.2.5-3
A =	2.50E+10	ksi^3	(1) Table 6.6.1.2.5-1

N = (365)(75)n(ADTT) = 142350000 (1) 6.6.1.2.5-3

Fatigue Limit is the larger of:

$\Delta Fn = (A/N)^{1/3} =$	5.600	ksi	(1) 6.6.1.2.5-2
$\Delta Fn = \Delta Fth/2 =$	24.0	ksi	
Therefore: $\Delta Fn =$	24.0	ksi	

Truck Moment for Fatigue:

$M_{LL+I} =$	36.07	k-ft	(Impact = 1.33)
Fatigue Dyn. Load allowance, IM =	1.15		

$M_{LL+I\ fatigue} = 0.75(M_{LL+I})(I_m/1.33)$

$M_{LL+I\ fatigue} =$	23.39	k-ft	
S =	213	in^3	
$F_{fatigue} =$	1.32	ksi	OK

Shear:

Vp = 0.58 Fy D tw (1) 6.10.9.2

D =	26.7	in	
tw =	0.460	in	
D/tw =	58.0		
k =	5.0		
E =	29000	ksi	
$1.12\sqrt{Ek/Fy} =$	60.3	>D/tw	
C =	1.0		(1) 6.10.9.3.2-4
Vp =	356.2	kips	
Vn = CVp =	356.18	kips	
Φ =	1.0		
ΦVn =	356.18	kips	> Vu OK

3/4 BEARING STIFFENER

Shear:

$V_{yield} = 0.58\ Fy\ D\ t_p$

$V_{rup} = 0.58\ R_p\ Fu\ A_{vn}$ (1) 6.10.9.2

D =	31	in	
t_p =	0.75	in	
V_{yield} =	674.3	kips	
$\phi_v{}^*V_{yield}$ =	674.3	kips	
F_u =	65.0	ksi	
R_p =	1.0		
Hole Diam =	1.000	in	
No. of Rows =	2.000		
No. of Bolts/Row =	7.000		
No. of Bolts = No. of Holes =	14		
A_{vn} =	12.8	in^2	
V_{rup} =	480.7	kips	
$\phi_{vr}{}^*V_{rup}$ =	384.5	kips	
ΦVn =	384.54	kips	> Vu OK

FINAL DESIGN - INTERIOR END DIAPHRAGM DESIGN - SPAN 1 VTRANS D37 IM 091-1(68)

Bolted Connection:

Connection Geometry:

Bolt Type =	F3125	
Bolt Diam., db =	0.875	in
A_{bolt} =	0.601	in^2
Hole Diam. =	1.00	in
No. Rows =	1	
Bolts/Row =	7	
Total Bolts =	7	
Number of Slip Planes, N_s =	1	
Edge Distance =	1.50	in

Note that web of W shape is thinner than the 1/2" connection plate, therefore only W shape is investigated for bearing and block shear.

t = 0.46

Nominal Shear Resistance:

Per (1) 6.13.2.7-1 (threads included), Rn = 0.45*0.83* A_b F_{ub} N_s

d_b =	0.875	in			
A_b =	0.60	in^2			
F_{ub} =	120	ksi			
Ns =	1				
Rn =	26.95	kips / bolt			
Total No. of bolts =	7				
Φs =	0.8				(1) 6.5.4.2
ΦRn =	150.93	kips	> Vu	OK	

Slip Resistance:

Per (1) 6.13.2.8-1, slip resistance Rn = $K_h\ K_s\ N_s\ P_t$

Kh =	1				(1) Table 6.13.2.8-2
Ks =	0.5				(1) Table 6.13.2.8-3, (2) 6.4.2.1.1
Ns =	1				
Pt =	39	kips			(1) Table 6.13.2.8-1
Rn =	19.50	kips/bolt			
Total Rn =	136.50	kips			
ϕ =	0.80				(1) 6.5.4.2
Rn =	109.20	kips			

Service II: Rr = 1.0 DL + 1.3 LL (1) 3.13.2.2-1

Rr =	14.12	kips	< Rn	OK	

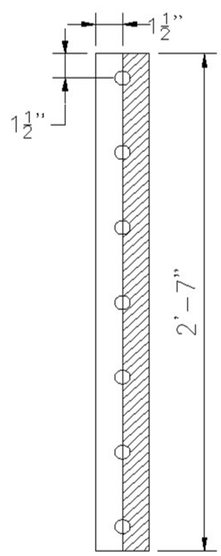
Bearing:

Per (1) 6.13.2.9-4, strength of bolt in bearing for regular holes, R_n = 1.2*L_c t F_u

h = d_b + 1/8 =	1.00	in			
Le_1 =	1.5	in	(edge distance)		
Lc = Le_1 - h/2 =	1.00	in			
t =	0.460	in			
F_u =	65	ksi			
R_n =	35.88	kips/bolt			
Φ_{bb} =	0.8				(1) 6.5.4.2
ΦR_n =	200.93	kips	> Vu	OK	

Block Shear:

Per (1) 6.13.5.3-1, block shear strength Rr = ϕ_{bs}*R_p *(0.58*F_u*A_{vn} + U_{bs}*F_u*A_{tn}) ≤ ϕ_{bs}*R_p*(0.58 F_y A_{vg} + U_{bs} F_u A_{tn})



NET AREA
BLOCK SHEAR

ϕ_{bs} =	0.8		(1) 6.5.4.2
R_p =	0.9	Assume punched to full sized	
F_u =	65	ksi	
Ubs =	1.00		
No. of bolts=	7		
Hole size =	1.00	in	
L_{nv} = D - ((No. of Bolts*0.5)*Hole Size) =	24.50	in	
width $_{nv}$= t =	0.460	in	
A_{vn} =	11.27	in^2	
0.58*F_u*A_{vn} =	424.88	kips	(1) 6.13.5.3-2
Tension Length, L_t =	1.5	in	
Tension Width = t =	0.460	in	
A_{tn} =(L_t - 0.5*Hole Size)*t =	0.460	in^2	
R_r =	327.441	kips	

R_r = ϕ_{bs}*R_p*(0.58 F_y A_{vg} + U_{bs} F_u A_{tn})

Length, D - Le_1 =	29.5	in	
Shear Width = Plate Thickness =	0.460	in	
Avg =	13.57	in^2	
R_r =	304.87	kips	
Block Shear Strength, R_r=	304.87	> Vu	OK

References:
1) AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017 with Interims thru 2018
2) VTrans Structures Design Manual, 5th Edition

Skew =	28.65	°
Dia. Sp. =	6.67	ft
Length of Dia., L =	7.60	ft

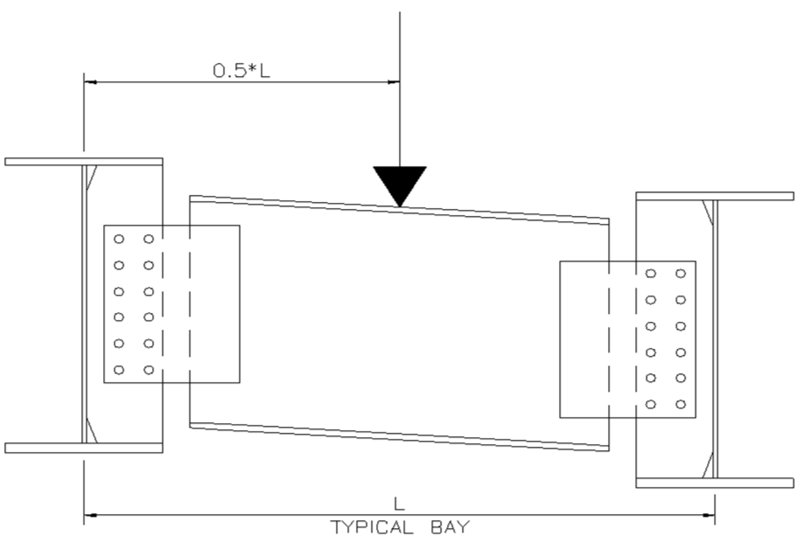
W Shape Self Weight: W27X84

Self Wt =	84	lb/ft
L =	7.60	ft
M_{DL} =	0.61	k-ft
V_{DL} =	0.32	kips

Live Load:

HL-93 wheel load is applied as shown below.

P =	16	kips	
Dyn. Load allowance, IM =	1.33		(1) Table 3.6.2.1-1
P' = P x IM =	21.28		
V_{LL} = 0.5*P x IM =	10.64	kips	
M_{LL+I} =	40.41	Kips-ft	



Determine Governing Loads: (1) Table 3.4.1-1

For Strength I, γ_p (DL) = 1.25
Live Load Factor = 1.75

Dead + Live Load:

M_u = 71.48 k-ft = 1.25 x 0.61 k-ft + 1.75 x 40.41 k-ft
V_u = 19.02 kips = 1.25 x 0.32 k + 1.75 x 10.64 k

Mu =	71.48	ft kips
Vu =	19.02	kips

FINAL DESIGN - INTERIOR END DIAPHRAGM DESIGN - SPAN 2 VTRANS D37 IM 091-1(68)

STEEL W BEAM

Flexure: (1) 6.12.2.2.5-1

L_b =	7.60	ft				
Fy =	50	ksi				
E =	29000	ksi				
D =	26.7	in				
D_c =	13.4	in^3				
t_f =	0.64	in				
b_f =	10.00	in				
t_w =	0.46	in				
$b_f/2*t_f$ =	7.81 =		10.00 in/	(2 x	0.64 in)	

For a W-Shaped beam that has a compression flange braced discretetly, the following shall be satisfied:

$$f_{bu} + \frac{1}{3} f_\ell \leq \phi_f F_{nc} \quad (6.10.8.1.1\text{-}1)$$

where:

ϕ_f = resistance factor for flexure specified in Article 6.5.4.2
f_{bu} = flange stress calculated without consideration of flange lateral bending determined as specified in Article 6.10.1.6 (ksi)
f_ℓ = flange lateral bending stress determined as specified in Article 6.10.1.6 (ksi)
F_{nc} = nominal flexural resistance of the flange determined as specified in Article 6.10.8.2 (ksi)

Check Local Buckling Resistance

Calculate λpf =

Lambda PF =	9.15	0.38 x	(29000/	50.00)^	0.5
Lambda F =	7.81				

Since Lambda f < Lambda PF, Fyc for Local Buckling = Fy

Check Lateral Torsional Buckling (LTB)

Lb = 7.60 ft

L_p = limiting unbraced length to achieve the nominal flexural resistance of $R_b R_h F_{yc}$ under uniform bending (in.)

$$= 1.0 r_t \sqrt{\frac{E}{F_{yc}}} \quad (6.10.8.2.3\text{-}4)$$

r_t = effective radius of gyration for lateral torsional buckling (in.)

$$= \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}} \quad (6.10.8.2.3\text{-}9)$$

A =	1.32 =	1 +	13.35 in x / 3 x	0.46 in / 10.00 in x	0.64	
B = 12*A =	15.84 =	12 x	1.32			
C = $B^{0.5}$ =	3.98 =	15.84^	0.5			
r_t =	2.51 in =	10.00 in/	3.98			
Lp =	60.5 in =	1 x	2.51 x	(29000 ksi/	50 ksi)^	0.5
Lp =	5.04 ft =	60.51 in/	12			

$$Lr = \pi r_t \sqrt{\frac{E}{F_{yr}}} \quad (6.10.8.2.3\text{-}5)$$

Fyr = min of 0.7*Fyc or Fyw. Since Fyc = Fyw, Fyr = 0.7*Fyc

Fyr = 35 ksi

Lr =	227.2 in =	3.14 x	2.51 x	(29000 ksi/	35 ksi)^	0.5
Lr =	18.94 ft =	227.23 in/	12			

Therefore Lp<Lb<Lr

- If $L_p < L_b \le L_r$, then:

$$F_{nc} = C_b\left[1-\left(1-\frac{F_{yr}}{R_h F_{yc}}\right)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right]R_b R_h F_{yc} \le R_b R_h F_{yc} \quad (6.10.8.2.3\text{-}2)$$

C_b = 1.00 (Conservatively Assume)

R_h =	1.00	(Not a hybrid member)			
$2^*D_c/t_w$ =	58.04 =	2 x	13.35 in/	0.46 in	
λ_{rw} =	137.27 =	5.7 x	(29000 ksi/	50.00 ksi)^	0.5
R_b =	1.00				

A = 0.18 = (91.16 in - 60.51 in) / (227.23 in - 60.51 in)

B = 0.30 = 1 - 35 ksi / (1.00 x 50.00)

C = 1-(A*B) = 0.94 = 1 - 0.18 x 0.30

FNC = Cb*C*Rb*Rh*Fyc = 47.24 ksi = 1.00 0.94 x 1.00 1.00 50 ksi

By inspection, LTB Controls, and FNC = 47.24 ksi

Check Bending Stress:

S_x = 213 in³

f_{bu} = 4.03 ksi = 857.78 k-in/ 213 in³

OK

Check Serviceability Limit State:

Service II for Flexure

$$F_{SII} = F_{DCI} + F_{DC2+DW} + 1.3F_{LL+I} \le 0.95F_y$$

0.95 Fy = 47.5 ksi

Sx = 213 in³

F_{DC} = M_{DC} / S_{NCDL} = 0.61 / 213.00 = 0.03

F_{LL} = M_{DC} / S_{NCDL} = 40.41 / 0.00 = 2.28

F_{SII} = 2.31 ksi OK

Fatigue: CATEGORY A:

$$\Delta F_n = \left(\frac{A}{N}\right)^{1/3} \ge \frac{1}{2}\Delta F_{TH}$$

ADTT =	8000	trucks/day	
p =	0.65	>3 lanes	
n =	1		
ΔF_{th} =	24	ksi	(1) Table 6.6.1.2.5-3
A =	2.50E+10	ksi³	(1) Table 6.6.1.2.5-1

N = (365)(75)n(ADTT) = 142350000 (1) 6.6.1.2.5-3

Fatigue Limit is the larger of:

$\Delta Fn = (A/N)^{1/3} =$	5.600	ksi		(1) 6.6.1.2.5-2
$\Delta Fn = \Delta Fth/2 =$	24.0	ksi		
Therefore: $\Delta Fn =$	24.0	ksi		

Truck Moment for Fatigue:

$M_{LL+I} =$	40.41	k-ft	(Impact = 1.33)	
Fatigue Dyn. Load allowance, IM =	1.15			

$M_{LL+I\ fatigue} = 0.75(M_{LL+I})(I_m/1.33)$

$M_{LL+I\ fatigue} =$	26.21	k-ft		
S =	213	in^3		
$F_{fatigue} =$	1.48	ksi		OK

Shear:

Vp = 0.58 Fy D tw (1) 6.10.9.2

D =	26.7	in		
tw =	0.460	in		
D/tw =	58.0			
k =	5.0			
E =	29000	ksi		
$1.12\sqrt{Ek/Fy} =$	60.3	>D/tw		
C =	1.0			(1) 6.10.9.3.2-4
Vp =	356.2	kips		
Vn = CVp =	356.18	kips		
Φ =	1.0			
ΦVn =	356.18	kips	> Vu	OK

1/2 BENT PLATE

Shear:

V_{yield} = 0.58 Fy D t_p

V_{rup} = 0.58 R_p Fu A_{vn} (1) 6.10.9.2

D =	18	in		
t_p =	0.50	in		
V_{yield} =	261.0	kips		
ϕ_v*V_{yield} =	261.0	kips		
F_u =	65.0	ksi		
R_p =	1.0			
Hole Diam =	1.000	in		
No. of Rows =	2.000			
No. of Bolts/Row =	6.000			
No. of Bolts = No. of Holes =	12			
A_{vn} =	3.0	in^2		
V_{rup} =	113.1	kips		
ϕ_{vr}*V_{rup} =	90.5	kips		
ΦVn =	90.48	kips	> Vu	OK

Bolted Connection:

Connection Geometry:

Bolt Type =	F3125	
Bolt Diam., db =	0.875	in
A_{bolt} =	0.601	in^2
Hole Diam. =	1.00	in
No. Rows =	2	
Bolts/Row =	6	
Total Bolts =	12	
Number of Slip Planes, N_s =	1	
Edge Distance =	1.50	in

Note that web of W shape is thinner than the 1/2" connection plate, therefore only W shape is investigated for bearing and block shear.

t = 0.46

Nominal Shear Resistance:

Per (1) 6.13.2.7-1 (threads included), Rn = $0.45*0.83* A_b F_{ub} N_s$

d_b =	0.875	in			
A_b =	0.60	in^2			
F_{ub} =	120	ksi			
Ns =	1				
Rn =	26.95	kips / bolt			
Total No. of bolts =	12				
Φs =	0.8				(1) 6.5.4.2
ΦRn =	258.73	kips	> Vu	OK	

Slip Resistance:

Per (1) 6.13.2.8-1, slip resistance Rn = $K_h K_s N_s P_t$

Kh =	1				(1) Table 6.13.2.8-2
Ks =	0.5				(1) Table 6.13.2.8-3, (2) 6.4.2.1.1
Ns =	1				
Pt =	39	kips			(1) Table 6.13.2.8-1
Rn =	19.50	kips/bolt			
Total Rn =	234.00	kips			
ϕ =	0.80				(1) 6.5.4.2
Rn =	187.20	kips			
Service II: Rr =	1.0 DL + 1.3 LL				(1) 3.13.2.2-1
Rr =	14.15	kips	< Rn	OK	

Bearing:

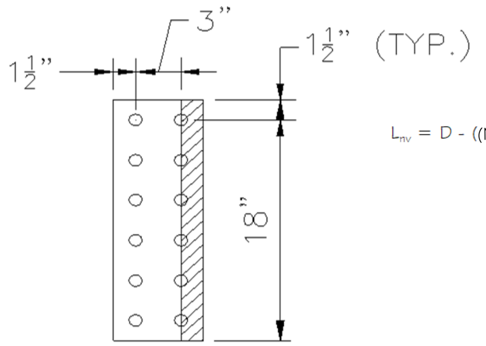
Per (1) 6.13.2.9-4, strength of bolt in bearing for regular holes, $R_n = 1.2*L_c\ t\ F_u$

$h = d_b + 1/8$ =	1.00	in			
Le_1 =	1.5	in	(edge distance)		
$Lc = Le_1 - h/2$ =	1.00	in			
t =	0.460	in			
F_u =	65	ksi			
R_n =	35.88	kips/bolt			
$Φ_{bb}$ =	0.8				(1) 6.5.4.2
$ΦR_n$ =	344.45	kips	> Vu	OK	

FINAL DESIGN - INTERIOR END DIAPHRAGM DESIGN - SPAN 2 VTRANS D37 IM 091-1(68)

Block Shear:

Per (1) 6.13.5.3-1, block shear strength $Rr = \phi_{bs}*R_p*(0.58*F_u*A_{vn} + U_{bs}*F_u*A_{tn}) \leq \phi_{bs}*R_p*(0.58\ F_y\ A_{vg} + U_{bs}\ F_u\ A_{tn})$



NET AREA
BLOCK SHEAR

ϕ_{bs} =	0.8		(1) 6.5.4.2
R_p =	0.9	Assume punched to full sized	
F_u =	65	ksi	
Ubs =	1.00		
No. of bolts=	6		
Hole size =	1.00	in	
L_{nv} = D - ((No. of Bolts*0.5)*Hole Size) =	12.50	in	
width $_{nv}$= t =	0.460	in	
A_{vn} =	5.75	in^2	
$0.58*F_u*A_{vn}$ =	216.78	kips	(1) 6.13.5.3-2
Tension Length, L_t =	1.5	in	
Tension Width = t =	0.460	in	
A_{tn} =(L_t - 0.5*Hole Size)*t =	0.460	in^2	
R_r =	177.606	kips	

$R_r = \phi_{bs}*R_p*(0.58\ F_y\ A_{vg} + U_{bs}\ F_u\ A_{tn})$

Length, D - Le_1 =	16.5	in	
Shear Width = Plate Thickness =	0.460	in	
Avg =	7.59	in^2	
R_r =	180.01	kips	
Block Shear Strength, R_r=	180.01	> Vu	OK

Welded Connection: (1) 6.13.3

$Rr = 0.6\Phi_{e2}$ Fexx (1) 6.13.3.2.2b-1

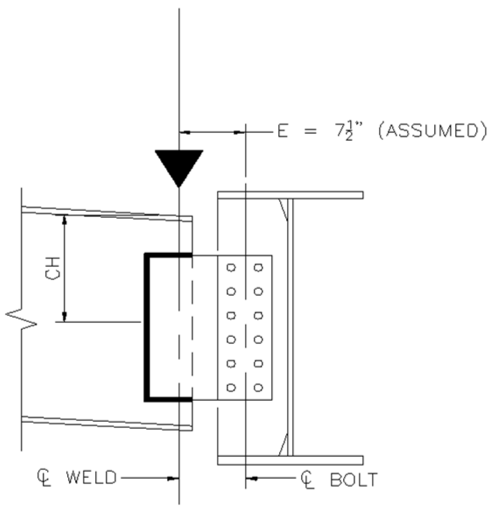
Φe2 =	0.8	
Fexx =	70	ksi
Rr =	33.60	ksi
Thickness of Fillet Weld, t =	0.3125	in

Shear Resistance of Fillet Weld:

v = 0.707 Rr t =	7.42	kips/in

Shear Resistance of Fillet Weld on Connection Plate:

Reference: Design of Welded Structures, Blodgett 1966:



Weld Treated as Line:

b = 5 in
d = 18 in
Aw = L = 28 in

$Jw = ((1/12)*(2b+d)^3) - (b^2(b+d)^2)/(2b+d)$ (Blodgett, Table 5 p.7.4-7)

$((1/12)*(2b+d)^3) =$ 1829.33
$(b^2(b+d)^2) =$ 13225.00
$(2b+d) =$ 28.00
Jw = 1357.01 in^3

Forces:

Vu = P = 19.02 kips
eccentricity, e = 7.5 in (Assumed)
T = Vu e = 142.64 k-in

Twisting (horiz. Component): ch = 13.25 in

fth = T Ch / Jw = 142.6411525 x 13 / 1357.0 = 1.39 kips/in

Twisting (vert. Component): cv = 2.5 in

ftv = T cv / Jw = 142.6411525 x 2.5 / 1357.0 = 0.26 kips/in

Vertical Shear: fnv = P/Aw = 19.02 / 28.0 = 0.68 kips/in

Resultant Force on Weld: $fr = \sqrt{(fth^2 + (ftv + fnv)^2)} =$ 1.68 kips/in

Allowable Force = 7.42 kips/in OK

Fatigue: CATEGORY E:

$$\Delta F_n = \left(\frac{A}{N}\right)^{1/3} \geq \frac{1}{2}\Delta F_{TH}$$

ADTT = 8000 trucks/day
p = 0.65 >3 lanes
n = 1
ΔF_{th} = 4.5 ksi (1) Table 6.6.1.2.5-3
A = 2.20E+09 ksi^3 (1) Table 6.6.1.2.5-1

N = (365)(75)n(ADTT) = 142350000 (1) 6.6.1.2.5-3

Fatigue Limit is the larger of:

$\Delta Fn = (A/N)^{1/3} =$ 2.49 ksi (1) 6.6.1.2.5-2

$\Delta Fn = \Delta Fth/2 =$ 2.3 ksi

Therefore: $\Delta Fn =$ 2.5 ksi

Truck Moment for Fatigue:

V_{LL+I} =	10.64	kips	
eccentricity, e =	12	in	(Assumed)
$V_{LL+I}\ e$ =	127.68	k-in	
$T = 0.75\ (V_{LL+I}\ e)\ (1.15/1.33)$ =	82.8	k-in	
Aw = L =	28	in	
J =	1357.01	in^3	

Vertical Shear: fnv = P/Aw = 10.64 / 28.0 = 0.38 kips/in

Channel tw = 0.3125 $F_{fatigue}$ = fnv / tw = 1.22 ksi OK

PRECAST CONCRETE DECK PANELS — VTRANS D37 IM 091-1(68)

References:
(1) AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017 with interims thru 2018
(2) Partial Depth Deck Panel Guidelines, PCI Northeast Bridge Technical Committee, 2nd Edition
(3) Prestress Bridge Design Aids, 2012
(4) Precast Concrete Deck Panel - Steel Girder, State of New Hampshire

Design Narrative:
Precast concrete deck panels are to act as stay-in-place forms and support the main concrete deck pour. The flexural strength of the deck in the initial condition shall be sufficient to support the panel selfweight and checked against allowable concrete compression limits. The final condition shall be sufficient to support the full weight of the deck, plus HL-93 live load, and shall be checked for tension stress. Assume panels are simply supported between girders. Shear is assumed to not control.

Geometry and Material Properties

Girder Spacing =	6.67 ft	
Panel Length =	5.92 ft	
Panel Width =	8.00 ft	
Panel Thickness =	3.5 in	
Final Deck Thickness (Dead Load) =	9 in	
Final Deck Thickness (Structural) =	8.25 in	
Design WS Thickness =	2.5 in	
Strand Diam. =	0.375 in	(3)
A_{ps} =	0.085 in²	(3)
E_{ps} =	28500.00 ksi	(3)
Unit Weight of Concrete =	0.150 kcf	
Unit Weight of WS =	0.140 kcf	
f'_{ci} =	4 ksi	
f'_c =	6 ksi	
f_{pu} =	270 ksi	
f_y =	60 ksi	

Load Factors

Evaluate the flexural resistance of the panel at the Strength I Limit State
Per (1) Table 3.4.1-1, the factors for the Strength I Limit State

Per (1) 5.9.2.3.2a, compression shall be evaluated at the Service I Limit State
Dead and Live Load factors at the Service I Limit State are equal to 1.0, per (1) Table 3.4.1-1

Live Load Factor =	1.75
Dead Load, DC γ =	1.25
Wearing Surface, DW γ =	1.50

Calculate Dead Load

Assume design is for 1' of slab width.

Panel Load =	0.044 klf/ft =	(3.5 in/	12) x	1 ft x	0.15 kcf		
Concrete Pour Load =	0.069 klf/ft =	((9.00 in -	3.50)/	12) x	1 ft x	0.15 kcf	
WS Load =	0.029 klf/ft =	(2.5 in/	12) x	1 ft x	0.14 kcf		

Factored Panel Load =	0.055 klf/ft =	1.25 x	0.044 klf/ft
Factored Conc. Pour Load =	0.086 klf/ft =	1.25 x	0.069 klf/ft
Factored WS Load =	0.044 klf/ft =	1.50 x	0.029 klf/ft

Factored M_{Panel} =	0.24 k-ft/ ft =	0.125 x	0.055 klf/ft x	5.92 ft ^ 2
Factored M_{Pour} =	0.38 k-ft/ ft =	0.125 x	0.086 klf/ft x	5.92 ft ^ 2
Factored M_{WS} =	0.19 k-ft/ ft =	0.125 x	0.044 klf/ft x	5.92 ft ^ 2

Determine Live Load Moment

Per (1) Table A4, for beam spacing = 6'-8"

Positive Live Load Moment = 5.07 k-ft/ ft

Calculate Design Moments for Strength

At Final Conditon

M_u =	9.68 k-ft =	(0.24 k-ft/ ft +	0.38 k-ft/ ft +	###########	(1.75 5 k-ft/ ft)) x	1 ft

Calculate Initial Presstressing Force

Per (4), space strands at 8", for a total of 12 strands per 8'-0" wide panel.
Per (4), force in each strand is equal to 17.2 kips

No. of Strands =	12		
Force/Strand =	17.2 k		
A_{strand} =	0.085 in²		
Panel Width =	8.00 ft		
A_{ps} =	1.02 in² =	12.00 x	0.085 in²
A_{ps} / ft Panel =	0.13 in²/ ft =	1.02 in²/	8.00 ft
Prestress Force =	206.40 k =	12.00 x	17.20 k
Prestress Force/ft Panel =	25.80 klf =	206.40 k/	8.00 k
f_{pi}=	202.35 ksi =	(25.80 k/	0.13 in²)

Calculate Initial Section Properties

Per (2) prestressing strands are centered vertically in the panel.

b =	12 in				
t =	3.5 in				
A =	42 in² =	12 in x	3.5 in		
I =	42.875 in4 =	0.083 x	12 in x	3.5 in ^	3
c_{top} =	1.75 in =	3.5 in/	2		
c_{bot} =	1.75 in =	3.5 in/	2		
S_{top} =	24.50 in³ =	42.875 in/	1.75 in		
S_{bot} =	24.50 in³ =	42.875 in/	1.75 in		
e =	0 in				

Calculate Pre-Stress Losses

Per (1) 5.9.9.1,

In pretensioned members:

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pLT} \quad (5.9.3.1\text{-}1)$$

Elastic Shortening

Per (1) 5.9.3.2.3a,

$$\Delta f_{pES} = \frac{E_p}{E_{ct}} f_{cgp} \quad (5.9.3.2.3a\text{-}1)$$

where:

f_{cgp} = concrete stress at the center of gravity of prestressing tendons due to the prestressing force immediately after transfer and the self-weight of the member at the section of maximum moment (ksi).

E_p = modulus of elasticity of prestressing steel (ksi)

E_{ct} = modulus of elasticity of concrete at transfer or time of load application (ksi)

E_p =	28500.00 ksi					
E_{ct} =	3644.15 ksi =	33000 x	(0.145 kcf ^	1.5) ^	(4 ksi ^	0.5)
M_{SELF} =	2.30 k-in =	12 x	(0.125 x	0.044 klf x	5.92 ft ^ 2)	
S_{top} =	24.50 in³					
f_{cpg} =	0.52 ksi =	(25.80 k/	42.00 in²) -	(2.30 k-in/	24.50 in³)	
Δf_{pES} =	4.07 ksi =	(28500.00 ksi/	3644.15 ksi) x	0.52 ksi		

Long Term Losses

Per (1) 5.9.3.3,

$$\Delta f_{pLT} = 10.0\frac{f_{pi}A_{ps}}{A_g}\gamma_h\gamma_{st} + 12.0\gamma_h\gamma_{st} + \Delta f_{pR} \qquad (5.9.3.3\text{-}1)$$

in which:

$$\gamma_h = 1.7 - 0.01H \qquad (5.9.3.3\text{-}2)$$

$$\gamma_{st} = \frac{5}{(1+f'_{ci})} \qquad (5.9.3.3\text{-}3)$$

where:

f_{pi} = prestressing steel stress immediately prior to transfer (ksi)
H = average annual ambient relative humidity (percent)
γ_h = correction factor for relative humidity of the ambient air
γ_{st} = correction factor for specified concrete strength at time of prestress transfer to the concrete member
Δf_{pR} = an estimate of relaxation loss taken as 2.4 ksi for low relaxation strand and in accordance with manufacturers recommendation for other types of strand (ksi)

H =	80	percent per (1) Figure 5.4.2.3.3-1			
γ_h =	0.90 =	1.7 -	(0.01 x	80)	
f'_{ci} =	4 ksi				
γ_{st} =	1.00 =	5/	(1 +	4 ksi)	
f_{pi} =	202.35 ksi				
A_{ps} =	0.13 in²				
A_g =	42.00 in²				
$f_{pi}*A_{ps}/A_g$ =	0.61 ksi =	(202.35 ksi x	0.13 in²)/	42.00 in²	
$\gamma_h*\gamma_{st}$ =	0.90 =	0.90 x	1.00		
Δf_{pR} =	2.4			(1) 5.9.3.3	
Δf_{pLT} =	18.73 ksi =	(10 x	0.61 ksi x	0.90) +	
		(12 x	0.90) +	2.4 ksi	

Total Losses

Δf_{PT} = 22.80 ksi = 4.07 ksi + 18.73 ksi

Check Panel Compression at Release

Per (1) 5.9.2.3.2a, compression shall be evaluated at the Service I Limit State
Dead and Live Load factors at the Service I Limit State are equal to 1.0, per (1) Table 3.4.1-1
Normalize prestressing force, P, to a 1 ft equivalent width.

P =	25.28 k =	(202.35 ksi -	4.07 ksi) x	0.13 in²	
A_g =	42.00 in²				
M_{SELF} =	2.30 k-in				
S_{top} =	24.50 k-in				
σ_c =	0.70 ksi =	(25.28 k/	42.00 in²) +	(2.30 k-in/	24.50 in³)

Per (2) compression in the panel should be less than or equal to approximately 0.75 ksi. By inspection, compression stress is ok.

Check Compression at Deck Casting

Per (1) 5.9.2.3.2a, compression shall be evaluated at the Service I Limit State
Dead and Live Load factors at the Service I Limit State are equal to 1.0, per (1) Table 3.4.1-1

P =	25.28 k				
A_g =	42.00 in²				
M_{SELF} =	2.30 k-in				
M_{POUR} =	1.22 k-in =	1.5 x	0.069 klf x	5.92 ft ^ 2	
S_{top} =	24.50 in³				

σ_c = 0.75 ksi = (25.28 k/ 42.00 in²) + ((2.30 k-in + 1.22 k-in)/ 24.50 in³)

Per (2) compression in the panel should be less than or equal to approximately 0.75 ksi. By inspection, compression stress is ok.

Check Panel for Compression under Permananet Loads

Deck f'_c =	4000 psi
Panel f'_c =	6000 psi
w_{con} =	150 pcf

E_{deck} =	3834253.51 psi =	33 x	(150 pcf ^	1.5) x (4000 psi ^	0.5)
E_{panel} =	4695982.33 psi =	33 x	(150 pcf ^	1.5) x (6000 psi ^	0.5)

n = 1.22 = 4695982.33 psi/ ##############

Determine Location of Neutral Axis

	b	b_{eff}	h	A	y	Ay
Deck	12 in	9.80 in	4.75 in	46.54 in²	2.375 in	110.53 in³
Panel	12 in	12 in	3.5 in	42.00 in²	6.5 in	273.00 in³
				88.54 in²		383.53 in³

y_{bar} = 4.33 in = 383.53 in³/ 88.54 in²

By inspection, the neutra axis is within the concrete deck pour, therefore, the entire panel is in tension.

Calculate Flexural Capacity of Deck Panel

Evaluate per (1) 5.6.3.1.

f'_c =	6	ksi
Deck Thickness =	9	in
α_1 =	0.85	
β_1 =	0.85	
b =	12	in

Per (1) 5.6.3.1.1-3,

$$c = \frac{A_{ps} f_{pu} + A_s f_s - A'_s f'_s}{\alpha_1 f'_c \beta_1 b + k A_{ps} \frac{f_{pu}}{d_p}} \qquad \textbf{(5.6.3.1.1-4)}$$

A_{ps} =	1.02 in²				
f_{pu} =	270 ksi				
k =	0.28		(1) Table C5.6.3.1.1-1		
d_p =	1.75 in =	3.5 in/	2		
A_{ps}*f_{pu} =	275.40 k =	1.02 in² x	270 ksi		
α_1*f'_c*β_1*b =	52.02 k/ in =	0.85 x	6 ksi x	0.85 x	12 in
k*A_{ps}*(f_{pu}/d_p) =	44.06 =	0.28 x	1.02 in² x	(270 ksi/	1.75 in)

$$c = 2.87 \text{ in} = \frac{275.40 + 0.00}{52.02 + 44.06}$$

Per (1) 5.6.3.2.2,

$$M_n = A_{ps} f_{ps} \left(d_p - \frac{a}{2}\right) + A_s f_s \left(d_s - \frac{a}{2}\right) - A'_s f'_s \left(d'_s - \frac{a}{2}\right) + \alpha_1 f'_c (b - b_w) h_f \left(\frac{a}{2} - \frac{h_f}{2}\right) \qquad \textbf{(5.6.3.2.2-1)}$$

a = 2.44 = 2.87 in x 0.85

Per (1) 5.6.3.1.1,

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) \qquad (5.6.3.1.1\text{-}1)$$

f_{pu} =	270 ksi							
k =	0.28							
c =	2.87 in							
d_p =	7.25 in =	9.00 in -	(3.50/	2)				
f_{ps} =	240.11 ksi =	270 ksi x		(1 -	(0.28 x	(2.87 in/	7.25 in)))	
A_{ps}*f_{ps}*(d_p-a/2) =	1477.29 k-in =	1.02 in² x	240.11 ksi x	(7.25 in -	(2.44 in/		2))	
M_n =	123.11 k-ft =	1477.29	0.083					
ϕ_f =	1.00		(1) 5.5.4.2					
ϕM_n =	123.11 k-ft =	1.00 x	123.11 k-ft					
M_u =	9.68 k-ft							

OK

Check Tensile Stress in Panel

<u>Calculate Applied Loads in Final Condition</u>

Per (1) 5.9.2.3.2b, evaluate tension stresses at the Service III Limit State.
Dead load factors at the Service III Limit State are equal to 1.0, per (1) Table 3.4.1-1
The live load factor at the Service III Limit State are equal to 0.8, per (1) Table 3.4.1-1

Initial compression force is appled to the panel section only.
Dead load due to panel selfweight is applied to the panel section only.
Dead Load due to whole deck is applied to the panel section only.
Dead Load due to wearing surface is applied to the full deck section.
Dead load due to exterior barrier, railing and snow fence was applied to the deck overhang and the resulting posative moments at the interior panels were calculated using aSTAAD model. That moment was also applied to the full deck section.

Live load is applied to the full deck section.

Panel Load =	0.044 klf			
Deck Load =	0.069 klf			
DW =	0.029 klf			
Panel Length =	5.92 ft			
Panel Moment =	0.191 k-ft =	0.125 x	0.044 klf x	5.92 ft ^ 2
Deck Moment =	0.301 k-ft =	0.125 x	0.069 klf x	5.92 ft ^ 2
WS Moment =	0.128 k-ft =	0.125 x	0.029 klf x	5.92 ft ^ 2
Barrier Moment =	0.24 k-ft			
Live Load Moment =	4.06 k-ft =	0.8 x	5.07 k-ft	

Calculate Deck Section Properties

Use full deck section in check of tensile stress.
Note that compression is positive.

Panel A_g =	42 in²				
Panel S_{bot} =	24.50 in³				
Full deck b =	12 in				
Full Structual Deck t_{slab} =	8.25 in				
Full Deck A_g =	99.00 in² =	8.25 in x	12 in		
Full Deck I =	562 in4 =	0.083 x	12 in x	8.25 in^ 3	
Full Deck c_{bot} =	4.13 in =	8.25 in/	2		
Full Deck S_{bot} =	136.13 in³ =	562 in4 /	4.13 in		
P =	22.89 k =	(202.35 k -	22.80 ksi) x	0.13 in²	
P/A_g =	0.545 ksi =	22.89 k/	42.00 in²		
M_{panel}/S_{bot} =	0.094 ksi =	2.297 k-in/	24.50 in³		
M_{deck}/S_{bot} =	0.147 ksi =	3.610 k-in/	24.50 in³		
$M_{WS}/S_{comp\ bot}$ =	0.011 ksi =	1.532 k-in/	136.13 in³		
$M_{bar}/S_{comp\ bot}$ =	0.021 ksi =	2.849 k-in/	136.13 in³		
$M_{LL}/S_{comp\ bot}$ =	0.358 ksi =	48.672 k-in/	136.13 in³		
σ_T =	-0.09 ksi =	0.545 ksi -	0.094 ksi -	0.147 ksi - 0.011 ksi -	0.358 ksi -
		0.02 ksi			

Per (2), limit tensile stress to $0.19 \cdot f'_c{}^{0.5}$

f'_c =	6 ksi			
Tensile Stress Limit =	-0.47 ksi =	0.19 x	(6 ksi ^	0.5)
	OK			

SUBSTRUCTURE CALCULATIONS

Soil Properties - Abutment 1 (B-105) VTRANS D37 IM 091-1(68)

References:
(1) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018
(2) Geotechnical Data Report, Teracon Consultants Inc., March 27. 2019

g_w = 0.062 kcf
g_{sat} = 0.125 kcf

Water Table = 23.00 ft (2)
Ground EL. At Boring = 466.00
BOF EL. = 451.46

Correct Blow Count for Overburden Pressure

	To Depth (ft)	h_1 (ft)	h_2 (ft)	σ'_v (ksf)	C_N	N blows/ft	$N1 = C_N N$ blows/ft	N_{60} = (ER/60%)N blows/ft	$N1_{60} = C_N N_{60}$ blows/ft
	2.00	2.00	0.00	0.25	1.70	12.00	20.37	16.00	27.15
	6.00	6.00	0.00	0.75	1.33	6.50	8.64	8.67	11.52
	10.00	10.00	0.00	1.25	1.16	4.00	4.64	5.33	6.18
BOF	14.00	14.00	0.00	1.75	1.05	8.00	8.37	10.67	11.16
	18.00	18.00	0.00	2.25	0.96	14.50	13.95	19.33	18.61
	22.50	22.50	0.00	2.81	0.89	11.00	9.77	14.67	13.02
	33.50	23.00	10.50	3.53	0.81	12.50	10.14	16.67	13.53
	40.75	23.00	17.75	3.99	0.77	44.00	33.93	58.67	45.24

h_1 = depth above water table
h_2 = depth below water table
$\sigma'_v = gh_1 + g' h_2$
$g' = g_{sat} - g_w$
$C_N = 0.77 \log_{10} (40 / \sigma'_v) < 2$ (1) 10.4.6.2.4.-1

Correct for Hammer Efficiency

N_{60} = (ER/60%)N (1) 10.4.6.2.4-2
ER = 0.80
N_{60} = 1.33 = 0.80/ 0.60
$N1_{60} = C_N N_{60}$ (1) 10.4.6.2.4-3

Calculate Drained Friction Angle

Table 10.4.6.2.4-1—Correlation of *SPT* $N1_{60}$ Values to Drained Friction Angle of Granular Soils (modified after Bowles, 1977)

$N1_{60}$	ϕ_f
<4	25–30
4	27–32
10	30–35
30	35–40
50	38–43

Use middle values of range:

$N1_{60}$	ϕ_f
<4	25.00
4.00	27.00
10.00	30.00
30.00	35.00
50.00	38.00

In determining the internal friction angle for soil below the footing, consider the soil up to two times the footing width.

Footing Width = 16.00 ft
Soil Depth = 32.00 ft = 2.00 x 16.00 ft

Depth of Footing = 14.54 ft

	Depth (ft)	$N1_{60}$	$N1_{60\ low}$	$N1_{60\ high}$	$\phi_{f\ low}$	$\phi_{f\ high}$	ϕ_f	
	2.00	27.15	10.00	30.00	30.00	35.00	34.29	
	6.00	11.52	10.00	30.00	30.00	35.00	30.38	Use ϕ_f = 30.00
	10.00	6.18	4.00	10.00	27.00	30.00	28.09	
BOF	14.00	11.16	10.00	30.00	30.00	35.00	30.29	
	18.00	18.61	10.00	30.00	30.00	35.00	32.15	
	22.50	13.02	10.00	30.00	30.00	35.00	30.76	Use ϕ_f = 31.00
	33.50	13.53	10.00	30.00	30.00	35.00	30.88	
	40.75	45.24	30.00	50.00	35.00	38.00	37.29	

Bearing Capacity - Abutment 1 (B-105)

References:
(1) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018

Bearing Resistance of Soil (10.6.3.1)

Bearing resistance of soil calculated per (1) 10.6.3.1

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma B N_{\gamma m} C_{w\gamma}$$ (1) 10.6.3.1.2a-1

Abutment Geometry

Footing width, B =	16.00	ft		See GRS Abutment Design
Footing length, L =	36.20	ft		See GRS Abutment Design
Embedment Depth, D_f =	4.00	ft		See GRS Abutment Design
Groundwater Depth, D_w =	23.00	ft		See GRS Abutment Design

Cohesive Term

ϕ_f =	0		
c =	0		
i_c =	1.00	(load inclination not considered)	(1) 10.6.3.1.2a-6
N_c =	0		(1) Table 10.6.3.1.2a-1

Surcharge Term (soil above footing)

ϕ_f =	30.00		
γ =	0.125	kcf	
D_f/B =	0.25		
$1.5B + D_f$ =	28.00		
i_q =	1.00	(load inclination not considered)	(1) 10.6.3.1.2a-7
N_q =	18.40		(1) Table 10.6.3.1.2a-1
C_{wq} =	1.00		(1) Table 10.6.3.1.2a-2
d_q =	1.00		(1) Table 10.6.3.1.2a-4

Unit Weight Term (soil below footing)

ϕ_f =	31.00		
γ =	0.125	kcf	
$1.5B + D_f$ =	28.00		
i_γ =	1.00	(load inclination not considered)	(1) 10.6.3.1.2a-7
N_γ =	26.00		(1) Table 10.6.3.1.2a-1
$C_{w\gamma}$ =	1.00		(1) Table 10.6.3.1.2a-2

Bearing Capacity - Abutment 1 (B-105)

Considerations for footings on slope

Is footing bearing on or near slope ? no

$q_{n\text{-sloping ground}} = RC_{BC}(cN_c+0.5\gamma BN_\gamma)$ (1) 10.6.3.1.2c-1

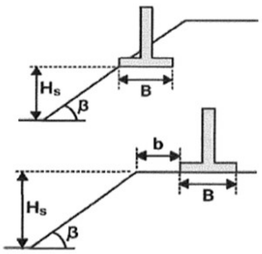
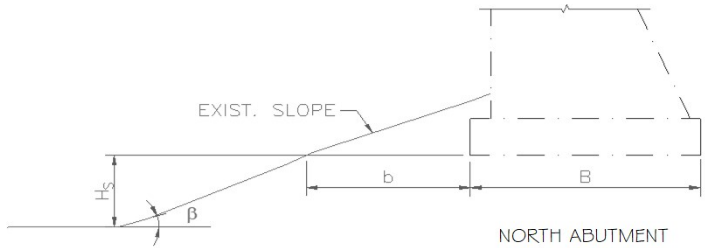


Figure 10.6.3.1.2c-1—Definition of Footing and Slope Geometric Parameters for Determination of RC_{BC}



Height of Slope, H_s =	0.00	ft
b =	0.00	ft
Angle of slope, β =	0.00	degrees
γ =	0.125	kcf
N_s =	c'=0	
φ =	31.00	

For β = 30 degrees, c' = 0

B/H_s	b/B	RC_{BC}	
2	0.50	0.64	(1) Table 10.6.3.1.2c-1
2	1.25	0.74	(1) Table 10.6.3.1.2c-1

Factored Bearing Resistance:

Strength, ϕ_b = 0.45 (1) 10.5.5.2.2

Extreme and Service, ϕ_b = 1 (1) 10.5.5.3.3

For footing w/o slope

$q_n = cN_{cm}+\gamma D_f N_{qm}C_{wq}+0.5\gamma BN_{\gamma m}C_{w\gamma}$ (1) 10.6.3.1.2a-1

$N_{cm} = N_c s_c i_c$ (1) 10.6.3.1.2a-2

$N_{qm} = N_q s_q d_q i_q$ (1) 10.6.3.1.2a-3

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ (1) 10.6.3.1.2a-4

$s_c = 1+(B/L)(N_q/N_c)$ (1) Table 10.6.3.1.2a-3

$s_q = 1+B/L(\tan\phi_f)$ (1) Table 10.6.3.1.2a-3

$s_\gamma = 1-0.4(B/L)$ (1) Table 10.6.3.1.2a-3

For footing w/ slope

$q_{n\text{-sloping ground}} = RC_{BC}(cN_c+0.5\gamma BN_\gamma C_{w\gamma})$ (1) 10.6.3.1.2c-1

Bearing Capacity - Abutment 1 (B-105)

Limit State	e_B (ft)	B' (ft)	s_c	s_q	s_γ	N_{cm}	N_{qm}	$N_{\gamma m}$	B'/H_s	b/B'	RC_{BC}	q_n (ksf)	$\phi_b q_n$ (ksf)
			Shape Correction Factor			Bearing Capacity Factor			Slope Consideration			Bearing Capacity	
Str I - A	2.50	11.00	0.00	1.18	0.88	0.00	21.63	22.84	n/a	n/a	n/a	26.52	11.93

FINAL DESIGN - GRS-IBS ABUTMENT - ABUTMENT 1 DESIGN — VTRANS D37 IM 091-1(68)

References:

1) GRS-IBS Interim Implementation Guide, FHWA, June 2018
2) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018

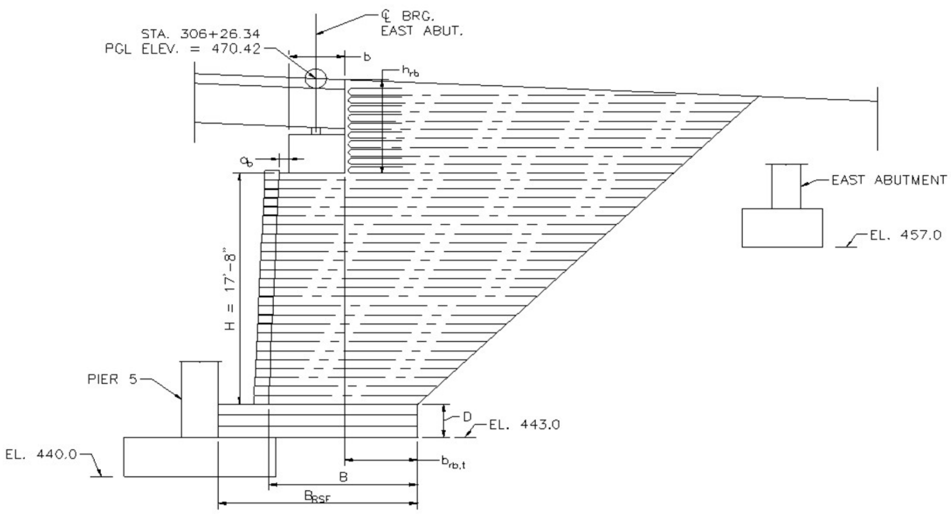
Material Properties

γ_f =	115.00 pcf	(Reinforced Fill)				
ϕ (Reinforced Soil)=	45	degrees				
K_a (Reinforced Soil) =	0.17 =		$\tan^2($	45 -	(45/	2))

Fill retained is in-situ soil.

γ_b =	125.00 pcf	(Retained Fill)				
ϕ (Existing Soil Above Footing)=	30.00	degrees				
K_{ab} (Existing Fill) =	0.33 =		$\tan^2($	45 -	(30/	2))

Geometry



Bridge Span Length =	83.94 ft			
Abut. Height =	21.04 ft			
H =	21.04 ft			
Superstructure Depth =	4.39 ft			
h_{rb} =	6.86 ft			
B =	12.5 ft			
b =	4.75 ft			
Toe Length, X_{RSF} =	2.50 ft			
Total Width, B_{RSF} =	16 ft			
$b_{rb,t}$ =	7.08 ft =	12.5 ft -	0.67 ft -	4.75 ft
Depth of RSF, D =	2.50 ft			
Setback Distance, a_b =	0.67 ft			
L =	31.20 ft			
h_{block} =	8.00 in			
D_{block} =	12.00 in			
L_{block} =	18.00 in			
Weight =	85 lbs	per block		

Reinforcement

S_v =	0.67 ft
d_{max} =	0.75 in
T_f =	5.90 klf

Load Factors
Per (3) Table 3.4.1-1 and Table 3.4.1-2.

γ_p (DC, Min.) =	0.90	3) Table 3.4.1-2
γ_p (DC, Max.) =	1.25	3) Table 3.4.1-2
γ_p (DW, Min.) =	0.65	3) Table 3.4.1-3
γ_p (DW, Max.) =	1.50	3) Table 3.4.1-4
γ_p (EH, Max) =	1.50	3) Table 3.4.1-2
γ_p (EH, Min.) =	0.90	3) Table 3.4.1-2
γ_p (EV, Min.) =	1.00	3) Table 3.4.1-2
γ_p (EV, Max.) =	1.35	3) Table 3.4.1-2
Factor (LS) =	1.75	3) Table 3.4.1-1
Factor (LL) =	1.75	3) Table 3.4.1-1

Calculate Average Height of Precast Beam Seat

Min. Height of Cap =	1.50 ft			
Cross Slope =	0.0625			
Cap Length =	31.20 ft			
Max. Height =	3.45 ft =	1.50 ft +	(31.20 ft x	0.0625)
Average Height =	2.47 ft			

Calculate Vertical Loads and Applied Pressures

Calculate q_{DL}

$$q_{DL} = \frac{Q_{DL}}{bL}$$

	Total DC Rxn	No. of Girders	DC1	DC2
N. Ext.	46.34 k =	1 x	(41.18 k +	5.16 k)
Typ. Int.	143.73 k =	3 x	(42.75 k +	5.16 k)
S. Ext.	42.46 k =	1 x	(37.3 k +	5.16 k)
Total Girder DC Reaction =	232.53 k			

	Total DW Rxn	No. of Girders	DW
N. Ext.	7.09 k =	1 x	7.09 k)
Typ. Int.	21.27 k =	3 x	7.09 k)
S. Ext.	7.09 k =	1 x	7.09 k)
Total Girder DW Reaction =	35.45 k		

Assume a concrete end block integral with the steel beam that is 3'-1" high x 12" wide x full width of abutment.

End Block Weight =	14.43 k =	3.083 ft x	1.00 ft x	31.20 ft x	0.150 kcf

Assume concrete deck beyond CL of bearing is equal to 1.60 SF, per AutoCAD

Additional Deck Weight =	7.49 k =	1.60 ft² x	0.150 kcf x	31.20 ft

Assume that girder extends additional 2'-0" beyond the CL of Bearing

Top Flange =	0.10 ft² =	16 in x	0.875 in x	0.007
Web =	0.12 ft² =	34 in x	0.5 in x	0.007
Bottom Flange =	0.14 ft² =	16 in x	1.25 in x	0.007
Total Steel Area =	0.35 ft²			

Additional Weight of Steel =	1.74 k =	0.35 ft² x	0.49 kcf x	2.00 ft x	5 girders	
Weight of Beam Seat =	55.01 k =	2.47 ft x	4.75 ft x	31.20 ft x	0.15 kcf	
Total DC Reaction (Unfactored) =	311.20 k =	232.53 k +	14.43 k +	7.49 k +	1.74 k +	55.01 k
Total DW Reaction (Unfactored) =	35.45 k					

q_{DC} =	2099.98 psf =	(311.196 kips x	1000)/	(4.75 ft x	31.20 ft)
q_{DW} =	239.22 psf =	(35.45 kips x	1000)/	(4.75 ft x	31.20 ft)

Calculate q_{LL}

Calculate Live Load Reactions Per Barrel

Roadway Width = 27.00 ft
Number of Trucks = 2
Impact = 1.33

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

Reaction (Single Truck) = 63.99 kips = (32 kips x 83.94 ft) + (32 kips x 69.94 ft) + (8 kips x 55.94 ft) / 83.94 ft

Lane Load Reaction (Single Truck) = 26.86 kips = 0.5 x 0.64 klf x 83.94 ft

Reaction - 1 Lane Loaded = 134.37 kips = 1.2 x ((63.99 kips x 1.33) + 26.86)

Reaction - 2 Lanes Loaded = 223.94 kips = 1 x ((2 x 63.99 kips x 1.33) + (2 x 27 kips))

Max. Reaction = 223.94 kips

Bridge LL Reaction = 223.94 kips

$$q_{LL} = \frac{Q_{LL}}{bL}$$

q_{LL} = 1511.20 psf = (223.94 kips x 1000)/ (4.75 ft x 31.20 ft)

Calculate Traffic Surcharge, q_T

Per (3) 3.11.6.4-1, the equivalent height of soil acting as a surcharge load shall be determined as follows:

Table 3.11.6.4-1—Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic

Abutment Height (ft)	h_{eq} (ft)
5.0	4.0
10.0	3.0
≥20.0	2.0

Note that linear interpolation shall be used for intermediate wall heights.

Total H = 23.54 ft
h_{eq} = 2.00 ft
γ_r = 125.00 pcf

$$q_t = h_{eq}\gamma_b$$

q_T = 250.00 psf = 2.00 ft x 125.00 pcf

Calculate Road Base Surchage, q_{rb}

γ_{rb} = 115.00 pcf

$$q_{rb} = H_{rb}\gamma_{rb}$$

q_{rb} = 788.94 psf = 6.86 ft x 115.00 pcf

Calculate Weight of GRS Abutment

$$W = BH\gamma_r$$

H = 21.04 ft
B = 12.5 ft
γ_r = 115.00 pcf

W = 30245.00 plf = 21.04 ft x 12.5 ft x 115.00 pcf

Calculate Weight of RSF

$$W_{RSF} = B_{RSF} D_{RSF} \gamma_r$$

B = 16 ft
D = 2.5 ft
γ_r = 125.00 pcf

W_{RSF} = 4600.00 plf = 16 ft x 2.5 ft x 115.00 pcf

Calculate Weight of Facing

N_{block} = 31.56 = 21.04 ft/ 0.67 ft
N_{block} = 32

$$W_{face} = N_{block} \frac{W_{block}}{L_{block}}$$

W_{face} = 1813.33 plf = 32 x (85 lbs/ (18.00 in/ 12)

Summary of Applied Loads

q_{DC} = 2099.98 psf
q_{DW} = 239.22 psf
q_{LL} = 1511.20 psf
q_t = 250.00 psf
q_{rb} = 788.94 psf
W = 30245.00 plf
W_{RSF} = 4600.00 plf
W_{face} = 1813.33 plf

Check Beam Seat Pressure

Per (1) 4.3.5.4, the service bearing pressure should be targeted to around 4 ksf.

DC Reaction = 311.20 kips
DW Reaction = 35.45 kips
LL Reaction = 223.94 kips
Total = 570.59 kips

b = 4.75 ft
L = 31.20 ft

q_{seat} = 3.85 ksf = 570.59 kips/ (4.75 ft x 31.20 ft)

OK

Calculte Direct Sliding Effects at RSF/GRS Interface

Calculate Sliding per the (1) and (3) Strength I Load Combination

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2$$ (1) Eq. 9

$$F_{rb} = q_{rb} K_{ab} H$$ (1) Eq. 10

$$F_t = q_t K_{ab} H$$ (1) Eq. 11

$$F_R = \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t$$ (1) Eq. 12

F_b =	9222.53 lbs =	0.5 x	125.00 pcf x	0.33 x	21.04 ft^ 2
F_{rb} =	5533.10 lbs =	788.94 psf x	0.33 x	21.04 ft	
F_t =	1753.33 lbs =	250.00 psf	0.33 x	21.04 ft	

F_R = 25.20 klf = (1.5 x (9222.53 lbs + 5533.10 lbs) + (1.75 1753.33 lbs))/ 1000

$$W_{T,R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EV\,MIN}(q_{rb} b_{rb,t})$$ (1) Eq. 14

$W_{T,R}$ = 47.18 klf = ((1.0 x 30245.00 plf) + (0.9 x 2099.98 psf x 4.75 ft) + (0.65 x 239.22 psf x 5 ft) +
(0.9 x 1813.33 plf) + (1.0 x 788.94 psf x 7.08 ft))/ 1000

Assume that μ = 2/3*tanϕ, per (1) C.1.6.1.1

μ = 0.67 = 0.67 x tan(45)

$$R_R = \Phi_\tau\left(W_{T,R}\mu\right)$$ (1) Eq. 13

R_R = 31.45 klf = 47.18 klf x 0.667

OK

Calculate Direct Sliding at Base of RSF

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2$$ (3) Eq. 17

$$F_{rb,RSF} = q_{rb}K_{ab}(H + D_{RSF})$$ (3) Eq. 18

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF})$$ (3) Eq. 19

$F_{b,\ RSF}$ = 11.54 klf = (0.5 x 125.00 pcf x 0.33 x (21.04 ft + 2.5)^ 2)/ 1000
$F_{rb,\ RSF}$ = 6.19 klf = (788.94 psf x 0.33 x (21.04 ft + 2.50))/ 1000
$F_{t,\ RSF}$ = 1.96 klf = 250.00 psf x 0.33 x (21.04 ft + 2.5 ft))/ 1000

$$F_{R,RSF} = \gamma_{EH\ MAX}\left(F_{b,RSF} + F_{rb,RSF}\right) + \gamma_{LS}F_{t,RSF}$$ (1) Eq. 20

$F_{R,RSF}$ = 30.04 klf = (1.5 x (11.54 klf + 6.19 klf)) + (1.75 x 1.96 klf)

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\ MIN}W_{RSF}$$ (1) Eq. 22

$W_{T,R,RSF}$ = 51.78 klf = 47.18 klf + (1.0 x 4.60 klf)

$$R_{R,RSF} = \Phi_\tau(W_{T,R,RSF}\mu_{RSF})$$ (1) Eq. 21

Note that ϕ = 1.0, per (1) 4.3.6.1.2
Per (2), and the previous calculation, the angle of internal friction beneath the RSF is equal to 31 degrees

$R_{R,\ RSF}$ = 31.11 klf = 1.0 x 51.78 klf x tan(31)

OK

Calculate External Bearing Resistance

Calculate V_R

Per (1) Eq. 26,

$$\sum V_R = \gamma_{EV\ MAX}(W) + \gamma_{EV\ MAX}(W_{RSF}) + \gamma_{DC\ MAX}(W_{face}) + \gamma_{LS}(q_t b_{rb,t}) + \gamma_{EH\ MAX}(q_{rb}b_{rb,t}) + \gamma_{DC\ MAX}(q_{DL}b) + \gamma_{LS}(q_{LL}b)$$

Note that $\gamma_{EV\ Max}$ is used in place of $\gamma_{EH\ Max}$ to factor q_{rb}*$b_{rb,t}$ since this is a summation of vertical reactions.

W =	40.83 klf =	1.35 x	30245.00 plf/	1000	
W_{RSF} =	6.21 klf =	1.35 x	4600.00 plf/	1000	
W_{FACE} =	2.27 klf =	1.25 x	1813.33 plf/	1000	
Q_t =	3.10 klf =	1.75 x	(250.00 psf x	7.08 ft)/	1000
Q_{rb} =	7.54 klf =	1.35 x	(788.94 psf x	7.08 ft)/	1000
Q_{DC} =	12.47 klf =	1.25 x	(2099.98 psf x	4.75 ft)/	1000
Q_{DW}=	1.70 klf =	1.5 x	(239.22 psf x	4.75 ft)/	1000
Q_{LL} =	12.56 klf =	1.75 x	(1511.20 psf x	4.75 ft)/	1000
Total =	86.69 klf				

Calculate $M_{D,R}$

Per (1) Eq. 28

$$\sum M_{D,R} = \gamma_{EH\,MAX} F_{b,RSF}\left(\frac{H+D_{RSF}}{3}\right) + \gamma_{LS} F_{t,RSF}\left(\frac{H+D_{RSF}}{2}\right) + \gamma_{EH\,MAX} F_{rb,RSF}\left(\frac{H+D_{RSF}}{2}\right)$$

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H+D_{RSF})^2$$ (3) Eq. 17

$$F_{rb,RSF} = q_{rb} K_{ab}(H+D_{RSF})$$ (3) Eq. 18

$$F_{t,RSF} = q_t K_{ab}(H+D_{RSF})$$ (3) Eq. 19

$F_{b,\,RSF}$ =	11.54 klf =	(0.5 x	125.00 pcf x	0.33 x	(21.04 ft +	2.5)^	2)/	1000
$F_{rb,\,RSF}$ =	6.19 klf =	788.94 psf x	0.33 x	(21.04 ft +	2.5 ft))/	1000		
$F_{t,\,RSF}$ =	1.96 klf =	250.00 psf x	0.33 x	(21.04 ft +	2.5 ft))/	1000		

M, $F_{b,\,RSF}$ =	135.88 k-ft/ ft =	1.5 x	11.54 klf x	(0.33 x	(21.04 ft +	2.5 ft))
M, $F_{t,\,RSF}$ =	40.41 k-ft/ ft =	1.75 x	1.96 klf x	(0.50 x	(21.04 ft +	2.5 ft))
M, $F_{rb,\,RSF}$ =	109.29 k-ft/ ft =	1.5 x	6.19 klf x	(0.50 x	(21.04 ft +	2.5 ft))
Total =	285.58 k-ft/ ft					

Note that $M_{D,R}$ is taken about the bottom center of the width of the RSF.

Calculate $M_{R,R}$

Per (1) Eq. 29

$$\sum M_{R,R} = (\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b)\left[\left(\frac{b}{2}+a_b\right) - \left(\frac{B_{RSF}}{2} - x_{RSF} - b_{block}\right)\right] +$$
$$(\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t})\left(\frac{B_{RSF}}{2} - \frac{b_{rb}}{2}\right) + \gamma_{EV\,MAX} W\left(\frac{B_{RSF}}{2} - \frac{B}{2}\right) +$$
$$\gamma_{DC\,MAX} W_{face}\left(B + \frac{b_{block}}{2} - \frac{B_{RSF}}{2}\right)$$

$Q_{DL}+Q_{LL}$ =	-38.99 k-ft =	((1.25 x	2.10 ksf) +	(1.50 x	0.24 ksf) +	(1.75 x	1.51 ksf)) x	4.75 ft x
		((2.375 ft x +	0.67 ft) -	(8 ft -	2.5 ft -	1.00 ft))		
Q_t+Q_{rb} =	47.45 k-ft =	((1.75 x	0.25 klf x	7.08 ft) +	(1.35 x	0.789 klf x	7.08 ft)) x	
		(8 ft -	3.54 ft)					
W =	71.45 k-ft =	1.35 x	30.25 klf x	(8 ft -	6.25 ft)			
W_{Face} =	-11.02 k-ft =	1.35 x	1.81 klf x	((2.5 ft +	1 ft) -	8 ft)		
Total =	68.90 k-ft/ ft							

Note that $M_{R,R}$ is taken about the bottom center of the width of the RSF.

Calculate Eccentricity, $e_{B,R}$

Per (1) Eq. 27

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R}$$

$e_{B,R}$ = 2.50 ft = (285.58 k-ft/ ft - 68.90 k-ft/ ft) / 86.69 klf

Calculate $\sigma_{v,base,R}$

Since $e_{B,R}$ = negative, set equal to zero in calcaulation of base bearing pressure.

Per (1) Eq. 25,

$$\sigma_{v,base,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}}$$

$\sigma_{v,base,R}$ = 7.88 ksf = 86.69 klf/ (16 ft - (2 x 2.50 ft)

Per bearing capacity calculation, factored bearing capacity = 11.93 ksf

OK

Calculate Internal Bearing Resistance

Per (1), Eq. 35

$$q_{n,an} = \left[0.7^{\left(\frac{S_v}{6d_{max}}\right)} \frac{T_f}{S_v}\right] K_{pr}$$

$q_{n,an}$ = nominal bearing resistance of the GRS abutment using the analytical method.
S_v = reinforcement spacing.
d_{max} = maximum grain size.
T_f = ultimate reinforcement strength.
K_{pr} = coefficient of passive earth pressure for the reinforced fill (calculated in equation 35).

K_{pr} = 5.83 = $\tan^2$(45 + (45/ 2))

$q_{n,an}$ = 27.36 ksf = [(0.7 ^ (0.67 ft/ (6 x 0.06 ft))) x (5.90 klf/ 0.67 in)] x 5.83

ϕ_{cap} = 0.45 (3) 4.3.7.2

$\phi_{cap} q_{n,an}$ = 12.31 ksf = 0.45 x 27.36 ksf

$$V_{applied,f} = \gamma_{DC\,MAX} q_{DL} + \gamma_{LL} q_{LL}$$

(1) Eq. 32

$V_{app,f}$ = 5.63 ksf = (1.25 x 2.10 ksf) + (1.50 x 0.24 ksf) + (1.75 x 1.51 ksf)

OK

Calculate and Check Deformations

Per (1), Eq. 37

$$q_{DL,allow\ @\varepsilon=1\%} = 0.2\left[0.7^{S_v/6d_{max}} \left(\frac{T_f}{S_v}\right) K_{pr}\right]$$

6*d_{max} = 0.375 ft = 6 x 0.75 in x 0.083

T_f/S_v = 8.85 = 5.90 klf/ 0.67 ft

S_v/6*d_{max} = 1.78 = 0.67 ft/ 0.375 ft

$q_{dl,\ all}$ = 5.47 ksf = 0.2 ((0.7 ^ 1.78 in) x 8.85 x 5.83 in)

q_{DL} = 2.10 ksf

OK

Calculate Lateral Strain

Per (1) 4.3.7.2.2, for the calculation of lateral strain, assume a vertical strain equal to 1% of the abutment height, H.

0.01*H = 0.21 ft = 0.01 x 21.04 ft

$$D_L = \frac{2b_q D_v}{H}$$ (1) Eq. 38

D_L = 0.01 ft = (2 x (4.75 ft + 0.67 ft) x 0.02 ft)/ 21.04 ft
D_L = 0.11 in = 0.01 ft x 12

Per (1) 4.3.7.2.2, the total lateral strain should be limited to twice the vertical strain.

Vertical Strain = 0.42 ft = 2 x 0.21 ft

OK

Calculate Reinforcement Strength

q_{DC} = 2099.98 psf
q_{DW} = 239.22 psf
q_{LL} = 1511.20 psf
q_t = 250.00 psf
q_{rb} = 788.94 psf
W = 30245.00 psf
W_{RSF} = 4600.00 psf
W_{face} = 1813.33 psf

The evaulation of the abutment for the strength limit state is conducted according to (1) 4.3.7.3.1. and using the following equations:

Required Reinforcment:

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 40

Factored Total Lateral Pressure:

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f}$$ (3) Eq. 41

Factored Lateral Pressure due to weight of GRS:

$$\sigma_{h,W,f} = \gamma_{EH\,MAX}(\gamma_r z K_{ar})$$ (3) Eq. 42

Where:
$\gamma_{EH\,MAX}$ = maximum horizontal earth pressure load factor.
γ_r = unit weight of reinforced backfill.
z = depth from the top of the wall.
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

Factored Latreal Pressure due to equivalent bridge load:

$$\sigma_{h,bridge,f} = \frac{(\gamma_{DC\,MAX} q_{DL} + \gamma_{LL} q_{LL}) - (\gamma_{EH\,MAX} q_{rb} + \gamma_{LS} q_t)}{\pi}\left[\alpha_b + \sin(\alpha_b)\cos(\alpha_b) + 2\beta_b\right]K_{ar} \quad (43)$$

Where:

$\gamma_{DC\,MAX}$ = maximum DL load factor.
q_{DL} = superstructure DL pressure.
γ_{LL} = bridge LL surcharge load factor.
q_{LL} = bridge LL pressure.
q_{rb} = surcharge due to the structural backfill of the integrated approach (i.e., road base).
γ_{LS} = LL surcharge load factor.
q_t = roadway LL surcharge.
α_b = angle between wall face and projection of the midline of the bridge surcharge to the wall face found using equation 46 (see figure 23).
β_b = angle between the projections of the inner and outer edge lines of the bridge surcharge to the wall face found using equation 47 (see figure 23).

(3) Eq. 43

$$\sigma_{h,rb,f} = \gamma_{EH\,MAX} q_{rb} K_{ar}$$

(3) Eq. 44

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar}$$

(3) Eq. 45

$$\alpha_b = \tan^{-1}\left(\frac{b}{2z}\right) - \beta_b$$

(3) Eq. 46

$$\beta_b = \tan^{-1}\left(\frac{-b}{2z}\right)$$

(3) Eq. 47

Factored Reinforcement Strength:

$$T_{f,f} = \Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right) = 0.4T_f$$

(3) Eq. 48

Strength Limit State

z	$\sigma_{h,W,f}$	α_b	β_b	$\sigma_{h,bridge,f}$	$\sigma_{h,rb,f}$	$\sigma_{h,t,f}$	$\sigma_{h,f}$	$T_{req,f}$	$T_{f,f}$	
0.67 ft	0.020 ksf	2.59	-1.30	0.682 ksf	0.394 ksf	0.146 ksf	1.242 ksf	1.561 klf	2.36 klf	OK
1.33 ft	0.039 ksf	2.12	-1.06	0.651 ksf	0.394 ksf	0.146 ksf	1.230 ksf	1.546 klf	2.36 klf	OK
2.00 ft	0.059 ksf	1.74	-0.87	0.597 ksf	0.394 ksf	0.146 ksf	1.196 ksf	1.504 klf	2.36 klf	OK
2.67 ft	0.079 ksf	1.46	-0.73	0.536 ksf	0.394 ksf	0.146 ksf	1.155 ksf	1.452 klf	2.36 klf	OK
3.33 ft	0.099 ksf	1.24	-0.62	0.478 ksf	0.394 ksf	0.146 ksf	1.117 ksf	1.404 klf	2.36 klf	OK
4.00 ft	0.118 ksf	1.07	-0.54	0.427 ksf	0.394 ksf	0.146 ksf	1.085 ksf	1.364 klf	2.36 klf	OK
4.67 ft	0.138 ksf	0.94	-0.47	0.383 ksf	0.394 ksf	0.146 ksf	1.061 ksf	1.334 klf	2.36 klf	OK
5.33 ft	0.158 ksf	0.84	-0.42	0.346 ksf	0.394 ksf	0.146 ksf	1.044 ksf	1.312 klf	2.36 klf	OK
6.00 ft	0.178 ksf	0.75	-0.38	0.315 ksf	0.394 ksf	0.146 ksf	1.033 ksf	1.298 klf	2.36 klf	OK
6.67 ft	0.197 ksf	0.68	-0.34	0.288 ksf	0.394 ksf	0.146 ksf	1.026 ksf	1.289 klf	2.36 klf	OK
7.33 ft	0.217 ksf	0.63	-0.31	0.265 ksf	0.394 ksf	0.146 ksf	1.023 ksf	1.285 klf	2.36 klf	OK
8.00 ft	0.237 ksf	0.58	-0.29	0.246 ksf	0.394 ksf	0.146 ksf	1.023 ksf	1.286 klf	2.36 klf	OK
8.67 ft	0.257 ksf	0.53	-0.27	0.229 ksf	0.394 ksf	0.146 ksf	1.025 ksf	1.289 klf	2.36 klf	OK
9.33 ft	0.276 ksf	0.50	-0.25	0.214 ksf	0.394 ksf	0.146 ksf	1.030 ksf	1.295 klf	2.36 klf	OK
10.00 ft	0.296 ksf	0.47	-0.23	0.200 ksf	0.394 ksf	0.146 ksf	1.037 ksf	1.303 klf	2.36 klf	OK
10.67 ft	0.316 ksf	0.44	-0.22	0.189 ksf	0.394 ksf	0.146 ksf	1.045 ksf	1.313 klf	2.36 klf	OK
11.33 ft	0.335 ksf	0.41	-0.21	0.178 ksf	0.394 ksf	0.146 ksf	1.054 ksf	1.325 klf	2.36 klf	OK
12.00 ft	0.355 ksf	0.39	-0.20	0.169 ksf	0.394 ksf	0.146 ksf	1.064 ksf	1.338 klf	2.36 klf	OK
12.67 ft	0.375 ksf	0.37	-0.19	0.160 ksf	0.394 ksf	0.146 ksf	1.076 ksf	1.352 klf	2.36 klf	OK
13.33 ft	0.395 ksf	0.35	-0.18	0.153 ksf	0.394 ksf	0.146 ksf	1.088 ksf	1.367 klf	2.36 klf	OK
14.00 ft	0.414 ksf	0.34	-0.17	0.146 ksf	0.394 ksf	0.146 ksf	1.100 ksf	1.383 klf	2.36 klf	OK
14.67 ft	0.434 ksf	0.32	-0.16	0.139 ksf	0.394 ksf	0.146 ksf	1.114 ksf	1.400 klf	2.36 klf	OK
15.33 ft	0.454 ksf	0.31	-0.15	0.133 ksf	0.394 ksf	0.146 ksf	1.128 ksf	1.417 klf	2.36 klf	OK
16.00 ft	0.474 ksf	0.29	-0.15	0.128 ksf	0.394 ksf	0.146 ksf	1.142 ksf	1.435 klf	2.36 klf	OK
16.67 ft	0.493 ksf	0.28	-0.14	0.123 ksf	0.394 ksf	0.146 ksf	1.157 ksf	1.454 klf	2.36 klf	OK
17.33 ft	0.513 ksf	0.27	-0.14	0.118 ksf	0.394 ksf	0.146 ksf	1.172 ksf	1.473 klf	2.36 klf	OK
18.00 ft	0.533 ksf	0.26	-0.13	0.114 ksf	0.394 ksf	0.146 ksf	1.187 ksf	1.492 klf	2.36 klf	OK
18.67 ft	0.552 ksf	0.25	-0.13	0.110 ksf	0.394 ksf	0.146 ksf	1.203 ksf	1.512 klf	2.36 klf	OK
19.33 ft	0.572 ksf	0.24	-0.12	0.106 ksf	0.394 ksf	0.146 ksf	1.219 ksf	1.532 klf	2.36 klf	OK
20.00 ft	0.592 ksf	0.24	-0.12	0.103 ksf	0.394 ksf	0.146 ksf	1.235 ksf	1.553 klf	2.36 klf	OK
20.67 ft	0.612 ksf	0.23	-0.11	0.100 ksf	0.394 ksf	0.146 ksf	1.252 ksf	1.573 klf	2.36 klf	OK
21.04 ft	0.623 ksf	0.22	-0.11	0.098 ksf	0.394 ksf	0.146 ksf	1.261 ksf	1.585 klf	2.36 klf	OK

The evaulation of the abutment for the service limit state is conducted according to (1) 4.3.7.3.2. and using the following equations:

The nominal required reinforcement strength:

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 50

The lateral pressure within the GRS Abutment at a given dpeth and location:

$$\sigma_h = \sigma_{h,W} + \sigma_{h,bridge,eq} + \sigma_{h,rb} + \sigma_{h,t}$$

(3) Eq. 51

The lateral pressure due to the equivalent bridge load:

$$\sigma_{h,bridge,eq} = \frac{(q_{DL} + q_{LL}) - (q_{rb} + q_t)}{\pi}\left[\alpha_b + \sin(\alpha_b)\cos(\alpha_b + 2\beta_b)\right]K_{ar} \quad (52)$$

Where:

q_{DL} = bridge DL pressure.
q_{LL} = bridge LL surcharge.
q_{rb} = surcharge due to the structural backfill of the integrated approach (i.e., road base).
q_t = roadway LL surcharge.

(3) Eq. 52

α_b = angle between wall face and projection of the midline of the bridge surcharge to the wall face found using equation 46 (see figure 23).
β_b = angle between the wall face and projection of the midline of the surcharge to the wall face found using equation 47 (see figure 23).
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

Lateral pressure due to the road base surcharge within GRS:

$$\sigma_{h,rb} = q_{rb} K_{ar}$$

(3) Eq. 53

Lateral pressure due to the traffic surchage within GRS:

$$\sigma_{h,t} = q_t K_{ar}$$

(3) Eq. 54

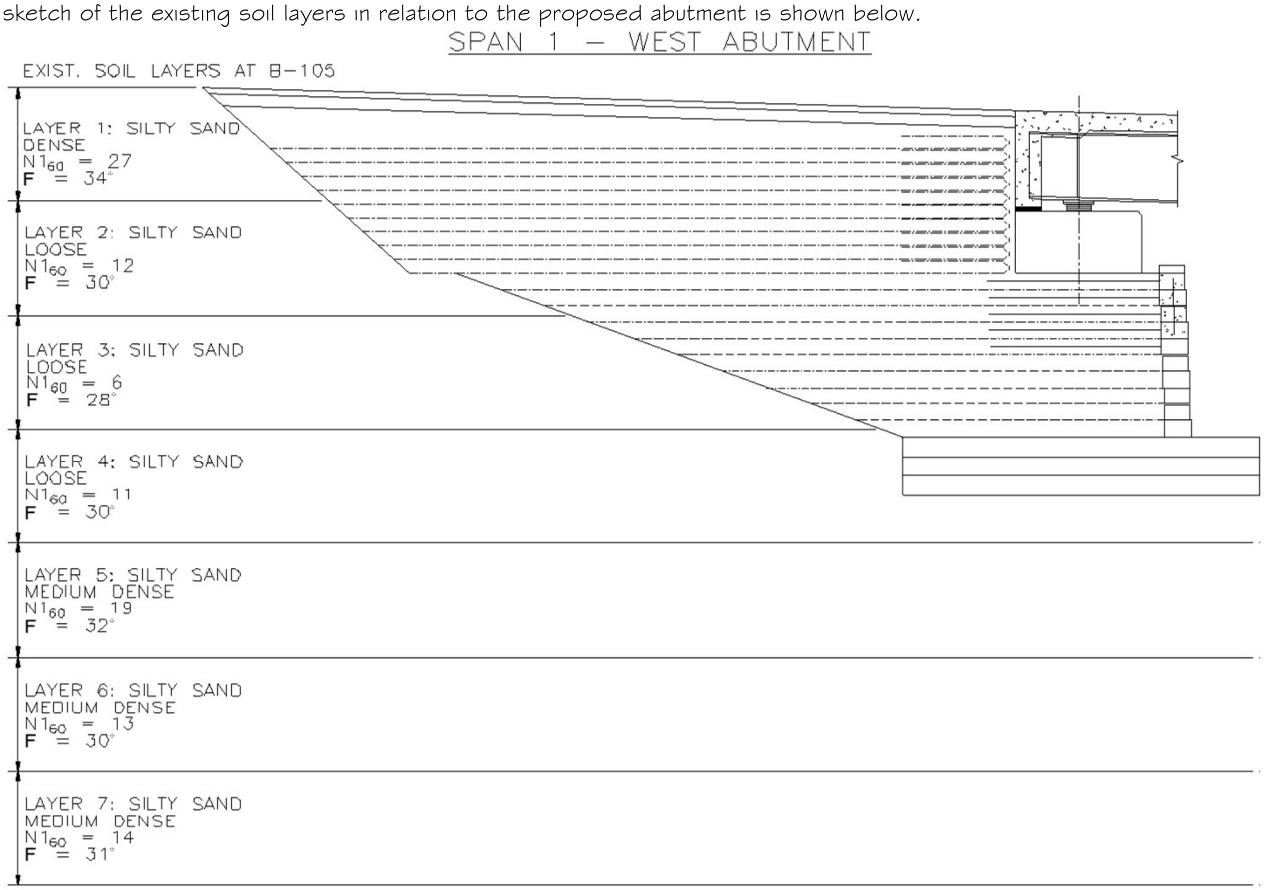
Service Limit State

z	$\sigma_{h,W}$	α_b	β_b	$\sigma_{h,bridge,eq}$	$\sigma_{h,rb}$	$\sigma_{h,t}$	σ_h	T_{req}	$T_{@\varepsilon = 2\%}$	
0.67 ft	0.013 ksf	2.59	-1.30	0.478 ksf	0.263 ksf	0.083 ksf	0.838 ksf	1.053 klf	1.25 klf	OK
1.33 ft	0.026 ksf	2.12	-1.06	0.456 ksf	0.263 ksf	0.083 ksf	0.829 ksf	1.042 klf	1.25 klf	OK
2.00 ft	0.039 ksf	1.74	-0.87	0.419 ksf	0.263 ksf	0.083 ksf	0.805 ksf	1.011 klf	1.25 klf	OK
2.67 ft	0.053 ksf	1.46	-0.73	0.376 ksf	0.263 ksf	0.083 ksf	0.775 ksf	0.974 klf	1.25 klf	OK
3.33 ft	0.066 ksf	1.24	-0.62	0.335 ksf	0.263 ksf	0.083 ksf	0.747 ksf	0.939 klf	1.25 klf	OK
4.00 ft	0.079 ksf	1.07	-0.54	0.299 ksf	0.263 ksf	0.083 ksf	0.725 ksf	0.911 klf	1.25 klf	OK
4.67 ft	0.092 ksf	0.94	-0.47	0.269 ksf	0.263 ksf	0.083 ksf	0.707 ksf	0.889 klf	1.25 klf	OK
5.33 ft	0.105 ksf	0.84	-0.42	0.243 ksf	0.263 ksf	0.083 ksf	0.694 ksf	0.873 klf	1.25 klf	OK
6.00 ft	0.118 ksf	0.75	-0.38	0.221 ksf	0.263 ksf	0.083 ksf	0.686 ksf	0.862 klf	1.25 klf	OK
6.67 ft	0.132 ksf	0.68	-0.34	0.202 ksf	0.263 ksf	0.083 ksf	0.680 ksf	0.855 klf	1.25 klf	OK
7.33 ft	0.145 ksf	0.63	-0.31	0.186 ksf	0.263 ksf	0.083 ksf	0.677 ksf	0.851 klf	1.25 klf	OK
8.00 ft	0.158 ksf	0.58	-0.29	0.172 ksf	0.263 ksf	0.083 ksf	0.677 ksf	0.850 klf	1.25 klf	OK
8.67 ft	0.171 ksf	0.53	-0.27	0.160 ksf	0.263 ksf	0.083 ksf	0.678 ksf	0.852 klf	1.25 klf	OK
9.33 ft	0.184 ksf	0.50	-0.25	0.150 ksf	0.263 ksf	0.083 ksf	0.680 ksf	0.855 klf	1.25 klf	OK
10.00 ft	0.197 ksf	0.47	-0.23	0.141 ksf	0.263 ksf	0.083 ksf	0.684 ksf	0.860 klf	1.25 klf	OK
10.67 ft	0.210 ksf	0.44	-0.22	0.132 ksf	0.263 ksf	0.083 ksf	0.689 ksf	0.866 klf	1.25 klf	OK
11.33 ft	0.224 ksf	0.41	-0.21	0.125 ksf	0.263 ksf	0.083 ksf	0.695 ksf	0.874 klf	1.25 klf	OK
12.00 ft	0.237 ksf	0.39	-0.20	0.118 ksf	0.263 ksf	0.083 ksf	0.702 ksf	0.882 klf	1.25 klf	OK
12.67 ft	0.250 ksf	0.37	-0.19	0.113 ksf	0.263 ksf	0.083 ksf	0.709 ksf	0.891 klf	1.25 klf	OK
13.33 ft	0.263 ksf	0.35	-0.18	0.107 ksf	0.263 ksf	0.083 ksf	0.717 ksf	0.901 klf	1.25 klf	OK
14.00 ft	0.276 ksf	0.34	-0.17	0.102 ksf	0.263 ksf	0.083 ksf	0.725 ksf	0.911 klf	1.25 klf	OK
14.67 ft	0.289 ksf	0.32	-0.16	0.098 ksf	0.263 ksf	0.083 ksf	0.733 ksf	0.922 klf	1.25 klf	OK
15.33 ft	0.303 ksf	0.31	-0.15	0.094 ksf	0.263 ksf	0.083 ksf	0.742 ksf	0.933 klf	1.25 klf	OK
16.00 ft	0.316 ksf	0.29	-0.15	0.090 ksf	0.263 ksf	0.083 ksf	0.752 ksf	0.945 klf	1.25 klf	OK
16.67 ft	0.329 ksf	0.28	-0.14	0.086 ksf	0.263 ksf	0.083 ksf	0.762 ksf	0.957 klf	1.25 klf	OK
17.33 ft	0.342 ksf	0.27	-0.14	0.083 ksf	0.263 ksf	0.083 ksf	0.771 ksf	0.970 klf	1.25 klf	OK
18.00 ft	0.355 ksf	0.26	-0.13	0.080 ksf	0.263 ksf	0.083 ksf	0.782 ksf	0.982 klf	1.25 klf	OK
18.67 ft	0.368 ksf	0.25	-0.13	0.077 ksf	0.263 ksf	0.083 ksf	0.792 ksf	0.995 klf	1.25 klf	OK
19.33 ft	0.381 ksf	0.24	-0.12	0.075 ksf	0.263 ksf	0.083 ksf	0.802 ksf	1.009 klf	1.25 klf	OK
20.00 ft	0.395 ksf	0.24	-0.12	0.072 ksf	0.263 ksf	0.083 ksf	0.813 ksf	1.022 klf	1.25 klf	OK
20.67 ft	0.408 ksf	0.23	-0.11	0.070 ksf	0.263 ksf	0.083 ksf	0.824 ksf	1.036 klf	1.25 klf	OK
21.04 ft	0.415 ksf	0.22	-0.11	0.069 ksf	0.263 ksf	0.083 ksf	0.830 ksf	1.043 klf	1.25 klf	OK

Final Design - Span 1 West Abutment - Settlement VTRANS D37 IM 091-1(68)

References:
(1) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018

The proposed Span 1 - West Abutment is assumed to be founded on the existing granular soil defined in boring B-105. A sketch of the existing soil layers in relation to the proposed abutment is shown below.



Note that the proposed bottom of footing elevation for the Span 1 - West Abutment is 451.46 ft. Therefore, the footing will be founded on layers of loose to medium dense silty sand.

Abutment Geometry

Footing Width, B =	16.00 ft			See "Abutment Design"
Abutment Length, L =	31.20 ft			See "Abutment Design"
e_B =	2.50 ft			See "Abutment Design"
B' =	11.00 ft =	16.00 ft -	(2.00 x	2.50 ft)

Settlement Analyses

Settlment Analyses per (1) 10.6.2.4

Total settlement, including elastic, consolidation, and secondary components may be taken as:

$S_t=S_e+S_c+S_s$ (Ref 1 - Eq. 10.6.2.4.1-1)

Final Design - Span 1 West Abutment - Settlement VTRANS D37 IM 091-1(68)

For cohesionless soils, only elastic settlement is typically considered, therefore $S_t = S_e$

Elastic settlement can be approximated using either the elastic half-space method or the empirical Hough method. Both methods are calculated here for reference.

Elastic Half-Space Method:

$$S_e = \frac{[q_o(1-v^2)\sqrt{A'}]}{144 E_s \beta_z}$$ (Ref 1 - Eq. 10.6.2.4.2-1)

Where:

q_o = applied vertical stress (ksf)

A' = effective area of footing (ft^2)

E_s = Young's modulus of soil taken from Article 10.4.6.3

B_z = shape factor taken as specified in table 10.6.2.4.2-1

v = Poisson's Ratio, taken as specified in Article 10.4.6.3

P_{SS} =	347 kips	(total vertical load on abutment, Service 1. See "Abutment Design")			
P_W =	944 kips =	30245 plf x	31.20 ft x	0.001	
P_{RSF} =	144 kips =	4600 plf x	31.20 ft x	0.001	
P_{FACE} =	57 kips =	1813.33 plf x	31.20 ft x	0.001	
P_{LL} =	224 kips				
P_{LS} =	55 kips =	250.00 psf x	7.08 ft x	31.20 ft x	0.001
P =	1770 kips				

A' = B'L' =	343 ft² =	11.00 ft x	31.20 ft
q_o = P/A' =	5.2 ksf =	1770 kips/	343 ft²

E_s =	4.17 ksi	(1) Table C10.4.6.3-1, Loose/Medium Dense Sand
v =	0.28	(1) Table C10.4.6.3-1, Loose/Medium Dense Sand

L/B =	2.8359553 =	31.20 ft/	11.00 ft
Footing Type =	Flexible		
βz =	1.123		

Se = 0.13 ft =
$$\frac{5.2\text{ ksf} \quad (1 - \quad 0.08)\ \text{x} \quad 18.53\text{ ft}}{(144\ \text{x} \quad 4.17\ \text{x} \quad 1.123)}$$

Se = 1.57 in

Final Design - Span 1 West Abutment - Settlement VTRANS D37 IM 091-1(68)

While the elastic half space method should provide reasonably accurate results, calculate total settlement using the Hough method for comparison.

Empirical Hough Method:

$$S_e = \sum_{i=1}^{n} \Delta H_i$$ (Ref 1 - Eq. 10.6.2.4.2-2)

Where: $$\Delta H_i = H_c \frac{1}{C'} \log\left(\frac{\sigma_o' + \Delta\sigma_v}{\sigma_o'}\right)$$ (Ref 1 - Eq. 10.6.2.4.2-3)

n = number of soil layers within zone of stress influence of the footing

H_c = initial height of each layer I (ft)

C' = bearing capacity index from Figure 10.6.2.4.2-1

σ'_o = initial vertical effective stress at the midpoint of layer I (ksf)

$\Delta\sigma_v$ = increase in vertical stress at the midpoint of layer I (ksf)

Depth of bottom of footing below grade, d = 14.54 ft See Soil Properties - B105
Water Table Depth = 23 ft
Soil Unit Weight = 0.125 kcf
Water Unit Weight = 0.062 kcf

Increase in vertical stress at point directly below footing is equal to the applied vertical stress, qo (ksf). Increase in vertical stress at a depth of z below the bottom of footing are assumed equal to applied load P divided by effective area at point of interest calculated assuming a 2:1 distribution slope, therefore A' = (B'+z)(L'+z)

Layer	Depth to Bottom of Layer	H_c	Midpoint Depth Below Footing, z	Sample Midpoint	h_1	h_2
1	2.00 ft	2.00 ft	0.00 ft	1.00	1.00 ft	0.00 ft
2	6.00 ft	4.00 ft	0.00 ft	4.00 ft	4.00 ft	0.00 ft
3	10.00 ft	4.00 ft	0.00 ft	8.00 ft	8.00 ft	0.00 ft
4	14.00 ft	4.00 ft	0.00 ft	12.00 ft	12.00 ft	0.00 ft
5	18.00 ft	4.00 ft	1.73 ft	16.00 ft	16.00 ft	0.00 ft
6	22.50 ft	4.50 ft	5.71 ft	20.25 ft	20.25 ft	0.00 ft
7	33.50 ft	11.00 ft	13.46 ft	28.00 ft	23.00 ft	5.00 ft
8	40.75 ft	7.25 ft	22.59 ft	37.13 ft	23.00 ft	14.13 ft

Final Design - Span 1 West Abutment - Settlement VTRANS D37 IM 091-1(68)

Layer	Depth to Bottom of Layer	σ'v	A'	Δσv (P/A')	N1 60	*C'	ΔH
1	2.00 ft	0.125 ksf	343 ft²	5.2 ksf	27		-
2	6.00 ft	0.500 ksf	343 ft²	5.2 ksf	12		-
3	10.00 ft	1.000 ksf	343 ft²	5.2 ksf	6		-
4	14.00 ft	1.500 ksf	343 ft²	5.2 ksf	11		-
5	18.00 ft	2.000 ksf	419 ft²	4.2 ksf	19	60	0.03 ft
6	22.50 ft	2.531 ksf	617 ft²	2.9 ksf	13	48	0.03 ft
7	33.50 ft	3.813 ksf	1092 ft²	1.6 ksf	14	50	0.03 ft
8	40.75 ft	5.525 ksf	1806 ft²	1.0 ksf	45	125	0.00 ft
						Total =	0.10 ft
						Total =	1.22 in

*In Figure 10.6.2.4.2-1 Clean well graded fine to coarse Sand was assumed

Soil Properties - Abutment 2 (B-102) VTRANS D37 IM 091-1(68)

References:
(1) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018
(2) Geotechnical Data Report, Teracon Consultants Inc., March 27. 2019

g_w = 0.062 kcf
g_{sat} = 0.125 kcf

Water Table = 7.00 ft (2)
Ground EL. At Boring = 452.00
BOF EL. = 451.46

Correct Blow Count for Overburden Pressure

	To Depth (ft)	h_1 (ft)	h_2 (ft)	σ'_v (ksf)	C_N	N blows/ft	$N1 = C_N N$ blows/ft	N_{60} = (ER/60%)N blows/ft	$N1_{60} = C_N N_{60}$ blows/ft
BOF	3.00	3.00	0.00	0.38	1.56	16.00	24.99	21.33	33.31
	7.00	7.00	0.00	0.88	1.28	13.00	16.62	17.33	22.16
	12.00	7.00	5.00	1.19	1.18	7.50	8.82	10.00	11.76
	19.00	7.00	12.00	1.63	1.07	11.00	11.78	14.67	15.71
	23.50	7.00	16.50	1.91	1.02	12.50	12.72	16.67	16.96
	33.50	7.00	26.50	2.53	0.92	14.50	13.38	19.33	17.84
	43.50	7.00	36.50	3.16	0.85	16.50	14.01	22.00	18.67

h_1 = depth above water table
h_2 = depth below water table
$\sigma'_v = g h_1 + g' h_2$
$g' = g_{sat} - g_w$
$C_N = 0.77 \log_{10} (40 / \sigma'_v) < 2$ (1) 10.4.6.2.4.-1

Correct for Hammer Efficiency

N_{60} = (ER/60%)N (1) 10.4.6.2.4-2
ER = 0.80
N_{60} = 1.33 = 0.80/ 0.60
$N1_{60} = C_N N_{60}$ (1) 10.4.6.2.4-3

Soil Properties - Abutment 2 (B-102) VTRANS D37 IM 091-1(68)

Calculate Drained Friction Angle

Table 10.4.6.2.4-1—Correlation of *SPT N*1$_{60}$ Values to Drained Friction Angle of Granular Soils (modified after Bowles, 1977)

$N1_{60}$	ϕ_f
<4	25–30
4	27–32
10	30–35
30	35–40
50	38–43

Use middle values of range:

$N1_{60}$	ϕ_f
<4	25.00
4.00	27.00
10.00	30.00
30.00	35.00
50.00	38.00

In determining the internal friction angle for soil below the footing, consider the soil up to two times the footing width.

Footing Width = 12.00 ft
Soil Depth = 24.00 ft = 2.00 x 12.00 ft

Footing Depth = 0.54 ft

	Depth (ft)	$N1_{60}$	$N1_{60\ low}$	$N1_{60\ high}$	$\phi_{f\ low}$	$\phi_{f\ high}$	ϕ_f	
BOF	3.00	33.31	30.00	50.00	35.00	38.00	35.50	Use ϕ_f = 35.00
	7.00	22.16	10.00	30.00	30.00	35.00	33.04	Use ϕ_f = 32.00
	12.00	11.76	10.00	30.00	30.00	35.00	30.44	
	19.00	15.71	10.00	30.00	30.00	35.00	31.43	
	23.50	16.96	10.00	30.00	30.00	35.00	31.74	

Bearing Capacity - Abutment 2 (B-105)

References:
(1) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018

Bearing Resistance of Soil (10.6.3.1)

Bearing resistance of soil calculated per (1) 10.6.3.1

$$q_n = cN_{cm}+\gamma D_f N_{qm} C_{wq}+0.5\gamma B N_{\gamma m} C_{w\gamma} \quad (1)\ 10.6.3.1.2a\text{-}1$$

Abutment Geometry

Footing width, B =	12.00	ft		See GRS Abutment Design
Footing length, L =	36.97	ft		See GRS Abutment Design
Embedment Depth, D_f =	4.00	ft		See GRS Abutment Design
Groundwater Depth, D_w =	7.00	ft		See GRS Abutment Design

Cohesive Term

ϕ_f =	0		
c =	0		
i_c =	1.00	(load inclination not considered)	(1) 10.6.3.1.2a-6
N_c =	0		(1) Table 10.6.3.1.2a-1

Surcharge Term (soil above footing)

ϕ_f =	35.00		
γ =	0.125	kcf	
D_f/B =	0.33		
$1.5B + D_f$ =	22.00		
i_q =	1.00	(load inclination not considered)	(1) 10.6.3.1.2a-7
N_q =	33.30		(1) Table 10.6.3.1.2a-1
C_{wq} =	1.00		(1) Table 10.6.3.1.2a-2
d_q =	1.00		(1) Table 10.6.3.1.2a-4

Unit Weight Term (soil below footing)

ϕ_f =	32.00		
γ =	0.125	kcf	
$1.5B + D_f$ =	22.00		
i_γ =	1.00	(load inclination not considered)	(1) 10.6.3.1.2a-7
N_γ =	30.20		(1) Table 10.6.3.1.2a-1
$C_{w\gamma}$ =	0.50		(1) Table 10.6.3.1.2a-2

Bearing Capacity - Abutment 2 (B-105)

Considerations for footings on slope

Is footing bearing on or near slope ? no

$q_{n\text{-sloping ground}} = RC_{BC}(cN_c+0.5\gamma BN_\gamma)$ (1) 10.6.3.1.2c-1

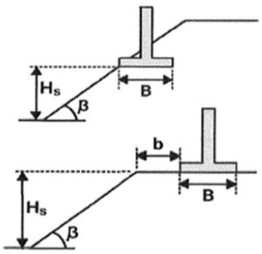
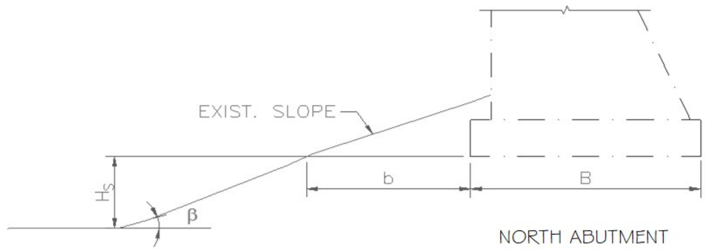


Figure 10.6.3.1.2c-1—Definition of Footing and Slope Geometric Parameters for Determination of RC_{BC}



Height of Slope, H_s = 0.00 ft

b = 0.00 ft

Angle of slope, β = 0.00 degrees

γ = 0.125 kcf

N_s = c'=0

ϕ = 32.00

For β = 30 degrees, c' = 0

B/H_s	b/B	RC_{BC}	
2	0.50	0.64	(1) Table 10.6.3.1.2c-1
2	1.25	0.74	(1) Table 10.6.3.1.2c-1

Factored Bearing Resistance:

Strength, ϕ_b = 0.45 (1) 10.5.5.2.2

Extreme and Service, ϕ_b = 1 (1) 10.5.5.3.3

For footing w/o slope

$q_n = cN_{cm}+\gamma D_f N_{qm} C_{wq}+0.5\gamma BN_{\gamma m} C_{w\gamma}$ (1) 10.6.3.1.2a-1

$N_{cm} = N_c s_c i_c$ (1) 10.6.3.1.2a-2

$N_{qm} = N_q s_q d_q i_q$ (1) 10.6.3.1.2a-3

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ (1) 10.6.3.1.2a-4

$s_c = 1+(B/L)(N_q/N_c)$ (1) Table 10.6.3.1.2a-3

$s_q = 1+B/L(\tan\phi_f)$ (1) Table 10.6.3.1.2a-3

$s_\gamma = 1-0.4(B/L)$ (1) Table 10.6.3.1.2a-3

For footing w/ slope

$q_{n\text{-sloping ground}} = RC_{BC}(cN_c+0.5\gamma BN_\gamma C_{w\gamma})$ (1) 10.6.3.1.2c-1

Bearing Capacity - Abutment 2 (B-105)

Limit State	e_B (ft)	B' (ft)	Shape Correction Factor s_c	s_q	s_γ	Bearing Capacity Factor N_{cm}	N_{qm}	$N_{\gamma m}$	Slope Consideration B'/H_s	b/B'	RC_{BC}	Bearing Capacity q_n (ksf)	$\phi_b q_n$ (ksf)
Str I - A	1.67	8.67	0.00	1.16	0.91	0.00	38.77	27.37	n/a	n/a	n/a	26.79	12.06

FINAL DESIGN - GRS-IBS ABUTMENT - ABUTMENT 2 DESIGN — VTRANS D37 IM 091-1(68)

References:

1) GRS-IBS Interim Implementation Guide, FHWA, June 2018
2) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018

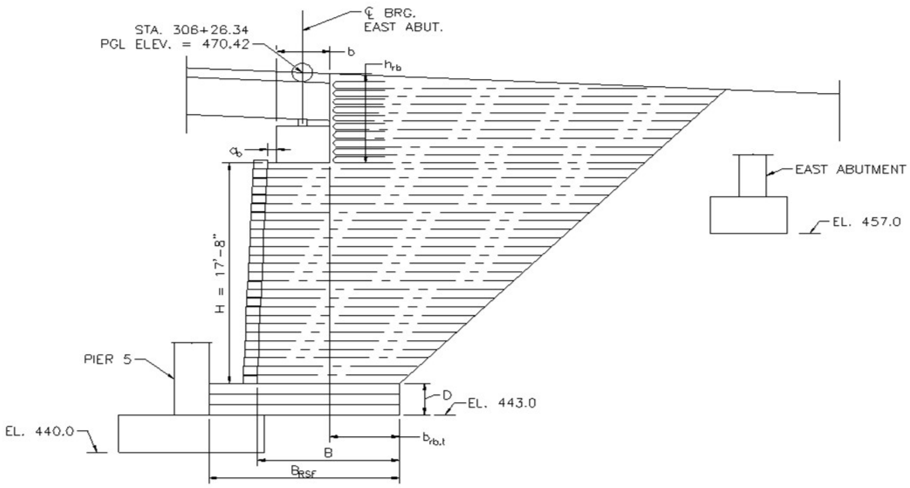
Material Properties

γ_f =	115.00 pcf	(Reinforced Fill)
ϕ (Reinforced Soil) =	45	degrees
K_a (Reinforced Soil) =	0.17 =	$\tan^2($ 45 - (45/ 2))

Assume that retained fill at East Abutment, Span 1 will be imported and of a higher quality than in-situ soil.

γ_b =	125.00 pcf	(Retained Fill)
ϕ (Existing Filll) =	34.00	degrees
K_{ab} (Existing Fill) =	0.28 =	$\tan^2($ 45 - (34/ 2))

Geometry



Bridge Span Length =	85.00 ft	
Abut. Height =	18.04 ft	
H =	18.04 ft	
Superstructure Depth =	4.39 ft	
h_{rb} =	6.88 ft	
B =	8.5 ft	
b =	4.75 ft	
Toe Length, X_{RSF} =	2.50 ft	
Total Width, B_{RSF} =	12 ft	
$b_{rb,t}$ =	3.08 ft =	8.5 ft - 0.67 ft - 4.75 ft
Depth of RSF, D =	2.50 ft	
Setback Distance, a_b =	0.67 ft	
L =	31.97 ft	
h_{block} =	8.00 in	
D_{block} =	12.00 in	
L_{block} =	18.00 in	
Weight =	85 lbs	per block

Reinforcement

S_v =	0.67 ft
d_{max} =	0.75 in
T_f =	5.90 klf

Load Factors

Per (3) Table 3.4.1-1 and Table 3.4.1-2.

γ_p (DC, Min.) =	0.90	3) Table 3.4.1-2
γ_p (DC, Max.) =	1.25	3) Table 3.4.1-2
γ_p (DW, Min.) =	0.65	3) Table 3.4.1-3
γ_p (DW, Max.) =	1.50	3) Table 3.4.1-4
γ_p (EH, Max) =	1.50	3) Table 3.4.1-2
γ_p (EH, Min.) =	0.90	3) Table 3.4.1-2
γ_p (EV, Min.) =	1.00	3) Table 3.4.1-2
γ_p (EV, Max.) =	1.35	3) Table 3.4.1-2
Factor (LS) =	1.75	3) Table 3.4.1-1
Factor (LL) =	1.75	3) Table 3.4.1-1

Calculate Average Height of Precast Beam Seat

Min. Height of Cap =	1.50 ft			
Cross Slope =	0.0625			
Cap Length =	31.97 ft			
Max. Height =	3.50 ft =	1.50 ft +	(31.97 ft x	0.0625)
Average Height =	2.50 ft			

Calculate Vertical Loads and Applied Pressures

Calculate q_{DL}

$$q_{DL} = \frac{Q_{DL}}{bL}$$

	Total DC Rxn	No. of Girders	DC1	DC2
N. Ext.	46.34 k =	1 x	(41.18 k +	5.16 k)
Typ. Int.	143.73 k =	3 x	(42.75 k +	5.16 k)
S. Ext.	42.49 k =	1 x	(37.33 k +	5.16 k)
Total Girder DC Reaction =	232.56 k			

	Total DW Rxn	No. of Girders	DW
N. Ext.	7.09 k =	1 x	7.09 k)
Typ. Int.	21.27 k =	3 x	7.09 k)
S. Ext.	7.09 k =	1 x	7.09 k)
Total Girder DW Reaction =	35.45 k		

Assume a concrete end block integral with the steel beam that is 3'-1" high x 12" wide x full width of abutment.

End Block Weight =	14.79 k =	3.083 ft x	1.00 ft x	31.97 ft x	0.150 kcf

Assume concrete deck beyond CL of bearing is equal to 1.60 SF, per AutoCAD

Additional Deck Weight =	7.67 k =	1.60 ft² x	0.150 kcf x	31.97 ft

Assume that girder extends additional 2'-0" beyond the CL of Bearing

Top Flange =	0.10 ft² =	16 in x	0.875 in x	0.007
Web =	0.12 ft² =	34 in x	0.5 in x	0.007
Bottom Flange =	0.14 ft² =	16 in x	1.25 in x	0.007
Total Steel Area =	0.35 ft²			

Additional Weight of Steel =	1.74 k =	0.35 ft² x	0.49 kcf x	2.00 ft x	5 girders	
Weight of Beam Seat =	56.92 k =	2.50 ft x	4.75 ft x	31.97 ft x	0.15 kcf	
Total DC Reaction (Unfactored) =	313.68 k =	232.56 k +	14.79 k +	7.67 k +	1.74 k +	56.92 k
Total DW Reaction (Unfactored) =	35.45 k					
q_{DC} =	2065.67 psf =	(313.676 kips x	1000)/	(4.75 ft x	31.97 ft)	
q_{DW} =	233.45 psf =	(35.45 kips x	1000)/	(4.75 ft x	31.97 ft)	

Calculate q_{LL}

Calculate Live Load Reactions Per Barrel

Roadway Width = 27.00 ft
Number of Trucks = 2
Impact = 1.33

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

Reaction (Single Truck) = 64.09 kips = [(32 kips x 85.00 ft) + (32 kips x 71.00 ft) + (8 kips x 57.00 ft)] / 85.00 ft

Lane Load Reaction (Single Truck) = 27.2 kips = 0.5 x 0.64 klf x 85.00 ft

Reaction - 1 Lane Loaded = 134.93 kips = 1.2 x ((64.09 kips x 1.33) + 27.2)
Reaction - 2 Lanes Loaded = 224.89 kips = 1 x ((2 x 64.09 kips x 1.33) + (2 x 27 kips))
Max. Reaction = 224.89 kips

Bridge LL Reaction = 224.89 kips

$$q_{LL} = \frac{Q_{LL}}{bL}$$

q_{LL} = 1480.99 psf = (224.89 kips x 1000)/ (4.75 ft x 31.97 ft)

Calculate Traffic Surcharge, q_T

Per (3) 3.11.6.4-1, the equivalent height of soil acting as a surcharge load shall be determined as follows:

Table 3.11.6.4-1—Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic

Abutment Height (ft)	h_{eq} (ft)
5.0	4.0
10.0	3.0
≥20.0	2.0

Note that linear interpolation shall be used for intermediate wall heights.

Total H = 20.54 ft
h_{eq} = 2.00 ft
γ_r = 125.00 pcf

$$q_t = h_{eq}\gamma_b$$

q_T = 250.00 psf = 2.00 ft x 125.00 pcf

Calculate Road Base Surchage, q_{rb}

γ_{rb} = 115.00 pcf

$$q_{rb} = H_{rb}\gamma_{rb}$$

q_{rb} = 791.71 psf = 6.88 ft x 115.00 pcf

Calculate Weight of GRS Abutment

$$W = BH\gamma_r$$

H = 18.04 ft
B = 8.5 ft
γ_r = 115.00 pcf

W = 17634.10 plf = 18.04 ft x 8.5 ft x 115.00 pcf

Calculate Weight of RSF

$$W_{RSF} = B_{RSF} D_{RSF} \gamma_r$$

B = 12 ft
D = 2.5 ft
γ_r = 125.00 pcf

W_{RSF} = 3450.00 plf = 12 ft x 2.5 ft x 115.00 pcf

Calculate Weight of Facing

N_{block} = 27.06 = 18.04 ft/ 0.67 ft
N_{block} = 28

$$W_{face} = N_{block} \frac{W_{block}}{L_{block}}$$

W_{face} = 1586.67 plf = 28 x (85 lbs/ (18.00 in/ 12)

Summary of Applied Loads

q_{DC} =	2065.67 psf
q_{DW} =	233.45 psf
q_{LL} =	1480.99 psf
q_t =	250.00 psf
q_{rb} =	791.71 psf
W =	17634.10 plf
W_{RSF} =	3450.00 plf
W_{face} =	1586.67 plf

Check Beam Seat Pressure

Per (1) 4.3.5.4, the service bearing pressure should be targeted to around 4 ksf.

DC Reaction =	313.68 kips
DW Reaction =	35.45 kips
LL Reaction =	224.89 kips
Total =	574.02 kips

b = 4.75 ft
L = 31.97 ft

q_{seat} = 3.78 ksf = 574.02 kips/ (4.75 ft x 31.97 ft)

OK

Calculte Direct Sliding Effects at RSF/GRS Interface

Calculate Sliding per the (1) and (3) Strength I Load Combination

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2$$ (1) Eq. 9

$$F_{rb} = q_{rb} K_{ab} H$$ (1) Eq. 10

$$F_t = q_t K_{ab} H$$ (1) Eq. 11

$$F_R = \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t$$ (1) Eq. 12

F_b = 5750.45 lbs = 0.5 x 125.00 pcf x 0.28 x 18.04 ft^ 2
F_{rb} = 4037.86 lbs = 791.71 psf x 0.28 x 18.04 ft
F_t = 1275.04 lbs = 250.00 psf 0.28 x 18.04 ft

F_R = 16.91 klf = (1.5 x (5750.45 lbs + 4037.86 lbs) + (1.75 1275.04 lbs))/ 1000

$$W_{T,R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EV\,MIN}(q_{rb} b_{rb,t})$$ (1) Eq. 14

$W_{T,R}$ = 31.05 klf = ((1.0 x 17634.10 plf) + (0.9 x 2065.67 psf x 4.75 ft) + (0.65 x 233.45 psf x 4.8 ft) +
(0.9 x 1586.67 plf) + (1.0 x 791.71 psf x 3.08 ft))/ 1000

Assume that μ = 2/3*tanϕ, per (1) C.1.6.1.1

μ = 0.67 = 0.67 x tan(45)

$$R_R = \Phi_\tau\left(W_{T,R}\mu\right)$$ (1) Eq. 13

R_R = 20.70 klf = 31.05 klf x 0.667

OK

Calculate Direct Sliding at Base of RSF

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2$$ (3) Eq. 17

$$F_{rb,RSF} = q_{rb}K_{ab}(H + D_{RSF})$$ (3) Eq. 18

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF})$$ (3) Eq. 19

$F_{b,\ RSF}$ = 7.45 klf = (0.5 x 125.00 pcf x 0.28 x (18.04 ft + 2.5)^ 2)/ 1000
$F_{rb,\ RSF}$ = 4.60 klf = (791.71 psf x 0.28 x (18.04 ft + 2.50))/ 1000
$F_{t,\ RSF}$ = 1.45 klf = 250.00 psf x 0.28 x (18.04 ft + 2.5 ft))/ 1000

$$F_{R,RSF} = \gamma_{EH\ MAX}\left(F_{b,RSF} + F_{rb,RSF}\right) + \gamma_{LS}F_{t,RSF}$$ (1) Eq. 20

$F_{R,RSF}$ = 20.62 klf = (1.5 x (7.45 klf + 4.60 klf)) + (1.75 x 1.45 klf)

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\ MIN}W_{RSF}$$

$W_{T,R,RSF}$ = 34.50 klf = 31.05 klf + (1.0 x 3.45 klf)

$$R_{R,RSF} = \Phi_\tau(W_{T,R,RSF}\mu_{RSF})$$ (1) Eq. 21

Note that ϕ = 1.0, per (1) 4.3.6.1.2
Per (2), and the previous calculation, the angle of internal friction beneath the RSF is equal to 32 degrees

$R_{R,\ RSF}$ = 21.56 klf = 1.0 x 34.50 klf x tan(32)

OK

Calculate External Bearing Resistance

Calculate V_R

Per (1) Eq. 26,

$$\sum V_R = \gamma_{EV\ MAX}(W) + \gamma_{EV\ MAX}(W_{RSF}) + \gamma_{DC\ MAX}(W_{face}) + \gamma_{LS}(q_t b_{rb,t}) + \gamma_{EH\ MAX}(q_{rb}b_{rb,t}) + \gamma_{DC\ MAX}(q_{DL}b) + \gamma_{LS}(q_{LL}b)$$

Note that $\gamma_{EV\ Max}$ is used in place of $\gamma_{EH\ Max}$ to factor q_{rb}*$b_{rb,t}$ since this is a summation of vertical reactions.

W =	23.81 klf =	1.35 x	17634.10 plf/	1000	
W_{RSF} =	4.66 klf =	1.35 x	3450.00 plf/	1000	
W_{FACE} =	1.98 klf =	1.25 x	1586.67 plf/	1000	
Q_t =	1.35 klf =	1.75 x	(250.00 psf x	3.08 ft)/	1000
Q_{rb} =	3.30 klf =	1.35 x	(791.71 psf x	3.08 ft)/	1000
Q_{DC} =	12.26 klf =	1.25 x	(2065.67 psf x	4.75 ft)/	1000
Q_{DW}=	1.66 klf =	1.5 x	(233.45 psf x	4.75 ft)/	1000
Q_{LL} =	12.31 klf =	1.75 x	(1480.99 psf	4.75 ft)/	1000
Total =	61.33 klf				

Calculate $M_{D,R}$

Per (1) Eq. 28

$$\sum M_{D,R} = \gamma_{EH\,MAX} F_{b,RSF}\left(\frac{H+D_{RSF}}{3}\right) + \gamma_{LS} F_{t,RSF}\left(\frac{H+D_{RSF}}{2}\right) + \gamma_{EH\,MAX} F_{rb,RSF}\left(\frac{H+D_{RSF}}{2}\right)$$

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H+D_{RSF})^2 \qquad \text{(3) Eq. 17}$$

$$F_{rb,RSF} = q_{rb} K_{ab}(H+D_{RSF}) \qquad \text{(3) Eq. 18}$$

$$F_{t,RSF} = q_t K_{ab}(H+D_{RSF}) \qquad \text{(3) Eq. 19}$$

$F_{b,\ RSF}$ =	7.45 klf =	(0.5 x	125.00 pcf x	0.28 x	(18.04 ft +	2.5)^ 2)/		1000
$F_{rb,\ RSF}$ =	4.60 klf =	791.71 psf x	0.28 x	(18.04 ft +	2.5 ft))/	1000		
$F_{t,\ RSF}$ =	1.45 klf =	250.00 psf x	0.28 x	(18.04 ft +	2.5 ft))/	1000		
M, $F_{b,\ RSF}$ =	76.56 k-ft/ ft =	1.5 x	7.45 klf x	(0.33 x	(18.04 ft +	2.5 ft))		
M, $F_{t,\ RSF}$ =	26.09 k-ft/ ft =	1.75 x	1.45 klf x	(0.50 x	(18.04 ft +	2.5 ft))		
M, $F_{rb,\ RSF}$ =	70.82 k-ft/ ft =	1.5 x	4.60 klf x	(0.50 x	(18.04 ft +	2.5 ft))		
Total =	173.47 k-ft/ ft							

Note that $M_{D,R}$ is taken about the bottom center of the width of the RSF.

Calculate $M_{R,R}$

Per (1) Eq. 29

$$\sum M_{R,R} = (\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b)\left[\left(\frac{b}{2}+a_b\right)-\left(\frac{B_{RSF}}{2}-x_{RSF}-b_{block}\right)\right] +$$
$$(\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t})\left(\frac{B_{RSF}}{2}-\frac{b_{rb}}{2}\right) + \gamma_{EV\,MAX} W\left(\frac{B_{RSF}}{2}-\frac{B}{2}\right) +$$
$$\gamma_{DC\,MAX} W_{face}\left(B+\frac{b_{block}}{2}-\frac{B_{RSF}}{2}\right)$$

$Q_{DL}+Q_{LL}$ =	14.21 k-ft =	((1.25 x	2.07 ksf) +	(1.50 x	0.23 ksf) +	(1.75 x	1.48 ksf)) x 4.75 ft x
		((2.375 ft x +	0.67 ft) -	(6 ft -	2.5 ft -	1.00 ft))	
Q_t+Q_{rb} =	20.71 k-ft =	((1.75 x	0.25 klf x	3.08 ft) +	(1.35 x	0.792 klf x	3.08 ft)) x
		(6 ft -	1.54 ft)				
W =	41.66 k-ft =	1.35 x	17.63 klf x	(6 ft -	4.25 ft)		
W_{Face} =	-5.36 k-ft =	1.35 x	1.59 klf x	((2.5 ft +	1 ft) -	6 ft)	
Total =	71.22 k-ft/ ft						

Note that $M_{R,R}$ is taken about the bottom center of the width of the RSF.

Calculate Eccentricity, $e_{B,R}$

Per (1) Eq. 27

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R}$$

$$e_{B,R} = 1.67\text{ ft} = \frac{173.47\text{ k-ft/ ft} - 71.22\text{ k-ft/ ft}}{61.33\text{ klf}}$$

Calculate $\sigma_{v,base,R}$

Since $e_{B,R}$ = negative, set equal to zero in calcaulation of base bearing pressure.

Per (1) Eq. 25,

$$\sigma_{v,base,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}}$$

$\sigma_{v,base,R}$ = 7.08 ksf = 61.33 klf/ (12 ft - (2 x 1.67 ft)

Per bearing capacity calculation, factored bearing capacity = 12.06 ksf

OK

Calculate Internal Bearing Resistance

Per (1), Eq. 35

$$q_{n,an} = \left[0.7^{\left(\frac{S_v}{6d_{max}}\right)} \frac{T_f}{S_v}\right] K_{pr}$$

$q_{n,an}$ = nominal bearing resistance of the GRS abutment using the analytical method.
S_v = reinforcement spacing.
d_{max} = maximum grain size.
T_f = ultimate reinforcement strength.
K_{pr} = coefficient of passive earth pressure for the reinforced fill (calculated in equation 35).

K_{pr} =	5.83 =	$\tan^2$(		45 +	(45/	2))			
$q_{n,an}$ =	27.36 ksf =	[(0.7 ^	(0.67 ft/	(6 x	0.06 ft))) x	(5.90 klf/	0.67 in)] x	5.83	
ϕ_{cap} =	0.45	(3) 4.3.7.2							
$\phi_{cap} q_{n,an}$ =	12.31 ksf =	0.45 x	27.36 ksf						

$$V_{applied,f} = \gamma_{DC\ MAX} q_{DL} + \gamma_{LL} q_{LL}$$

(1) Eq. 32

$V_{app,f}$ =	5.52 ksf =	(1.25 x	2.07 ksf) +	(1.50 x	0.23 ksf) +	(1.75 x	1.48 ksf)

OK

Calculate and Check Deformations

Per (1), Eq. 37

$$q_{DL,allow\ @\varepsilon=1\%} = 0.2\left[0.7^{S_v/6d_{max}}\left(\frac{T_f}{S_v}\right)K_{pr}\right]$$

6^*d_{max} =	0.375 ft =	6 x	0.75 in x	0.083		
T_f/S_v =	8.85 =	5.90 klf/	0.67 ft			
$S_v/6^*d_{max}$ =	1.78 =	0.67 ft/	0.375 ft			
$q_{dl,\ all}$ =	5.47 ksf =	0.2	((0.7 ^	1.78 in) x	8.85 x	5.83 in)
q_{DL} =	2.07 ksf					

OK

Calculate Lateral Strain

Per (1) 4.3.7.2.2, for the calculation of lateral strain, assume a vertical strain equal to 1% of the abutment height, H.

0.01*H = 0.18 ft = 0.01 x 18.04 ft

$$D_L = \frac{2b_q D_v}{H}$$

(1) Eq. 38

D_L =	0.01 ft =	(2 x	(4.75 ft +	0.67 ft) x	0.02 ft)/	18.04 ft
D_L =	0.11 in =	0.01 ft x	12			

Per (1) 4.3.7.2.2, the total lateral strain should be limited to twice the vertical strain.

Vertical Strain = 0.36 ft = 2 x 0.18 ft

OK

Calculate Reinforcement Strength

q_{DC} = 2065.67 psf
q_{DW} = 233.45 psf
q_{LL} = 1480.99 psf
q_t = 250.00 psf
q_{rb} = 791.71 psf
W = 17634.10 psf
W_{RSF} = 3450.00 psf
W_{face} = 1586.67 psf

The evaluation of the abutment for the strength limit state is conducted according to (1) 4.3.7.3.1. and using the following equations:

Required Reinforcment:

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$

(3) Eq. 40

Factored Total Lateral Pressure:

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f}$$

(3) Eq. 41

Factored Lateral Pressure due to weight of GRS:

$$\sigma_{h,W,f} = \gamma_{EH\ MAX}(\gamma_r z K_{ar})$$

Where:
$\gamma_{EH\ MAX}$ = maximum horizontal earth pressure load factor.
γ_r = unit weight of reinforced backfill.
z = depth from the top of the wall.
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

(3) Eq. 42

Factored Latreal Pressure due to equivalent bridge load:

$$\sigma_{h,bridge,f} = \frac{(\gamma_{DC\ MAX} q_{DL} + \gamma_{LL} q_{LL}) - (\gamma_{EH\ MAX} q_{rb} + \gamma_{LS} q_t)}{\pi}\left[\alpha_b + \sin(\alpha_b)\cos(\alpha_b) + 2\beta_b\right]K_{ar} \qquad (43)$$

Where:

$\gamma_{DC\ MAX}$ = maximum DL load factor.
q_{DL} = superstructure DL pressure.
γ_{LL} = bridge LL surcharge load factor.
q_{LL} = bridge LL pressure.
q_{rb} = surcharge due to the structural backfill of the integrated approach (i.e., road base).
γ_{LS} = LL surcharge load factor.
q_t = roadway LL surcharge.
α_b = angle between wall face and projection of the midline of the bridge surcharge to the wall face found using equation 46 (see figure 23).
β_b = angle between the projections of the inner and outer edge lines of the bridge surcharge to the wall face found using equation 47 (see figure 23).

(3) Eq. 43

$$\sigma_{h,rb,f} = \gamma_{EH\ MAX} q_{rb} K_{ar}$$

(3) Eq. 44

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar}$$

(3) Eq. 45

$$\alpha_b = \tan^{-1}\left(\frac{b}{2z}\right) - \beta_b$$

(3) Eq. 46

$$\beta_b = \tan^{-1}\left(\frac{-b}{2z}\right)$$

(3) Eq. 47

Factored Reinforcement Strength:

$$T_{f,f} = \Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right) = 0.4T_f$$

(3) Eq. 48

Strength Limit State

z	$\sigma_{h,W,f}$	α_b	β_b	$\sigma_{h,bridge,f}$	$\sigma_{h,rb,f}$	$\sigma_{h,t,f}$	$\sigma_{h,f}$	$T_{req,f}$	$T_{f,f}$	
0.67 ft	0.020 ksf	2.59	-1.30	0.663 ksf	0.336 ksf	0.124 ksf	1.142 ksf	1.436 klf	2.36 klf	OK
1.33 ft	0.039 ksf	2.12	-1.06	0.633 ksf	0.336 ksf	0.124 ksf	1.132 ksf	1.422 klf	2.36 klf	OK
2.00 ft	0.059 ksf	1.74	-0.87	0.581 ksf	0.336 ksf	0.124 ksf	1.099 ksf	1.382 klf	2.36 klf	OK
2.67 ft	0.079 ksf	1.46	-0.73	0.521 ksf	0.336 ksf	0.124 ksf	1.060 ksf	1.332 klf	2.36 klf	OK
3.33 ft	0.099 ksf	1.24	-0.62	0.465 ksf	0.336 ksf	0.124 ksf	1.023 ksf	1.286 klf	2.36 klf	OK
4.00 ft	0.118 ksf	1.07	-0.54	0.415 ksf	0.336 ksf	0.124 ksf	0.993 ksf	1.248 klf	2.36 klf	OK
4.67 ft	0.138 ksf	0.94	-0.47	0.373 ksf	0.336 ksf	0.124 ksf	0.970 ksf	1.219 klf	2.36 klf	OK
5.33 ft	0.158 ksf	0.84	-0.42	0.337 ksf	0.336 ksf	0.124 ksf	0.954 ksf	1.199 klf	2.36 klf	OK
6.00 ft	0.178 ksf	0.75	-0.38	0.306 ksf	0.336 ksf	0.124 ksf	0.943 ksf	1.186 klf	2.36 klf	OK
6.67 ft	0.197 ksf	0.68	-0.34	0.280 ksf	0.336 ksf	0.124 ksf	0.937 ksf	1.178 klf	2.36 klf	OK
7.33 ft	0.217 ksf	0.63	-0.31	0.258 ksf	0.336 ksf	0.124 ksf	0.935 ksf	1.175 klf	2.36 klf	OK
8.00 ft	0.237 ksf	0.58	-0.29	0.239 ksf	0.336 ksf	0.124 ksf	0.935 ksf	1.176 klf	2.36 klf	OK
8.67 ft	0.257 ksf	0.53	-0.27	0.222 ksf	0.336 ksf	0.124 ksf	0.938 ksf	1.179 klf	2.36 klf	OK
9.33 ft	0.276 ksf	0.50	-0.25	0.208 ksf	0.336 ksf	0.124 ksf	0.944 ksf	1.186 klf	2.36 klf	OK
10.00 ft	0.296 ksf	0.47	-0.23	0.195 ksf	0.336 ksf	0.124 ksf	0.950 ksf	1.195 klf	2.36 klf	OK
10.67 ft	0.316 ksf	0.44	-0.22	0.184 ksf	0.336 ksf	0.124 ksf	0.959 ksf	1.205 klf	2.36 klf	OK
11.33 ft	0.335 ksf	0.41	-0.21	0.173 ksf	0.336 ksf	0.124 ksf	0.968 ksf	1.217 klf	2.36 klf	OK
12.00 ft	0.355 ksf	0.39	-0.20	0.164 ksf	0.336 ksf	0.124 ksf	0.979 ksf	1.230 klf	2.36 klf	OK
12.67 ft	0.375 ksf	0.37	-0.19	0.156 ksf	0.336 ksf	0.124 ksf	0.990 ksf	1.245 klf	2.36 klf	OK
13.33 ft	0.395 ksf	0.35	-0.18	0.149 ksf	0.336 ksf	0.124 ksf	1.003 ksf	1.260 klf	2.36 klf	OK
14.00 ft	0.414 ksf	0.34	-0.17	0.142 ksf	0.336 ksf	0.124 ksf	1.016 ksf	1.276 klf	2.36 klf	OK
14.67 ft	0.434 ksf	0.32	-0.16	0.136 ksf	0.336 ksf	0.124 ksf	1.029 ksf	1.293 klf	2.36 klf	OK
15.33 ft	0.454 ksf	0.31	-0.15	0.130 ksf	0.336 ksf	0.124 ksf	1.043 ksf	1.311 klf	2.36 klf	OK
16.00 ft	0.474 ksf	0.29	-0.15	0.125 ksf	0.336 ksf	0.124 ksf	1.058 ksf	1.329 klf	2.36 klf	OK
16.67 ft	0.493 ksf	0.28	-0.14	0.120 ksf	0.336 ksf	0.124 ksf	1.072 ksf	1.348 klf	2.36 klf	OK
17.33 ft	0.513 ksf	0.27	-0.14	0.115 ksf	0.336 ksf	0.124 ksf	1.088 ksf	1.367 klf	2.36 klf	OK
18.04 ft	0.534 ksf	0.26	-0.13	0.111 ksf	0.336 ksf	0.124 ksf	1.104 ksf	1.388 klf	2.36 klf	OK

The evaulation of the abutment for the service limit state is conducted according to (1) 4.3.7.3.2. and using the following equations:

The nominal required reinforcement strength:

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 50

The lateral pressure within the GRS Abutment at a given dpeth and location:

$$\sigma_h = \sigma_{h,W} + \sigma_{h,bridge,eq} + \sigma_{h,rb} + \sigma_{h,t}$$ (3) Eq. 51

The lateral pressure due to the equivalent bridge load:

$$\sigma_{h,bridge,eq} = \frac{(q_{DL} + q_{LL}) - (q_{rb} + q_t)}{\pi}\left[\alpha_b + \sin(\alpha_b)\cos(\alpha_b + 2\beta_b)\right]K_{ar} \qquad (52)$$

Where:

q_{DL} = bridge DL pressure.
q_{LL} = bridge LL surcharge.
q_{rb} = surcharge due to the structural backfill of the integrated approach (i.e., road base).
q_t = roadway LL surcharge.

(3) Eq. 52

α_b = angle between wall face and projection of the midline of the bridge surcharge to the wall face found using equation 46 (see figure 23).
β_b = angle between the wall face and projection of the midline of the surcharge to the wall face found using equation 47 (see figure 23).
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

Lateral pressure due to the road base surcharge within GRS:

$$\sigma_{h,rb} = q_{rb} K_{ar}$$ (3) Eq. 53

Lateral pressure due to the traffic surchage within GRS:

$$\sigma_{h,t} = q_t K_{ar}$$ (3) Eq. 54

Service Limit State

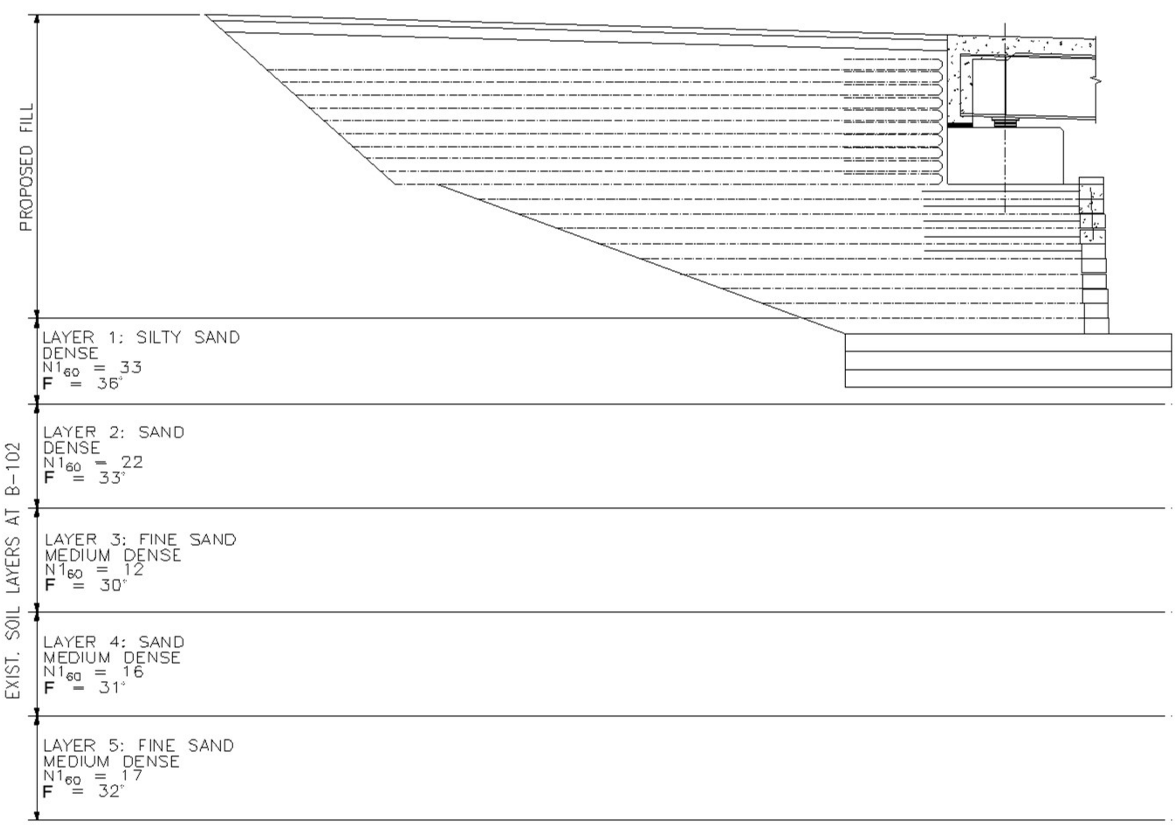
z	$\sigma_{h,W}$	α_b	β_b	$\sigma_{h,bridge,eq}$	$\sigma_{h,rb}$	$\sigma_{h,t}$	σ_h	T_{req}	$T_{@\varepsilon = 2\%}$	
0.67 ft	0.013 ksf	2.59	-1.30	0.466 ksf	0.224 ksf	0.071 ksf	0.773 ksf	0.972 klf	1.25 klf	OK
1.33 ft	0.026 ksf	2.12	-1.06	0.445 ksf	0.224 ksf	0.071 ksf	0.765 ksf	0.962 klf	1.25 klf	OK
2.00 ft	0.039 ksf	1.74	-0.87	0.408 ksf	0.224 ksf	0.071 ksf	0.742 ksf	0.932 klf	1.25 klf	OK
2.67 ft	0.053 ksf	1.46	-0.73	0.366 ksf	0.224 ksf	0.071 ksf	0.713 ksf	0.897 klf	1.25 klf	OK
3.33 ft	0.066 ksf	1.24	-0.62	0.327 ksf	0.224 ksf	0.071 ksf	0.687 ksf	0.863 klf	1.25 klf	OK
4.00 ft	0.079 ksf	1.07	-0.54	0.292 ksf	0.224 ksf	0.071 ksf	0.665 ksf	0.836 klf	1.25 klf	OK
4.67 ft	0.092 ksf	0.94	-0.47	0.262 ksf	0.224 ksf	0.071 ksf	0.648 ksf	0.815 klf	1.25 klf	OK
5.33 ft	0.105 ksf	0.84	-0.42	0.236 ksf	0.224 ksf	0.071 ksf	0.636 ksf	0.800 klf	1.25 klf	OK
6.00 ft	0.118 ksf	0.75	-0.38	0.215 ksf	0.224 ksf	0.071 ksf	0.628 ksf	0.789 klf	1.25 klf	OK
6.67 ft	0.132 ksf	0.68	-0.34	0.197 ksf	0.224 ksf	0.071 ksf	0.623 ksf	0.783 klf	1.25 klf	OK
7.33 ft	0.145 ksf	0.63	-0.31	0.181 ksf	0.224 ksf	0.071 ksf	0.621 ksf	0.780 klf	1.25 klf	OK
8.00 ft	0.158 ksf	0.58	-0.29	0.168 ksf	0.224 ksf	0.071 ksf	0.620 ksf	0.780 klf	1.25 klf	OK
8.67 ft	0.171 ksf	0.53	-0.27	0.156 ksf	0.224 ksf	0.071 ksf	0.622 ksf	0.781 klf	1.25 klf	OK
9.33 ft	0.184 ksf	0.50	-0.25	0.146 ksf	0.224 ksf	0.071 ksf	0.625 ksf	0.785 klf	1.25 klf	OK
10.00 ft	0.197 ksf	0.47	-0.23	0.137 ksf	0.224 ksf	0.071 ksf	0.629 ksf	0.790 klf	1.25 klf	OK
10.67 ft	0.210 ksf	0.44	-0.22	0.129 ksf	0.224 ksf	0.071 ksf	0.634 ksf	0.797 klf	1.25 klf	OK
11.33 ft	0.224 ksf	0.41	-0.21	0.122 ksf	0.224 ksf	0.071 ksf	0.640 ksf	0.804 klf	1.25 klf	OK
12.00 ft	0.237 ksf	0.39	-0.20	0.115 ksf	0.224 ksf	0.071 ksf	0.647 ksf	0.813 klf	1.25 klf	OK
12.67 ft	0.250 ksf	0.37	-0.19	0.110 ksf	0.224 ksf	0.071 ksf	0.654 ksf	0.822 klf	1.25 klf	OK
13.33 ft	0.263 ksf	0.35	-0.18	0.104 ksf	0.224 ksf	0.071 ksf	0.662 ksf	0.832 klf	1.25 klf	OK
14.00 ft	0.276 ksf	0.34	-0.17	0.100 ksf	0.224 ksf	0.071 ksf	0.670 ksf	0.843 klf	1.25 klf	OK
14.67 ft	0.289 ksf	0.32	-0.16	0.095 ksf	0.224 ksf	0.071 ksf	0.679 ksf	0.854 klf	1.25 klf	OK
15.33 ft	0.303 ksf	0.31	-0.15	0.091 ksf	0.224 ksf	0.071 ksf	0.688 ksf	0.865 klf	1.25 klf	OK
16.00 ft	0.316 ksf	0.29	-0.15	0.088 ksf	0.224 ksf	0.071 ksf	0.698 ksf	0.877 klf	1.25 klf	OK
16.67 ft	0.329 ksf	0.28	-0.14	0.084 ksf	0.224 ksf	0.071 ksf	0.707 ksf	0.889 klf	1.25 klf	OK
17.33 ft	0.342 ksf	0.27	-0.14	0.081 ksf	0.224 ksf	0.071 ksf	0.717 ksf	0.902 klf	1.25 klf	OK
18.04 ft	0.356 ksf	0.26	-0.13	0.078 ksf	0.224 ksf	0.071 ksf	0.728 ksf	0.915 klf	1.25 klf	OK
18.04 ft	0.356 ksf	0.26	-0.13	0.078 ksf	0.224 ksf	0.071 ksf	0.728 ksf	0.915 klf	1.25 klf	OK

Final Design - Abutment 2 - Settlement VTRANS D37 IM 091-1(68)

References:
(1) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018

The proposed Span 1 - East Abutment is assumed to be founded on the existing granular soil defined in boring B-102. A sketch of the existing soil layers in relation to the proposed abutment is shown below.

SPAN 1 – EAST ABUTMENT



Note that the proposed bottom of footing elevation for the Span 1 - East Abutment is 451.46 ft. Therefore, the footing will be founded on layers of medium dense to dense silty sand.

Abutment Geometry

Footing Width, B =	12.00 ft		See "Abutment Design"	
Abutment Length, L =	31.97 ft		See "Abutment Design"	
e_B =	1.67 ft		See "Abutment Design"	
B' =	8.67 ft =	12.00 ft -	(2.00 x	1.67 ft)

Final Design - Abutment 2 - Settlement VTRANS D37 IM 091-1(68)

Settlement Analyses

Settlment Analyses per (1) 10.6.2.4

Total settlement, including elastic, consolidation, and secondary components may be taken as:

$S_t = S_e + S_c + S_s$ (Ref 1 - Eq. 10.6.2.4.1-1)

For cohesionless soils, only elastic settlement is typically considered, therefore $S_t = S_e$

Elastic settlement can be approximated using either the elastic half-space method or the empirical Hough method. Both methods are calculated here for reference.

Elastic Half-Space Method:

$$S_e = \frac{[q_o(1 - v^2)\sqrt{A'}]}{144 E_s \beta_z}$$ (Ref 1 - Eq. 10.6.2.4.2-1)

Where:

q_o = applied vertical stress (ksf)

A' = effective area of footing (ft^2)

E_s = Young's modulus of soil taken from Article 10.4.6.3

B_z = shape factor taken as specified in table 10.6.2.4.2-1

v = Poisson's Ratio, taken as specified in Article 10.4.6.3

P_{SS} =	349 kips	(total vertical load on abutment, Service 1. See "Abutment Design")				
P_W =	564 kips =	17634.1 plf x	31.97 ft x	0.001		
P_{RSF} =	110 kips =	3450 plf x	31.97 ft x	0.001		
P_{FACE} =	51 kips =	1586.67 plf x	31.97 ft x	0.001		
P_{LL} =	225 kips					
P_{LS} =	25 kips =	250.00 psf x	3.08 ft x	31.97 ft x	0.001	
P =	1323 kips					

A' = B'L' = 277 ft² = 8.67 ft x 31.97 ft

q_o = P/A' = 4.8 ksf = 1323 kips/ 277 ft²

E_s = 4.17 ksi (1) Table C10.4.6.3-1, Loose/Medium Dense Sand

v = 0.28 (1) Table C10.4.6.3-1, Loose/Medium Dense Sand

L/B = 3.6891535 = 31.97 ft/ 8.67 ft

Footing Type = Flexible

βz = 1.161

Se = 0.11 ft = $\frac{4.8 \text{ ksf} \quad (1 - \quad 0.08) \times \quad 16.64 \text{ ft}}{(144 \times \quad 4.17 \times \quad 1.161)}$

Se = 1.26 in

Final Design - Abutment 2 - Settlement VTRANS D37 IM 091-1(68)

While the elastic half space method should provide reasonably accurate results, calculate total settlement using the Hough method for comparison.

Empirical Hough Method:

$$S_e = \sum_{i=1}^{n} \Delta H_i$$ (Ref 1 - Eq. 10.6.2.4.2-2)

Where: $$\Delta H_i = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$$ (Ref 1 - Eq. 10.6.2.4.2-3)

n = number of soil layers within zone of stress influence of the footing

H_c = initial height of each layer I (ft)

C' = bearing capacity index from Figure 10.6.2.4.2-1

σ'_o = initial vertical effective stress at the midpoint of layer I (ksf)

$\Delta\sigma_v$ = increase in vertical stress at the midpoint of layer I (ksf)

Depth of bottom of footing below grade, d =	0.54 ft	See Soil Properties - B105
Water Table Depth =	7 ft	
Soil Unit Weight =	0.125 kcf	
Water Unit Weight =	0.062 kcf	

Final Design - Abutment 2 - Settlement VTRANS D37 IM 091-1(68)

Increase in vertical stress at point directly below footing is equal to the applied vertical stress, qo (ksf). Increase in vertical stress at a depth of z below the bottom of footing are assumed equal to applied load P divided by effective area at point of interest calculated assuming a 2:1 distribution slope, therefore A' = (B'+z)(L'+z)

Layer	Depth to Bottom of Layer	H_c	Midpoint Depth Below Footing, z	Sample Midpoint	h_1	h_2
1	3.00 ft	3.00 ft	1.23 ft	2.00	2.00 ft	0.00 ft
2	7.00 ft	4.00 ft	4.46 ft	5.00 ft	5.00 ft	0.00 ft
3	12.00 ft	5.00 ft	8.96 ft	9.50 ft	7.00 ft	2.50 ft
4	19.00 ft	7.00 ft	14.96 ft	15.50 ft	7.00 ft	8.50 ft
5	23.50 ft	4.50 ft	20.71 ft	21.25 ft	7.00 ft	14.25 ft
6	33.50 ft	10.00 ft	27.96 ft	28.50 ft	7.00 ft	21.50 ft
7	43.50 ft	10.00 ft	37.96 ft	38.50 ft	7.00 ft	31.50 ft

Layer	Depth to Bottom of Layer	σ'_v	A'	$\Delta\sigma_v$ (P/A')	$N1_{60}$	*C'	ΔH
1	3.00 ft	0.250 ksf	329 ft²	4.0 ksf	33	105	0.04 ft
2	7.00 ft	0.625 ksf	478 ft²	2.8 ksf	22	80	0.04 ft
3	12.00 ft	1.344 ksf	721 ft²	1.8 ksf	12	60	0.03 ft
4	19.00 ft	2.470 ksf	1109 ft²	1.2 ksf	16	70	0.02 ft
5	23.50 ft	3.548 ksf	1547 ft²	0.9 ksf	17	70	0.01 ft
6	33.50 ft	4.908 ksf	2195 ft²	0.6 ksf	18	70	0.01 ft
7	43.50 ft	6.784 ksf	3260 ft²	0.4 ksf	19	71	0.00 ft
						Total =	0.14 ft
						Total =	1.64 in

*In Figure 10.6.2.4.2-1 Well graded silty sand was assumed

Soil Properties - Abutment 3 (B-103) VTRANS D37 IM 091-1(68)

References:
(1) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018
(2) Geotechnical Data Report, Teracon Consultants Inc., March 27. 2019

g_w = 0.062 kcf
g_{sat} = 0.125 kcf

Water Table = 8.00 ft (2)
Ground EL. At Boring = 448.00
BOF EL. = 443.46

Correct Blow Count for Overburden Pressure

	To Depth (ft)	h_1 (ft)	h_2 (ft)	σ'_v (ksf)	C_N	N blows/ft	$N1 = C_N N$ blows/ft	N_{60} = (ER/60%)N blows/ft	$N1_{60} = C_N N_{60}$ blows/ft
BOF	4.50	4.50	0.00	0.56	1.43	12.00	17.11	16.00	22.82
	10.50	8.00	2.50	1.16	1.18	6.50	7.70	8.67	10.27
	14.50	8.00	6.50	1.41	1.12	9.00	10.07	12.00	13.43
	19.50	8.00	11.50	1.72	1.05	11.00	11.57	14.67	15.43
	24.50	8.00	16.50	2.03	1.00	12.50	12.45	16.67	16.61
	33.50	8.00	25.50	2.60	0.91	24.00	21.95	32.00	29.27
	41.00	8.00	33.00	3.07	0.86	57.00	48.96	76.00	65.28

h_1 = depth above water table
h_2 = depth below water table
$\sigma'_v = gh_1 + g' h_2$
$g' = g_{sat} - g_w$
$C_N = 0.77 \log_{10} (40 / \sigma'_v) < 2$ (1) 10.4.6.2.4.-1

Correct for Hammer Efficiency

N_{60} = (ER/60%)N (1) 10.4.6.2.4-2
ER = 0.80
N_{60} = 1.33 = 0.80/ 0.60
$N1_{60} = C_N N_{60}$ (1) 10.4.6.2.4-3

Soil Properties - Abutment 3 (B-103) VTRANS D37 IM 091-1(68)

Calculate Drained Friction Angle

Table 10.4.6.2.4-1—Correlation of *SPT* $N1_{60}$ Values to Drained Friction Angle of Granular Soils (modified after Bowles, 1977)

$N1_{60}$	ϕ_f
<4	25–30
4	27–32
10	30–35
30	35–40
50	38–43

Use middle values of range:

$N1_{60}$	ϕ_f
<4	25.00
4.00	27.00
10.00	30.00
30.00	35.00
50.00	38.00

Footing Width = 14.00 ft
Soil Depth = 28.00 ft = 2.00 x 14.00 ft

Footing Depth = 4.54 ft

	Depth (ft)	$N1_{60}$	$N1_{60\ low}$	$N1_{60\ high}$	$\phi_{f\ low}$	$\phi_{f\ high}$	ϕ_f		
BOF	4.50	22.82	10.00	30.00	30.00	35.00	33.20	Use ϕ_f =	33.00
	10.50	10.27	10.00	30.00	30.00	35.00	30.07		
	14.50	13.43	10.00	30.00	30.00	35.00	30.86	Use ϕ_f =	31.00
	19.50	15.43	10.00	30.00	30.00	35.00	31.36		
	24.50	16.61	10.00	30.00	30.00	35.00	31.65		

References:
(1) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018

Bearing Resistance of Soil (10.6.3.1)

Bearing resistance of soil calculated per (1) 10.6.3.1

$$q_n = cN_{cm}+\gamma D_f N_{qm} C_{wq}+0.5\gamma B N_{\gamma m} C_{w\gamma}$$ (1) 10.6.3.1.2a-1

Abutment Geometry

Footing width, B =	14.00	ft		See GRS Abutment Design
Footing length, L =	39.19	ft		See GRS Abutment Design
Embedment Depth, D_f =	4.00	ft		See GRS Abutment Design
Groundwater Depth, D_w =	8.00	ft		See GRS Abutment Design

Cohesive Term

ϕ_f =	0		
c =	0		
i_c =	1.00	(load inclination not considered)	(1) 10.6.3.1.2a-6
N_c =	0		(1) Table 10.6.3.1.2a-1

Surcharge Term (soil above footing)

ϕ_f =	33.00		
γ =	0.125	kcf	
D_f/B =	0.29		
$1.5B + D_f$ =	25.00		
i_q =	1.00	(load inclination not considered)	(1) 10.6.3.1.2a-7
N_q =	26.10		(1) Table 10.6.3.1.2a-1
C_{wq} =	1.00		(1) Table 10.6.3.1.2a-2
d_q =	1.00		(1) Table 10.6.3.1.2a-4

Unit Weight Term (soil below footing)

ϕ_f =	31.00		
γ =	0.125	kcf	
$1.5B + D_f$ =	25.00		
i_γ =	1.00	(load inclination not considered)	(1) 10.6.3.1.2a-7
N_γ =	26.00		(1) Table 10.6.3.1.2a-1
$C_{w\gamma}$ =	0.50		(1) Table 10.6.3.1.2a-2

Soil Properties - Abutment 3 (B-103) VTRANS D37 IM 091-1(68)

Considerations for footings on slope

Is footing bearing on or near slope ? no

$q_{n\text{-sloping ground}} = RC_{BC}(cN_c+0.5\gamma BN_\gamma)$ (1) 10.6.3.1.2c-1

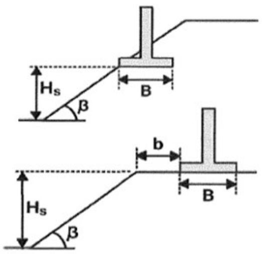
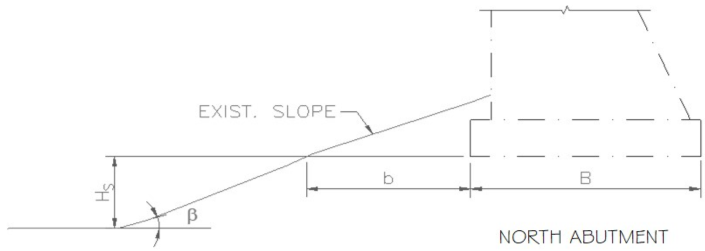


Figure 10.6.3.1.2c-1—Definition of Footing and Slope Geometric Parameters for Determination of RC_{BC}



Height of Slope, H_s = 0.00 ft

b = 0.00 ft

Angle of slope, β = 0.00 degrees

γ = 0.125 kcf

N_s = c'=0

ϕ = 31.00

For β = 30 degrees, c' = 0

B/H_s	b/B	RC_{BC}	
2	0.50	0.64	(1) Table 10.6.3.1.2c-1
2	1.25	0.74	(1) Table 10.6.3.1.2c-1

Factored Bearing Resistance:

Strength, ϕ_b = 0.45 (1) 10.5.5.2.2

Extreme and Service, ϕ_b = 1 (1) 10.5.5.3.3

For footing w/o slope

$q_n = cN_{cm}+\gamma D_f N_{qm} C_{wq}+0.5\gamma BN_{\gamma m}C_{w\gamma}$ (1) 10.6.3.1.2a-1

$N_{cm} = N_c s_c i_c$ (1) 10.6.3.1.2a-2

$N_{qm} = N_q s_q d_q i_q$ (1) 10.6.3.1.2a-3

$N_{\gamma m} = N_\gamma s_\gamma i_\gamma$ (1) 10.6.3.1.2a-4

$s_c = 1+(B/L)(N_q/N_c)$ (1) Table 10.6.3.1.2a-3

$s_q = 1+B/L(\tan\phi_f)$ (1) Table 10.6.3.1.2a-3

$s_\gamma = 1-0.4(B/L)$ (1) Table 10.6.3.1.2a-3

For footing w/ slope

$q_{n\text{-sloping ground}} = RC_{BC}(cN_c+0.5\gamma BN_\gamma C_{w\gamma})$ (1) 10.6.3.1.2c-1

Soil Properties - Abutment 3 (B-103) VTRANS D37 IM 091-1(68)

Limit State	e_B (ft)	B' (ft)	Shape Correction Factor s_c	s_q	s_γ	Bearing Capacity Factor N_{cm}	N_{qm}	$N_{\gamma m}$	Slope Consideration B'/H_s	b/B'	RC_{BC}	Bearing Capacity q_n (ksf)	$\phi_b q_n$ (ksf)
Str I - A	2.39	9.21	0.00	1.15	0.91	0.00	30.08	23.55	n/a	n/a	n/a	21.82	9.82

FINAL DESIGN - GRS-IBS ABUTMENT - ABUTMENT 3 DESIGN | VTRANS D37 IM 091-1(68)

References:

1) GRS-IBS Interim Implementation Guide, FHWA, June 2018

2) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018

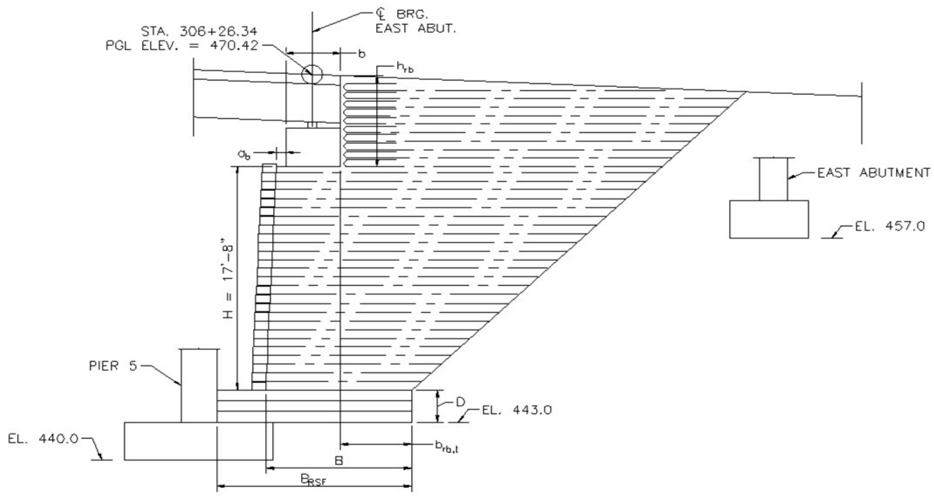
Material Properties

γ_f =	115.00 pcf	(Reinforced Fill)				
ϕ (Reinforced Soil)=	45	degrees				
K_a (Reinforced Soil) =	0.17 =		$\tan^2($	45 -	(45/	2))

Assume that retained fill at West Abutment, Span 2 will be imported and of a higher quality than in-situ soil.

γ_b =	125.00 pcf	(Retained Fill)				
ϕ (Existing Filll)=	34.00	degrees				
K_{ab} (Existing Fill) =	0.28 =		$\tan^2($	45 -	(34/	2))

Geometry



Bridge Span Length =	85.94 ft			
Abut. Height =	21.54 ft			
H =	21.54 ft			
Superstructure Depth =	4.39 ft			
h_{rb} =	6.95 ft			
B =	10.5 ft			
b =	4.75 ft			
Toe Length, X_{RSF} =	2.50 ft			
Total Width, B_{RSF} =	14 ft			
$b_{rb,t}$ =	5.08 ft =	10.5 ft -	0.67 ft -	4.75 ft
Depth of RSF, D =	2.50 ft			
Setback Distance, a_b =	0.67 ft			
L =	34.19 ft			
h_{block} =	8.00 in			
D_{block} =	12.00 in			
L_{block} =	18.00 in			
Weight =	85 lbs	per block		

Reinforcement

S_v =	0.67 ft
d_{max} =	0.75 in
T_f =	5.90 klf

Load Factors
Per (3) Table 3.4.1-1 and Table 3.4.1-2.

γ_p (DC, Min.) =	0.90	3) Table 3.4.1-2
γ_p (DC, Max.) =	1.25	3) Table 3.4.1-2
γ_p (DW, Min.) =	0.65	3) Table 3.4.1-3
γ_p (DW, Max.) =	1.50	3) Table 3.4.1-4
γ_p (EH, Max) =	1.50	3) Table 3.4.1-2
γ_p (EH, Min.) =	0.90	3) Table 3.4.1-2
γ_p (EV, Min.) =	1.00	3) Table 3.4.1-2
γ_p (EV, Max.) =	1.35	3) Table 3.4.1-2
Factor (LS) =	1.75	3) Table 3.4.1-1
Factor (LL) =	1.75	3) Table 3.4.1-1

Calculate Average Height of Precast Beam Seat

Min. Height of Cap =	1.50 ft			
Cross Slope =	0.0625			
Cap Length =	34.19 ft			
Max. Height =	3.64 ft =	1.50 ft +	(34.19 ft x	0.0625)
Average Height =	2.57 ft			

Calculate Vertical Loads and Applied Pressures

Calculate q_{DL}

$$q_{DL} = \frac{Q_{DL}}{bL}$$

	Total DC Rxn	No. of Girders	DC1	DC2
N. Ext.	47.53 k =	1 x	(42.24 k +	5.29 k)
Typ. Int.	147.12 k =	3 x	(43.75 k +	5.29 k)
S. Ext.	41.32 k =	1 x	(36.03 k +	5.29 k)
Total Girder DC Reaction =	235.97 k			

	Total DW Rxn	No. of Girders	DW
N. Ext.	7.26 k =	1 x	7.26 k)
Typ. Int.	21.78 k =	3 x	7.26 k)
S. Ext.	7.26 k =	1 x	7.26 k)
Total Girder DW Reaction =	36.30 k		

Assume a concrete end block integral with the steel beam that is 3'-1" high x 12" wide x full width of abutment.

End Block Weight =	15.81 k =	3.083 ft x	1.00 ft x	34.19 ft x	0.150 kcf

Assume concrete deck beyond CL of bearing is equal to 1.60 SF, per AutoCAD

Additional Deck Weight =	8.21 k =	1.60 ft² x	0.150 kcf x	34.19 ft

Assume that girder extends additional 2'-0" beyond the CL of Bearing

Top Flange =	0.10 ft² =	16 in x	0.875 in x	0.007
Web =	0.12 ft² =	34 in x	0.5 in x	0.007
Bottom Flange =	0.14 ft² =	16 in x	1.25 in x	0.007
Total Steel Area =	0.35 ft²			

Additional Weight of Steel =	1.74 k =	0.35 ft² x	0.49 kcf x	2.00 ft x	5 girders	
Weight of Beam Seat =	62.56 k =	2.57 ft x	4.75 ft x	34.19 ft x	0.15 kcf	
Total DC Reaction (Unfactored) =	324.28 k =	235.97 k +	15.81 k +	8.21 k +	1.74 k +	62.56 k
Total DW Reaction (Unfactored) =	36.30 k					

q_{DC} =	1996.94 psf =	(324.284 kips x	1000)/	(4.75 ft x	34.19 ft)
q_{DW} =	223.54 psf =	(36.3 kips x	1000)/	(4.75 ft x	34.19 ft)

Calculate q_{LL}

Calculate Live Load Reactions Per Barrel

Roadway Width = 27.00 ft
Number of Trucks = 2
Impact = 1.33

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

Reaction (Single Truck) = 64.18 kips = ((32 kips x 85.94 ft) + (32 kips x 71.94 ft) + (8 kips x 57.94 ft)) / 85.94 ft

Lane Load Reaction (Single Truck) = 27.5 kips = 0.5 x 0.64 klf x 85.94 ft

Reaction - 1 Lane Loaded = 135.43 kips = 1.2 x ((64.18 kips x 1.33) + 27.5)
Reaction - 2 Lanes Loaded = 225.72 kips = 1 x ((2 x 64.18 kips x 1.33) + (2 x 27.5 kips))
Max. Reaction = 225.72 kips

Bridge LL Reaction = 225.72 kips

$$q_{LL} = \frac{Q_{LL}}{bL}$$

q_{LL} = 1389.98 psf = (225.72 kips x 1000)/ (4.75 ft x 34.19 ft)

Calculate Traffic Surcharge, q_T

Per (3) 3.11.6.4-1, the equivalent height of soil acting as a surcharge load shall be determined as follows:

Table 3.11.6.4-1—Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic

Abutment Height (ft)	h_{eq} (ft)
5.0	4.0
10.0	3.0
≥20.0	2.0

Note that linear interpolation shall be used for intermediate wall heights.

H = 24.04 ft
h_{eq} = 2.00 ft
γ_r = 125.00 pcf

$$q_t = h_{eq}\gamma_b$$

q_T = 250.00 psf = 2.00 ft x 125.00 pcf

Calculate Road Base Surcharge, q_{rb}

γ_{rb} = 115.00 pcf

$$q_{rb} = H_{rb}\gamma_{rb}$$

q_{rb} = 799.68 psf = 6.95 ft x 115.00 pcf

Calculate Weight of GRS Abutment

$$W = BH\gamma_r$$

H = 21.54 ft
B = 10.5 ft
γ_r = 115.00 pcf

W = 26009.55 plf = 21.54 ft x 10.5 ft x 115.00 pcf

Calculate Weight of RSF

$$W_{RSF} = B_{RSF} D_{RSF} \gamma_r$$

B = 14 ft
D = 2.5 ft
γ_r = 125.00 pcf

W_{RSF} = 4025.00 plf = 14 ft x 2.5 ft x 115.00 pcf

Calculate Weight of Facing

N_{block} = 32.31 = 21.54 ft/ 0.67 ft
N_{block} = 33

$$W_{face} = N_{block} \frac{W_{block}}{L_{block}}$$

W_{face} = 1870.00 plf = 33 x (85 lbs/ (18.00 in/ 12)

Summary of Applied Loads

q_{DC} = 1996.94 psf
q_{DW} = 223.54 psf
q_{LL} = 1389.98 psf
q_t = 250.00 psf
q_{rb} = 799.68 psf
W = 26009.55 plf
W_{RSF} = 4025.00 plf
W_{face} = 1870.00 plf

Check Beam Seat Pressure

Per (1) 4.3.5.4, the service bearing pressure should be targeted to around 4 ksf.

eaction = 324.28 kips
eaction = 36.30 kips
eaction = 225.72 kips
Total = 586.30 kips

b = 4.75 ft
L = 34.19 ft

q_{beat} = 3.61 ksf = 586.30 kips/ (4.75 ft x 34.19 ft)

OK

Calculte Direct Sliding Effects at RSF/GRS Interface

Calculate Sliding per the (1) and (3) Strength I Load Combination

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2$$ (1) Eq. 9

$$F_{rb} = q_{rb} K_{ab} H$$ (1) Eq. 10

$$F_t = q_t K_{ab} H$$ (1) Eq. 11

$$F_R = \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t$$ (1) Eq. 12

F_b =	8198.23 lbs =	0.5 x	125.00 pcf x	0.28 x	21.54 ft^ 2	
F_{rb} =	4869.82 lbs =	799.68 psf x	0.28 x	21.54 ft		
F_t =	1522.42 lbs =	250.00 psf	0.28 x	21.54 ft		

F_R = 22.27 klf = (1.5 x (8198.23 lbs + 4869.82 lbs) + (1.75 1522.42 lbs))/ 1000

$$W_{T,R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EV\,MIN}(q_{rb} b_{rb,t})$$ Eq. 14

$W_{T,R}$ = 40.98 klf = ((1.0 x 26009.55 plf) + (0.9 x 1996.94 psf x 4.75 ft) + (0.65 x 223.54 psf x 4.75 ft) +
(0.9 x 1870.00 plf) + (1.0 x 799.68 psf x 5.08 ft))/ 1000

Assume that μ = 2/3*tanϕ, per (1) C.1.6.1.1

μ = 0.67 = 0.67 x tan(45)

$$R_R = \Phi_\tau\left(W_{T,R}\mu\right)$$ (1) Eq. 13

R_R = 27.32 klf = 40.98 klf x 0.667

OK

Calculate Direct Sliding at Base of RSF

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2$$ 17

$$F_{rb,RSF} = q_{rb}K_{ab}(H + D_{RSF})$$. 18

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF})$$. 19

$F_{b, RSF}$ =	10.21 klf =	(0.5 x	125.00 pcf x	0.28 x	(21.54 ft +	2.5)^ 2)/	1000
$F_{rb, RSF}$ =	5.44 klf =	(799.68 psf x	0.28 x	(21.54 ft +	2.50))/ 1000		
$F_{t, RSF}$ =	1.70 klf =	250.00 psf x	0.28 x	(21.54 ft +	2.5 ft))/	1000	

$$F_{R,RSF} = \gamma_{EH\,MAX}\left(F_{b,RSF} + F_{rb,RSF}\right) + \gamma_{LS}F_{t,RSF}$$) Eq. 20

$F_{R,RSF}$ = 26.44 klf = (1.5 x (10.21 klf + 5.44 klf)) + (1.75 x 1.70 klf)

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\,MIN}W_{RSF}$$ 1) Eq. 22

$W_{T,R,RSF}$ = 45.01 klf = 40.98 klf + (1.0 x 4.03 klf)

$$R_{R,RSF} = \Phi_\tau(W_{T,R,RSF}\mu_{RSF})$$]. 21

Note that ϕ = 1.0, per (1) 4.3.6.1.2

Per (2), and the previous calculation, the angle of internal friction beneath the RSF is equal to 31 degrees

$R_{R, RSF}$ = 27.04 klf = 1.0 x 45.01 klf x tan(31)

OK

Calculate External Bearing Resistance

Calculate V_R

Per (1) Eq. 26,

$$\sum V_R = \gamma_{EV\,MAX}(W) + \gamma_{EV\,MAX}(W_{RSF}) + \gamma_{DC\,MAX}(W_{face}) + \gamma_{LS}(q_t b_{rb,t}) + \gamma_{EH\,MAX}(q_{rb} b_{rb,t}) + \gamma_{DC\,MAX}(q_{DL} b) + \gamma_{LS}(q_{LL} b)$$

Note that $\gamma_{EV\,Max}$ is used in place of $\gamma_{EH\,Max}$ to factor q_{rb}*$b_{rb,t}$ since this is a summation of vertical reactions.

W =	35.11 klf =	1.35 x	26009.55 plf/	1000	
W_{RSF} =	5.43 klf =	1.35 x	4025.00 plf/	1000	
W_{FACE} =	2.34 klf =	1.25 x	1870.00 plf/	1000	
Q_t =	2.22 klf =	1.75 x	(250.00 psf x	5.08 ft)/	1000
Q_{rb} =	5.49 klf =	1.35 x	(799.68 psf x	5.08 ft)/	1000
Q_{DC} =	11.86 klf =	1.25 x	(1996.94 psf x	4.75 ft)/	1000
Q_{DW}=	1.59 klf =	1.5 x	(223.54 psf x	4.75 ft)/	1000
Q_{LL} =	11.55 klf =	1.75 x	(1389.98 psf	4.75 ft)/	1000
Total =	75.60 klf				

Calculate $M_{D,R}$

Per (1) Eq. 28

$$\sum M_{D,R} = \gamma_{EH\,MAX} F_{b,RSF}\left(\frac{H + D_{RSF}}{3}\right) + \gamma_{LS} F_{t,RSF}\left(\frac{H + D_{RSF}}{2}\right) + \gamma_{EH\,MAX} F_{rb,RSF}\left(\frac{H + D_{RSF}}{2}\right)$$

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2$$ (3) Eq. 17

$$F_{rb,RSF} = q_{rb} K_{ab}(H + D_{RSF})$$ (3) Eq. 18

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF})$$ (3) Eq. 19

$F_{b,\ RSF}$ =	10.21 klf =	(0.5 x	125.00 pcf x	0.28 x	(21.54 ft +	2.5)^ 2)/	1000
$F_{rb,\ RSF}$ =	5.44 klf =	799.68 psf x	0.28 x	(21.54 ft +	2.5 ft))/	1000	
$F_{t,\ RSF}$ =	1.70 klf =	250.00 psf x	0.28 x	(21.54 ft +	2.5 ft))/	1000	

$F_{b,\ RSF}$ =	122.74 k-ft/ ft =	1.5 x	10.21 klf x	(0.33 x	(21.54 ft +	2.5 ft))
$F_{t,\ RSF}$ =	35.74 k-ft/ ft =	1.75 x	1.70 klf x	(0.50 x	(21.54 ft +	2.5 ft))
$F_{rb,\ RSF}$ =	97.99 k-ft/ ft =	1.5 x	5.44 klf x	(0.50 x	(21.54 ft +	2.5 ft))
Total =	256.48 k-ft/ ft					

Note that $M_{D,R}$ is taken about the bottom center of the width of the RSF.

Calculate $M_{R,R}$

Per (1) Eq. 29

$$\sum M_{R,R} = (\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b)\left[\left(\frac{b}{2} + a_b\right) - \left(\frac{B_{RSF}}{2} - x_{RSF} - b_{block}\right)\right] + \left(\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t}\right)\left(\frac{B_{RSF}}{2} - \frac{b_{rb}}{2}\right) + \gamma_{EV\,MAX} W\left(\frac{B_{RSF}}{2} - \frac{B}{2}\right) + \gamma_{DC\,MAX} W_{face}\left(B + \frac{b_{block}}{2} - \frac{B_{RSF}}{2}\right)$$

$_{DL}+Q_{LL}$ =	-11.46 k-ft =	((1.25 x	2.00 ksf) +	(1.50 x	0.22 ksf) +	(1.75 x	1.39 ksf)) x 4.75 ft x
		((2.375 ft x +	0.67 ft) -	(7 ft -	2.5 ft -	1.00 ft))	
Q_t+Q_{rb} =	34.38 k-ft =	((1.75 x	0.25 klf x	5.08 ft) +	(1.35 x	0.800 klf x	5.08 ft)) x
		(7 ft -	2.54 ft)				
W =	61.45 k-ft =	1.35 x	26.01 klf x	(7 ft -	5.25 ft)		
W_{Face} =	-8.84 k-ft =	1.35 x	1.87 klf x	((2.5 ft +	1 ft) -	7 ft)	
Total =	75.53 k-ft/ ft						

Note that $M_{R,R}$ is taken about the bottom center of the width of the RSF.

Calculate Eccentricity, $e_{B,R}$

Per (1) Eq. 27

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R}$$

$e_{B,R}$ = 2.39 ft = (256.48 k-ft/ ft - 75.53 k-ft/ ft) / 75.60 klf

Calculate $\sigma_{v,base,R}$

Since $e_{B,R}$ = negative, set equal to zero in calcaulation of base bearing pressure.

Per (1) Eq. 25,

$$\sigma_{v,base,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}}$$

$\sigma_{v,base,R}$ = 8.21 ksf = 75.60 klf/ (14 ft - (2 x 2.39 ft)

Per bearing capacity calculation, factored bearing capacity = 9.82 ksf

OK

Calculate Internal Bearing Resistance

Per (1), Eq. 35

$$q_{n,an} = \left[0.7^{\left(\frac{S_v}{6d_{max}}\right)} \frac{T_f}{S_v}\right] K_{pr}$$

$q_{n,an}$ = nominal bearing resistance of the GRS abutment using the analytical method.
S_v = reinforcement spacing.
d_{max} = maximum grain size.
T_f = ultimate reinforcement strength.
K_{pr} = coefficient of passive earth pressure for the reinforced fill (calculated in equation 35).

K_{pr} = 5.83 = tan^2(45 + (45/ 2))

$q_{n,an}$ = 27.36 ksf = [(0.7 ^ (0.67 ft/ (6 x 0.06 ft))) x (5.90 klf/ 0.67 in)] x 5.83

ϕ_{cap} = 0.45 (3) 4.3.7.2

$\phi_{cap}q_{n,an}$ = 12.31 ksf = 0.45 x 27.36 ksf

$$V_{applied,f} = \gamma_{DC\ MAX} q_{DL} + \gamma_{LL} q_{LL}$$

(1) Eq. 32

$V_{app,f}$ = 5.26 ksf = (1.25 x 2.00 ksf) + (1.50 x 0.22 ksf) + (1.75 x 1.39 ksf)

OK

Calculate and Check Deformations

Per (1), Eq. 37

$$q_{DL,allow\ @\varepsilon=1\%} = 0.2\left[0.7^{S_v/6d_{max}} \left(\frac{T_f}{S_v}\right) K_{pr}\right]$$

6^*d_{max} = 0.375 ft = 6 x 0.75 in x 0.083

T_f/S_v = 8.85 = 5.90 klf/ 0.67 ft

$/6^*d_{max}$ = 1.78 = 0.67 ft/ 0.375 ft

$q_{dl,\ all}$ = 5.47 ksf = 0.2 ((0.7 ^ 1.78 in) x 8.85 x 5.83 in)

q_{DL} = 2.00 ksf

OK

Calculate Lateral Strain

Per (1) 4.3.7.2.2, for the calculation of lateral strain, assume a vertical strain equal to 1% of the abutment height, H.

).01*H =	0.22 ft =	0.01 x	21.54 ft

$$D_L = \frac{2b_q D_v}{H}$$ (1) Eq. 38

D_L =	0.01 ft =	(2 x	(4.75 ft +	0.67 ft) x	0.02 ft)/	21.54 ft
D_L =	0.11 in =	0.01 ft x	12			

Per (1) 4.3.7.2.2, the total lateral strain should be limited to twice the vertical strain.

Strain =	0.43 ft =	2 x	0.22 ft

OK

Calculate Reinforcement Strength

q_{DC} =	1996.94 psf
q_{DW} =	223.54 psf
q_{LL} =	1389.98 psf
q_t =	250.00 psf
q_{rb} =	799.68 psf
W =	26009.55 psf
W_{RSF} =	4025.00 psf
W_{face} =	1870.00 psf

The evaulation of the abutment for the strength limit state is conducted according to (1) 4.3.7.3.1. and using the following equations:

Required Reinforcment:

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 40

Factored Total Lateral Pressure:

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f}$$ (3) Eq. 41

Factored Lateral Pressure due to weight of GRS:

$$\sigma_{h,W,f} = \gamma_{EH\,MAX}(\gamma_r z K_{ar})$$

Where:
$\gamma_{EH\,MAX}$ = maximum horizontal earth pressure load factor.
γ_r = unit weight of reinforced backfill.
z = depth from the top of the wall.
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

(3) Eq. 42

Factored Latreal Pressure due to equivalent bridge load:

$$\sigma_{h,bridge,f} = \frac{(\gamma_{DC\,MAX} q_{DL} + \gamma_{LL} q_{LL}) - (\gamma_{EH\,MAX} q_{rb} + \gamma_{LS} q_t)}{\pi}\left[\alpha_b + \sin(\alpha_b)\cos(\alpha_b) + 2\beta_b\right] K_{ar} \quad (43)$$

Where:

$\gamma_{DC\,MAX}$ = maximum DL load factor.
q_{DL} = superstructure DL pressure.
γ_{LL} = bridge LL surcharge load factor.
q_{LL} = bridge LL pressure.
q_{rb} = surcharge due to the structural backfill of the integrated approach (i.e., road base).
γ_{LS} = LL surcharge load factor.
q_t = roadway LL surcharge.
α_b = angle between wall face and projection of the midline of the bridge surcharge to the wall face found using equation 46 (see figure 23).
β_b = angle between the projections of the inner and outer edge lines of the bridge surcharge to the wall face found using equation 47 (see figure 23).

(3) Eq. 43

$$\sigma_{h,rb,f} = \gamma_{EH\,MAX} q_{rb} K_{ar}$$

(3) Eq. 44

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar}$$

(3) Eq. 45

$$\alpha_b = \tan^{-1}\left(\frac{b}{2z}\right) - \beta_b$$

(3) Eq. 46

$$\beta_b = \tan^{-1}\left(\frac{-b}{2z}\right)$$

(3) Eq. 47

Factored Reinforcement Strength:

$$T_{f,f} = \Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right) = 0.4T_f$$

(3) Eq. 48

Strength Limit State

z	$\sigma_{h,W,f}$	α_b	β_b	$\sigma_{h,bridge,f}$	$\sigma_{h,rb,f}$	$\sigma_{h,t,f}$	$\sigma_{h,f}$	$T_{req,f}$	$T_{f,f}$	
0.67 ft	0.020 ksf	2.59	-1.30	0.617 ksf	0.339 ksf	0.124 ksf	1.099 ksf	1.382 klf	2.36 klf	OK
1.33 ft	0.039 ksf	2.12	-1.06	0.589 ksf	0.339 ksf	0.124 ksf	1.091 ksf	1.371 klf	2.36 klf	OK
2.00 ft	0.059 ksf	1.74	-0.87	0.540 ksf	0.339 ksf	0.124 ksf	1.062 ksf	1.335 klf	2.36 klf	OK
2.67 ft	0.079 ksf	1.46	-0.73	0.485 ksf	0.339 ksf	0.124 ksf	1.027 ksf	1.290 klf	2.36 klf	OK
3.33 ft	0.099 ksf	1.24	-0.62	0.432 ksf	0.339 ksf	0.124 ksf	0.994 ksf	1.249 klf	2.36 klf	OK
4.00 ft	0.118 ksf	1.07	-0.54	0.386 ksf	0.339 ksf	0.124 ksf	0.967 ksf	1.216 klf	2.36 klf	OK
4.67 ft	0.138 ksf	0.94	-0.47	0.347 ksf	0.339 ksf	0.124 ksf	0.948 ksf	1.191 klf	2.36 klf	OK
5.33 ft	0.158 ksf	0.84	-0.42	0.313 ksf	0.339 ksf	0.124 ksf	0.934 ksf	1.174 klf	2.36 klf	OK
6.00 ft	0.178 ksf	0.75	-0.38	0.285 ksf	0.339 ksf	0.124 ksf	0.925 ksf	1.163 klf	2.36 klf	OK
6.67 ft	0.197 ksf	0.68	-0.34	0.261 ksf	0.339 ksf	0.124 ksf	0.921 ksf	1.157 klf	2.36 klf	OK
7.33 ft	0.217 ksf	0.63	-0.31	0.240 ksf	0.339 ksf	0.124 ksf	0.920 ksf	1.156 klf	2.36 klf	OK
8.00 ft	0.237 ksf	0.58	-0.29	0.222 ksf	0.339 ksf	0.124 ksf	0.922 ksf	1.159 klf	2.36 klf	OK
8.67 ft	0.257 ksf	0.53	-0.27	0.207 ksf	0.339 ksf	0.124 ksf	0.926 ksf	1.164 klf	2.36 klf	OK
9.33 ft	0.276 ksf	0.50	-0.25	0.193 ksf	0.339 ksf	0.124 ksf	0.932 ksf	1.172 klf	2.36 klf	OK
10.00 ft	0.296 ksf	0.47	-0.23	0.181 ksf	0.339 ksf	0.124 ksf	0.940 ksf	1.182 klf	2.36 klf	OK
10.67 ft	0.316 ksf	0.44	-0.22	0.171 ksf	0.339 ksf	0.124 ksf	0.949 ksf	1.193 klf	2.36 klf	OK
11.33 ft	0.335 ksf	0.41	-0.21	0.161 ksf	0.339 ksf	0.124 ksf	0.960 ksf	1.206 klf	2.36 klf	OK
12.00 ft	0.355 ksf	0.39	-0.20	0.153 ksf	0.339 ksf	0.124 ksf	0.971 ksf	1.220 klf	2.36 klf	OK
12.67 ft	0.375 ksf	0.37	-0.19	0.145 ksf	0.339 ksf	0.124 ksf	0.983 ksf	1.235 klf	2.36 klf	OK
13.33 ft	0.395 ksf	0.35	-0.18	0.138 ksf	0.339 ksf	0.124 ksf	0.996 ksf	1.251 klf	2.36 klf	OK
14.00 ft	0.414 ksf	0.34	-0.17	0.132 ksf	0.339 ksf	0.124 ksf	1.009 ksf	1.268 klf	2.36 klf	OK
14.67 ft	0.434 ksf	0.32	-0.16	0.126 ksf	0.339 ksf	0.124 ksf	1.023 ksf	1.286 klf	2.36 klf	OK
15.33 ft	0.454 ksf	0.31	-0.15	0.121 ksf	0.339 ksf	0.124 ksf	1.037 ksf	1.304 klf	2.36 klf	OK
16.00 ft	0.474 ksf	0.29	-0.15	0.116 ksf	0.339 ksf	0.124 ksf	1.052 ksf	1.323 klf	2.36 klf	OK
16.67 ft	0.493 ksf	0.28	-0.14	0.111 ksf	0.339 ksf	0.124 ksf	1.067 ksf	1.342 klf	2.36 klf	OK
17.33 ft	0.513 ksf	0.27	-0.14	0.107 ksf	0.339 ksf	0.124 ksf	1.083 ksf	1.361 klf	2.36 klf	OK
18.00 ft	0.533 ksf	0.26	-0.13	0.103 ksf	0.339 ksf	0.124 ksf	1.099 ksf	1.381 klf	2.36 klf	OK
18.67 ft	0.552 ksf	0.25	-0.13	0.100 ksf	0.339 ksf	0.124 ksf	1.115 ksf	1.401 klf	2.36 klf	OK
19.33 ft	0.572 ksf	0.24	-0.12	0.096 ksf	0.339 ksf	0.124 ksf	1.131 ksf	1.422 klf	2.36 klf	OK
20.00 ft	0.592 ksf	0.24	-0.12	0.093 ksf	0.339 ksf	0.124 ksf	1.148 ksf	1.443 klf	2.36 klf	OK
20.67 ft	0.612 ksf	0.23	-0.11	0.090 ksf	0.339 ksf	0.124 ksf	1.165 ksf	1.464 klf	2.36 klf	OK
21.33 ft	0.631 ksf	0.22	-0.11	0.087 ksf	0.339 ksf	0.124 ksf	1.182 ksf	1.485 klf	2.36 klf	OK
21.54 ft	0.638 ksf	0.22	-0.11	0.087 ksf	0.339 ksf	0.124 ksf	1.187 ksf	1.492 klf	2.36 klf	OK

The evaulation of the abutment for the service limit state is conducted according to (1) 4.3.7.3.2. and using the following equations:

The nominal required reinforcement strength:

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$

(3) Eq. 50

The lateral pressure within the GRS Abutment at a given dpeth and location:

$$\sigma_h = \sigma_{h,W} + \sigma_{h,bridge,eq} + \sigma_{h,rb} + \sigma_{h,t}$$ (3) Eq. 51

The lateral pressure due to the equivalent bridge load:

$$\sigma_{h,bridge,eq} = \frac{(q_{DL} + q_{LL}) - (q_{rb} + q_t)}{\pi}\left[\alpha_b + \sin(\alpha_b)\cos(\alpha_b + 2\beta_b)\right]K_{ar} \quad (52)$$

Where:

q_{DL} = bridge DL pressure.
q_{LL} = bridge LL surcharge.
q_{rb} = surcharge due to the structural backfill of the integrated approach (i.e., road base).
q_t = roadway LL surcharge.

(3) Eq. 52

α_b = angle between wall face and projection of the midline of the bridge surcharge to the wall face found using equation 46 (see figure 23).
β_b = angle between the wall face and projection of the midline of the surcharge to the wall face found using equation 47 (see figure 23).
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

Lateral pressure due to the road base surcharge within GRS:

$$\sigma_{h,rb} = q_{rb} K_{ar}$$ (3) Eq. 53

Lateral pressure due to the traffic surchage within GRS:

$$\sigma_{h,t} = q_t K_{ar}$$ (3) Eq. 54

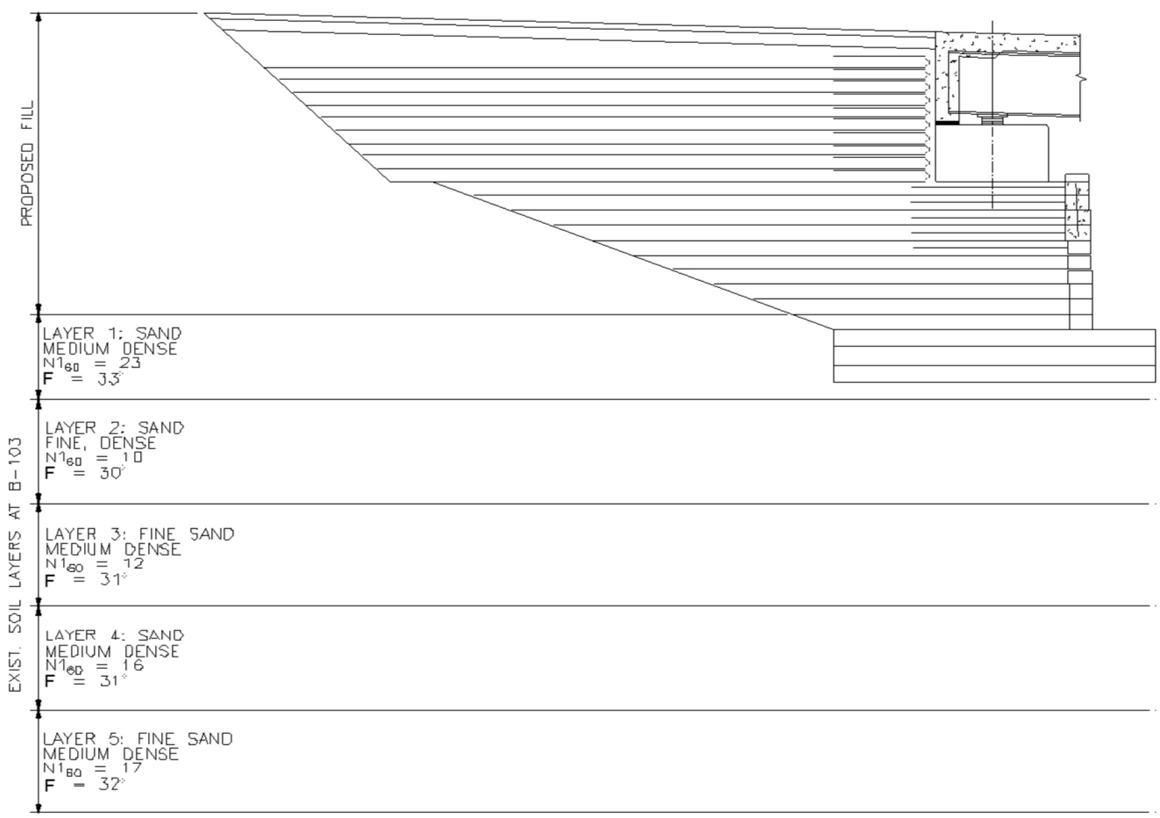
Service Limit State

z	$\sigma_{h,W}$	α_b	β_b	$\sigma_{h,bridge,eq}$	$\sigma_{h,rb}$	$\sigma_{h,t}$	σ_h	T_{req}	$T_{@\varepsilon\ =\ 2\%}$	
0.67 ft	0.013 ksf	2.59	-1.30	0.436 ksf	0.226 ksf	0.071 ksf	0.746 ksf	0.937 klf	1.25 klf	OK
1.33 ft	0.026 ksf	2.12	-1.06	0.416 ksf	0.226 ksf	0.071 ksf	0.739 ksf	0.928 klf	1.25 klf	OK
2.00 ft	0.039 ksf	1.74	-0.87	0.381 ksf	0.226 ksf	0.071 ksf	0.718 ksf	0.902 klf	1.25 klf	OK
2.67 ft	0.053 ksf	1.46	-0.73	0.342 ksf	0.226 ksf	0.071 ksf	0.692 ksf	0.870 klf	1.25 klf	OK
3.33 ft	0.066 ksf	1.24	-0.62	0.305 ksf	0.226 ksf	0.071 ksf	0.668 ksf	0.839 klf	1.25 klf	OK
4.00 ft	0.079 ksf	1.07	-0.54	0.273 ksf	0.226 ksf	0.071 ksf	0.648 ksf	0.815 klf	1.25 klf	OK
4.67 ft	0.092 ksf	0.94	-0.47	0.245 ksf	0.226 ksf	0.071 ksf	0.634 ksf	0.796 klf	1.25 klf	OK
5.33 ft	0.105 ksf	0.84	-0.42	0.221 ksf	0.226 ksf	0.071 ksf	0.623 ksf	0.783 klf	1.25 klf	OK
6.00 ft	0.118 ksf	0.75	-0.38	0.201 ksf	0.226 ksf	0.071 ksf	0.616 ksf	0.775 klf	1.25 klf	OK
6.67 ft	0.132 ksf	0.68	-0.34	0.184 ksf	0.226 ksf	0.071 ksf	0.612 ksf	0.770 klf	1.25 klf	OK
7.33 ft	0.145 ksf	0.63	-0.31	0.170 ksf	0.226 ksf	0.071 ksf	0.611 ksf	0.768 klf	1.25 klf	OK
8.00 ft	0.158 ksf	0.58	-0.29	0.157 ksf	0.226 ksf	0.071 ksf	0.612 ksf	0.769 klf	1.25 klf	OK
8.67 ft	0.171 ksf	0.53	-0.27	0.146 ksf	0.226 ksf	0.071 ksf	0.614 ksf	0.772 klf	1.25 klf	OK
9.33 ft	0.184 ksf	0.50	-0.25	0.137 ksf	0.226 ksf	0.071 ksf	0.617 ksf	0.776 klf	1.25 klf	OK
10.00 ft	0.197 ksf	0.47	-0.23	0.128 ksf	0.226 ksf	0.071 ksf	0.622 ksf	0.782 klf	1.25 klf	OK
10.67 ft	0.210 ksf	0.44	-0.22	0.121 ksf	0.226 ksf	0.071 ksf	0.628 ksf	0.789 klf	1.25 klf	OK
11.33 ft	0.224 ksf	0.41	-0.21	0.114 ksf	0.226 ksf	0.071 ksf	0.634 ksf	0.797 klf	1.25 klf	OK
12.00 ft	0.237 ksf	0.39	-0.20	0.108 ksf	0.226 ksf	0.071 ksf	0.641 ksf	0.806 klf	1.25 klf	OK
12.67 ft	0.250 ksf	0.37	-0.19	0.103 ksf	0.226 ksf	0.071 ksf	0.649 ksf	0.816 klf	1.25 klf	OK
13.33 ft	0.263 ksf	0.35	-0.18	0.098 ksf	0.226 ksf	0.071 ksf	0.657 ksf	0.826 klf	1.25 klf	OK
14.00 ft	0.276 ksf	0.34	-0.17	0.093 ksf	0.226 ksf	0.071 ksf	0.666 ksf	0.837 klf	1.25 klf	OK
14.67 ft	0.289 ksf	0.32	-0.16	0.089 ksf	0.226 ksf	0.071 ksf	0.675 ksf	0.849 klf	1.25 klf	OK
15.33 ft	0.303 ksf	0.31	-0.15	0.085 ksf	0.226 ksf	0.071 ksf	0.685 ksf	0.860 klf	1.25 klf	OK
16.00 ft	0.316 ksf	0.29	-0.15	0.082 ksf	0.226 ksf	0.071 ksf	0.694 ksf	0.873 klf	1.25 klf	OK
16.67 ft	0.329 ksf	0.28	-0.14	0.079 ksf	0.226 ksf	0.071 ksf	0.704 ksf	0.885 klf	1.25 klf	OK
17.33 ft	0.342 ksf	0.27	-0.14	0.076 ksf	0.226 ksf	0.071 ksf	0.714 ksf	0.898 klf	1.25 klf	OK
18.00 ft	0.355 ksf	0.26	-0.13	0.073 ksf	0.226 ksf	0.071 ksf	0.725 ksf	0.911 klf	1.25 klf	OK
18.67 ft	0.368 ksf	0.25	-0.13	0.070 ksf	0.226 ksf	0.071 ksf	0.735 ksf	0.924 klf	1.25 klf	OK
19.33 ft	0.381 ksf	0.24	-0.12	0.068 ksf	0.226 ksf	0.071 ksf	0.746 ksf	0.938 klf	1.25 klf	OK
20.00 ft	0.395 ksf	0.24	-0.12	0.066 ksf	0.226 ksf	0.071 ksf	0.757 ksf	0.952 klf	1.25 klf	OK
20.67 ft	0.408 ksf	0.23	-0.11	0.064 ksf	0.226 ksf	0.071 ksf	0.768 ksf	0.966 klf	1.25 klf	OK
21.33 ft	0.421 ksf	0.22	-0.11	0.062 ksf	0.226 ksf	0.071 ksf	0.779 ksf	0.980 klf	1.25 klf	OK
21.54 ft	0.425 ksf	0.22	-0.11	0.061 ksf	0.226 ksf	0.071 ksf	0.783 ksf	0.984 klf	1.25 klf	OK

References:
(1) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018

The proposed Span 2 - West Abutment is assumed to be founded on the existing granular soil defined in boring B-103. A sketch of the existing soil layers in relation to the proposed abutment is shown below.

SPAN 2 – WEST ABUTMENT



Note that the proposed bottom of footing elevation for the Span 1 - West Abutment is 443.46 ft. Therefore, the footing will be founded on layers of medium dense to dense silty sand.

Abutment Geometry

Footing Width, B =	14.00 ft			See "Abutment Design"
Abutment Length, L =	34.19 ft			See "Abutment Design"
e_B =	2.39 ft			See "Abutment Design"
B' =	9.21 ft =	14.00 ft -	(2.00 x	2.39 ft)

Settlement Analyses

Settlment Analyses per (1) 10.6.2.4

Total settlement, including elastic, consolidation, and secondary components may be taken as:

$$S_t = S_e + S_c + S_s$$ (Ref 1 - Eq. 10.6.2.4.1-1)

Final Design - Abutment 3 - Settlement VTRANS D37 IM 091-1(68)

For cohesionless soils, only elastic settlement is typically considered, therefore $S_t = S_e$

Elastic settlement can be approximated using either the elastic half-space method or the empirical Hough method. Both methods are calculated here for reference.

Elastic Half-Space Method:

$$S_e = \frac{[q_o(1 - v^2)\sqrt{A'}]}{144 E_s \beta_z}$$ (Ref 1 - Eq. 10.6.2.4.2-1)

Where:

q_o = applied vertical stress (ksf)

A' = effective area of footing (ft^2)

E_s = Young's modulus of soil taken from Article 10.4.6.3

B_z = shape factor taken as specified in table 10.6.2.4.2-1

v = Poisson's Ratio, taken as specified in Article 10.4.6.3

P_{SS} =	361 kips	(total vertical load on abutment, Service 1. See "Abutment Design")			
P_W =	889 kips =	26009.6 plf x	34.19 ft	0.001	
P_{RSF} =	138 kips =	4025 plf x	34.19 ft	0.001	
P_{FACE} =	64 kips =	1870 plf x	34.19 ft	0.001	
P_{LL} =	226 kips				
P_{LS} =	43 kips =	250.00 psf x	5.08 ft x	34.19 ft x	0.001
P =	1720 kips				

A' = B'L' = 315 ft² = 9.21 ft x 34.19 ft

q_o = P/A' = 5.5 ksf = 1720 kips/ 315 ft²

E_s = 4.17 ksi (1) Table C10.4.6.3-1, Loose/Medium Dense Sand

v = 0.28 (1) Table C10.4.6.3-1, Loose/Medium Dense Sand

L/B = 3.7107649 = 34.19 ft/ 9.21 ft

Footing Type = Flexible

βz = 1.162

Se = 0.13 ft = $\frac{5.5 \text{ ksf} \quad (1 - \quad 0.08) \text{ x} \quad 17.75 \text{ ft}}{(144 \text{ x} \quad 4.17 \text{ x} \quad 1.162)}$

Se = 1.54 in

Final Design - Abutment 3 - Settlement VTRANS D37 IM 091-1(68)

While the elastic half space method should provide reasonably accurate results, calculate total settlement using the Hough method for comparison.

Empirical Hough Method:

$$S_e = \sum_{i=1}^{n} \Delta H_i$$ (Ref 1 - Eq. 10.6.2.4.2-2)

Where: $$\Delta H_i = H_c \frac{1}{C'} \log\left(\frac{\sigma_o' + \Delta\sigma_v}{\sigma_o'}\right)$$ (Ref 1 - Eq. 10.6.2.4.2-3)

n = number of soil layers within zone of stress influence of the footing

H_c = initial height of each layer I (ft)

C' = bearing capacity index from Figure 10.6.2.4.2-1

σ'_o = initial vertical effective stress at the midpoint of layer I (ksf)

$\Delta\sigma_v$ = increase in vertical stress at the midpoint of layer I (ksf)

Depth of bottom of footing below grade, d =	4.54 ft	See Soil Properties - B105
Water Table Depth =	8 ft	
Soil Unit Weight =	0.125 kcf	
Water Unit Weight =	0.062 kcf	

Increase in vertical stress at point directly below footing is equal to the applied vertical stress, qo (ksf). Increase in vertical stress at a depth of z below the bottom of footing are assumed equal to applied load P divided by effective area at point of interest calculated assuming a 2:1 distribution slope, therefore A' = (B'+z)(L'+z)

Layer	Depth to Bottom of Layer	H_c	Midpoint Depth Below Footing, z	Sample Midpoint	h_1	h_2
1	4.50 ft	4.50 ft	0.00 ft	3.50	3.50 ft	0.00 ft
2	10.50 ft	6.00 ft	2.98 ft	7.50 ft	7.50 ft	0.00 ft
3	14.50 ft	4.00 ft	7.96 ft	12.50 ft	8.00 ft	4.50 ft
4	19.50 ft	5.00 ft	12.46 ft	17.00 ft	8.00 ft	9.00 ft
5	24.50 ft	5.00 ft	17.46 ft	22.00 ft	8.00 ft	14.00 ft
6	33.50 ft	9.00 ft	24.46 ft	29.00 ft	8.00 ft	21.00 ft
7	41.00 ft	7.50 ft	32.71 ft	37.25 ft	8.00 ft	29.25 ft

Final Design - Abutment 3 - Settlement VTRANS D37 IM 091-1(68)

Layer	Depth to Bottom of Layer	σ'_v	A'	$\Delta\sigma_v$ (P/A')	$N1_{60}$	*C'	ΔH
1	4.50 ft	0.438 ksf	315 ft²	5.5 ksf	23		-
2	10.50 ft	0.938 ksf	453 ft²	3.8 ksf	10	55	0.08 ft
3	14.50 ft	1.844 ksf	724 ft²	2.4 ksf	13	60	0.02 ft
4	19.50 ft	2.688 ksf	1011 ft²	1.7 ksf	15	62	0.02 ft
5	24.50 ft	3.626 ksf	1378 ft²	1.2 ksf	17	70	0.01 ft
6	33.50 ft	4.940 ksf	1975 ft²	0.9 ksf	29	100	0.01 ft
7	41.00 ft	6.487 ksf	2805 ft²	0.6 ksf	65	170	0.00 ft
						Total =	0.14 ft
						Total =	1.62 in

*In Figure 10.6.2.4.2-1 Well graded silty sand and gravel

Soil Properties - Abutment 4 (B-106) VTRANS D37 IM 091-1(68)

References:
(1) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018
(2) Geotechnical Data Report, Teracon Consultants Inc., March 27. 2019

g_w = 0.062 kcf
g_{sat} = 0.125 kcf

Water Table = 26.00 ft (2)
Ground EL. At Boring = 457.00
BOF EL. = 442.46

Correct Blow Count for Overburden Pressure

	To Depth (ft)	h_1 (ft)	h_2 (ft)	σ'_v (ksf)	C_N	N blows/ft	$N1 = C_N N$ blows/ft	N_{60} = (ER/60%)N blows/ft	$N1_{60} = C_N N_{60}$ blows/ft
	2.00	2.00	0.00	0.25	1.70	9.50	16.12	12.67	21.50
	8.00	8.00	0.00	1.00	1.23	8.00	9.87	10.67	13.16
	12.00	12.00	0.00	1.50	1.10	10.00	10.98	13.33	14.64
BOF	16.00	16.00	0.00	2.00	1.00	8.50	8.52	11.33	11.35
	21.00	21.00	0.00	2.63	0.91	16.50	15.03	22.00	20.04
	30.50	26.00	4.50	3.53	0.81	23.50	19.07	31.33	25.43
	40.50	26.00	14.50	4.16	0.76	36.00	27.25	48.00	36.34
	50.50	26.00	24.50	4.78	0.71	65.50	46.52	87.33	62.02
	60.50	26.00	34.50	5.41	0.67	31.50	21.07	42.00	28.10

h_1 = depth above water table
h_2 = depth below water table
$\sigma'_v = gh_1 + g' h_2$
$g' = g_{sat} - g_w$
$C_N = 0.77 \log_{10}(40 / \sigma'_v) < 2$ (1) 10.4.6.2.4.-1

Correct for Hammer Efficiency

N_{60} = (ER/60%)N (1) 10.4.6.2.4-2
ER = 0.80
N_{60} = 1.33 = 0.80/ 0.60
$N1_{60}$ = $C_N N_{60}$ (1) 10.4.6.2.4-3

Soil Properties - Abutment 4 (B-106) VTRANS D37 IM 091-1(68)

Calculate Drained Friction Angle

Table 10.4.6.2.4-1—Correlation of *SPT* $N1_{60}$ Values to Drained Friction Angle of Granular Soils (modified after Bowles, 1977)

$N1_{60}$	ϕ_f
<4	25–30
4	27–32
10	30–35
30	35–40
50	38–43

Use middle values of range:

$N1_{60}$	ϕ_f
<4	25.00
4.00	27.00
10.00	30.00
30.00	35.00
50.00	38.00

In determining the internal friction angle for soil below the footing, consider the soil up to two times the footing width.

Footing Width = 14.00 ft
Soil Depth = 28.00 ft = 2.00 x 14.00 ft

Footing Depth = 14.54

	Depth (ft)	$N1_{60}$	$N1_{60\ low}$	$N1_{60\ high}$	$\phi_{f\ low}$	$\phi_{f\ high}$	ϕ_f		
	2.00	21.50	10.00	30.00	30.00	35.00	32.87	Use ϕ_f =	31.00
	8.00	13.16	10.00	30.00	30.00	35.00	30.79		
	12.00	14.64	10.00	30.00	30.00	35.00	31.16		
BOF	16.00	11.35	10.00	30.00	30.00	35.00	30.34		
	21.00	20.04	10.00	30.00	30.00	35.00	32.51	Use ϕ_f =	32.00
	30.50	25.43	10.00	30.00	30.00	35.00	33.86		

References:

1) GRS-IBS Interim Implementation Guide, FHWA, June 2018

2) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018

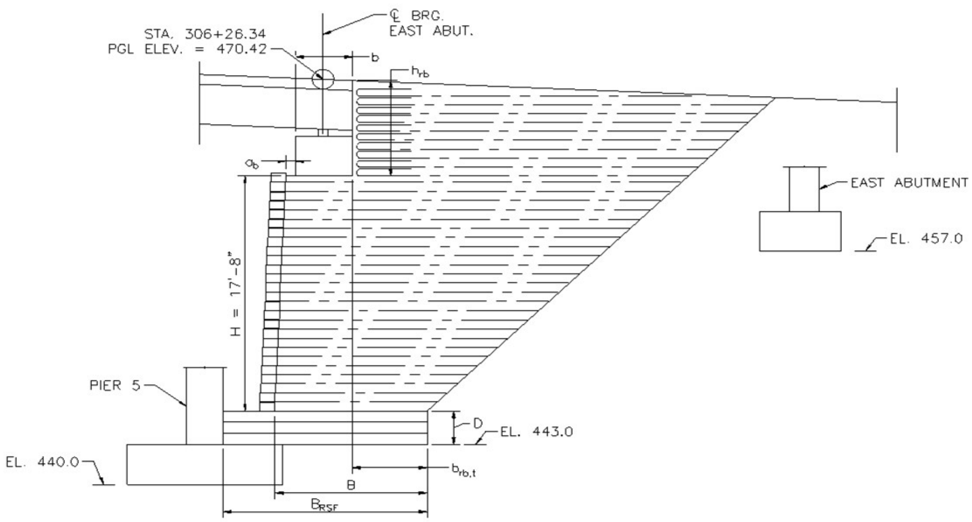
Material Properties

γ_f =	115.00 pcf	(Reinforced Fill)				
ϕ (Reinforced Soil) =	45	degrees				
K_a (Reinforced Soil) =	0.17 =		$\tan^2($	45 -	(45/	2))

Fill retained is in-situ soil.

γ_b =	125.00 pcf	(Retained Fill)				
ϕ (Existing Soil Above Footing) =	31.00	degrees				
K_{ab} =	0.32 =		$\tan^2($	45 -	(31/	2))

Geometry



Bridge Span Length =	85.94 ft			
Height of E. Abut. =	19.04 ft			
H =	19.04 ft			
Superstructure Depth =	4.39 ft			
h_{rb} =	7.03 ft			
B =	10.5 ft			
b =	4.75 ft			
Toe Length, X_{RSF} =	2.50 ft			
Total Width, B_{RSF} =	14 ft			
$b_{rb,t}$ =	5.08 ft =	10.5 ft -	0.67 ft -	4.75 ft
Depth of RSF, D =	2.50 ft			
Setback Distance, a_b =	0.67 ft			
L =	36.70 ft			

h_{block} =	8.00 in	
D_{block} =	12.00 in	
L_{block} =	18.00 in	
Weight =	85 lbs	per block

Reinforcement

S_v = 0.67 ft
d_{max} = 0.75 in
T_f = 5.90 klf

Load Factors
Per (3) Table 3.4.1-1 and Table 3.4.1-2.

γ_p (DC, Min.) =	0.90	3) Table 3.4.1-2
γ_p (DC, Max.) =	1.25	3) Table 3.4.1-2
γ_p (DW, Min.) =	0.65	3) Table 3.4.1-3
γ_p (DW, Max.) =	1.50	3) Table 3.4.1-4
γ_p (EH, Max) =	1.50	3) Table 3.4.1-2
γ_p (EH, Min.) =	0.90	3) Table 3.4.1-2
γ_p (EV, Min.) =	1.00	3) Table 3.4.1-2
γ_p (EV, Max.) =	1.35	3) Table 3.4.1-2
Factor (LS) =	1.75	3) Table 3.4.1-1
Factor (LL) =	1.75	3) Table 3.4.1-1

Calculate Average Height of Precast Beam Seat

Min. Height of Cap =	1.50 ft			
Cross Slope =	0.0625			
Cap Length =	36.70 ft			
Max. Height =	3.79 ft =	1.50 ft +	(36.70 ft x	0.0625)
Average Height =	2.65 ft			

Calculate Vertical Loads and Applied Pressures

Calculate q_{DL}

$$q_{DL} = \frac{Q_{DL}}{bL}$$

	Total DC Rxn	No. of Girders	DC1	DC2
N. Ext.	47.53 k =	1 x	(42.24 k +	5.29 k)
Typ. Int.	147.12 k =	3 x	(43.75 k +	5.29 k)
S. Ext.	41.32 k =	1 x	(36.03 k +	5.29 k)
Total Girder DC Reaction =	235.97 k			

	Total DW Rxn	No. of Girders	DW
N. Ext.	7.26 k =	1 x	7.26 k)
Typ. Int.	21.78 k =	3 x	7.26 k)
S. Ext.	7.26 k =	1 x	7.26 k)
Total Girder DW Reaction =	36.30 k		

Assume a concrete end block integral with the steel beam that is 3'-1" high x 12" wide x full width of abutment.

End Block Weight = 16.97 k = 3.083 ft x 1.00 ft x 36.70 ft x 0.150 kcf

Assume concrete deck beyond CL of bearing is equal to 1.60 SF, per AutoCAD

Additional Deck Weight = 8.81 k = 1.60 ft² x 0.150 kcf x 36.70 ft

Assume that girder extends additional 2'-0" beyond the CL of Bearing

Top Flange =	0.10 ft² =	16 in x	0.875 in x	0.007
Web =	0.12 ft² =	34 in x	0.5 in x	0.007
Bottom Flange =	0.14 ft² =	16 in x	1.25 in x	0.007
Total Steel Area =	0.35 ft²			

Additional Weight of Steel =	1.74 k =	0.35 ft² x	0.49 kcf x	2.00 ft x	5 girders	
Weight of Beam Seat =	69.21 k =	2.65 ft x	4.75 ft x	36.70 ft x	0.15 kcf	
Total DC Reaction (Unfactored) =	332.70 k =	235.97 k +	16.97 k +	8.81 k +	1.74 k +	69.21 k
Total DW Reaction (Unfactored) =	36.30 k					
q_{DC} =	1908.50 psf =	(332.7 kips x	1000)/	(4.75 ft x	36.70 ft)	
q_{DW} =	208.23 psf =	(36.3 kips x	1000)/	(4.75 ft x	36.70 ft)	

Calculate q_{LL}

Calculate Live Load Reactions Per Barrel

Roadway Width = 27.00 ft
Number of Trucks = 2
Impact = 1.33

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

Reaction (Single Truck) = 64.18 kips = $\dfrac{(32 \text{ kips} \times 85.94 \text{ ft}) + (32 \text{ kips} \times 71.94 \text{ ft}) + (8 \text{ kips} \times 57.94 \text{ ft})}{85.94 \text{ ft}}$

Lane Load Reaction (Single Truck) = 27.5 kips = 0.5 x 0.64 klf x 85.94 ft

Reaction - 1 Lane Loaded = 135.43 kips = 1.2 x ((64.18 kips x 1.33) + 27.5)
Reaction - 2 Lanes Loaded = 225.72 kips = 1 x ((2 x 64.18 kips x 1.33) + (2 x 27.5 kips))
Max. Reaction = 225.72 kips

Bridge LL Reaction = 225.72 kips

$$q_{LL} = \frac{Q_{LL}}{bL}$$

q_{LL} = 1294.82 psf = (225.72 kips x 1000)/ (4.75 ft x 36.70 ft)

Calculate Traffic Surcharge, q_T

Per (3) 3.11.6.4-1, the equivalent height of soil acting as a surcharge load shall be determined as follows:

Table 3.11.6.4-1—Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic

Abutment Height (ft)	h_{eq} (ft)
5.0	4.0
10.0	3.0
≥20.0	2.0

Note that linear interpolation shall be used for intermediate wall heights.

H = 21.54 ft
h_{eq} = 2.00 ft
γ_r = 125.00 pcf

$$q_t = h_{eq}\gamma_b$$

q_T = 250.00 psf = 2.00 ft x 125.00 pcf

Calculate Road Base Surchage, q_{rb}

γ_{rb} = 115.00 pcf

$$q_{rb} = H_{rb}\gamma_{rb}$$

q_{rb} = 808.71 psf = 7.03 ft x 115.00 pcf

Calculate Weight of GRS Abutment

$$W = BH\gamma_r$$

H = 19.04 ft
B = 10.5 ft
γ_r = 115.00 pcf

W = 22990.80 plf = 19.04 ft x 10.5 ft x 115.00 pcf

Calculate Weight of RSF

$$W_{RSF} = B_{RSF} D_{RSF} \gamma_r$$

B =	14	ft
D =	2.5	ft
γ_r =	125.00	pcf

W_{RSF} =	4025.00 plf =	14 ft x	2.5 ft x	115.00 pcf	

Calculate Weight of Facing

N_{block} =	28.56 =	19.04 ft/	0.67 ft
N_{block} =	29		

$$W_{face} = N_{block} \frac{W_{block}}{L_{block}}$$

W_{face} =	1643.33 plf =	29 x	(85 lbs/	(18.00 in/	12)

Summary of Applied Loads

q_{DC} =	1908.50 psf
q_{DW} =	208.23 psf
q_{LL} =	1294.82 psf
q_t =	250.00 psf
q_{rb} =	808.71 psf
W =	22990.80 plf
W_{RSF} =	4025.00 plf
W_{face} =	1643.33 plf

Check Beam Seat Pressure

Per (1) 4.3.5.4, the service bearing pressure should be targeted to around 4 ksf.

DC Reaction =	332.70 kips
DW Reaction =	36.30 kips
LL Reaction =	225.72 kips
Total =	594.72 kips

b =	4.75 ft
L =	36.70 ft

q_{seat} =	3.41 ksf =	594.72 kips/	(4.75 ft x	36.70 ft)

OK

Calculte Direct Sliding Effects at RSF/GRS Interface

Calculate Sliding per the (1) and (3) Strength I Load Combination

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2$$ (1) Eq. 9

$$F_{rb} = q_{rb} K_{ab} H$$ (1) Eq. 10

$$F_t = q_t K_{ab} H$$ (1) Eq. 11

$$F_R = \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t$$ (1) Eq. 12

F_b =	7252.67 lbs =	0.5 x	125.00 pcf x	0.32 x	19.04 ft^ 2	
F_{rb} =	4928.85 lbs =	808.71 psf x	0.32 x	19.04 ft		
F_t =	1523.67 lbs =	250.00 psf	0.32 x	19.04 ft		

F_R = 20.94 klf = (1.5 x (7252.67 lbs + 4928.85 lbs) + (1.75 1523.67 lbs))/ 1000

$$W_{T,R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EV\,MIN}(q_{rb} b_{rb,t})$$ 4

$W_{T,R}$ = 37.38 klf = ((1.0 x 22990.80 plf) + (0.9 x 1908.50 psf x 4.75 ft) + (0.65 x 208.23 psf x 4.75 ft) +
(0.9 x 1643.33 plf) + (1.0 x 808.71 psf x 5.08 ft))/ 1000

Assume that μ = 2/3*tanϕ, per (1) C.1.6.1.1

μ = 0.67 = 0.67 x tan(45)

$$R_R = \Phi_\tau (W_{T,R}\mu)$$ (1) Eq. 13

R_R = 24.92 klf = 37.38 klf x 0.667

OK

Calculate Direct Sliding at Base of RSF

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2$$ (3) Eq. 17

$$F_{rb,RSF} = q_{rb} K_{ab}(H + D_{RSF})$$ (3) Eq. 18

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF})$$ (3) Eq. 19

$F_{b,RSF}$ = 9.28 klf = (0.5 x 125.00 pcf x 0.32 x (19.04 ft + 2.5)^ 2)/ 1000
$F_{rb,RSF}$ = 5.58 klf = 808.71 psf x 0.32 x (19.04 ft + 2.5 ft))/ 1000
$F_{t,RSF}$ = 1.72 klf = 250.00 psf x 0.32 x (19.04 ft + 2.5 ft))/ 1000

$$F_{R,RSF} = \gamma_{EH\,MAX}(F_{b,RSF} + F_{rb,RSF}) + \gamma_{LS} F_{t,RSF}$$ (1) Eq. 20

$F_{R,RSF}$ = 25.30 klf = 1.5 x (9.28 + 5.58) + (1.75 x 1.72 klf)

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\,MIN} W_{RSF}$$ (1) Eq. 22

$/_{T,R,RSF}$ = 41.41 klf = 37.38 klf + (1.0 x 4.03 klf)

$$R_{R,RSF} = \Phi_\tau(W_{T,R,RSF}\mu_{RSF})$$ (1) Eq. 21

Note that ϕ = 1.0, per (1) 4.3.6.1.2

Per (2), and the previous calculation, the angle of internal friction beneath the RSF is equal to 32 degrees

$R_{R,\ RSF}$ = 25.87 klf = 1.0 x 41.41 klf x tan(32)

OK

Calculate External Bearing Resistance

Calculate V_R

Per (1) Eq. 26,

$$\sum V_R = \gamma_{EV\,MAX}(W) + \gamma_{EV\,MAX}(W_{RSF}) + \gamma_{DC\,MAX}(W_{face}) + \gamma_{LS}(q_t b_{rb,t}) + \gamma_{EH\,MAX}(q_{rb} b_{rb,t}) + \gamma_{DC\,MAX}(q_{DL} b) + \gamma_{LS}(q_{LL} b)$$

Note that $\gamma_{EV\ Max}$ is used in place of $\gamma_{EH\ Max}$ to factor q_{rb}*$b_{rb,t}$ since this is a summation of vertical reactions.

W =	31.04 klf =	1.35 x	22990.80 plf/	1000	
W_{RSF} =	5.43 klf =	1.35 x	4025.00 plf/	1000	
W_{FACE} =	2.05 klf =	1.25 x	1643.33 plf/	1000	
Q_t =	2.22 klf =	1.75 x	(250.00 psf x	5.08 ft)/	1000
Q_{rb} =	5.55 klf =	1.35 x	(808.71 psf x	5.08 ft)/	1000
Q_{DC} =	11.33 klf =	1.25 x	(1908.50 psf x	4.75 ft)/	1000
Q_{DW}=	1.48 klf =	1.5 x	(208.23 psf x	4.75 ft)/	1000
Q_{LL} =	10.76 klf =	1.75 x	(1294.82 psf	4.75 ft)/	1000
Total =	69.88 klf				

Calculate $M_{D,R}$

Per (1) Eq. 28

$$\sum M_{D,R} = \gamma_{EH\,MAX} F_{b,RSF}\left(\frac{H + D_{RSF}}{3}\right) + \gamma_{LS} F_{t,RSF}\left(\frac{H + D_{RSF}}{2}\right) + \gamma_{EH\,MAX} F_{rb,RSF}\left(\frac{H + D_{RSF}}{2}\right)$$

$F_{b,\ RSF}$ =	99.97 k-ft/ ft =	1.5 x	9.28 klf x	(0.33 x	(19.04 ft +	2.5 ft))
$F_{t,\ RSF}$ =	32.49 k-ft/ ft =	1.75 x	1.72 klf x	(0.50 x	(19.04 ft +	2.5 ft))
$F_{rb,\ RSF}$ =	90.08 k-ft/ ft =	1.5 x	5.58 klf x	(0.50 x	(19.04 ft +	2.5 ft))
Total =	222.54 k-ft/ ft					

Note that $M_{D,R}$ is taken about the bottom center of the width of the RSF.

Calculate $M_{R,R}$

Per (1) Eq. 29

$$\sum M_{R,R} = (\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b)\left[\left(\frac{b}{2} + a_b\right) - \left(\frac{B_{RSF}}{2} - x_{RSF} - b_{block}\right)\right] + \left(\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t}\right)\left(\frac{B_{RSF}}{2} - \frac{b_{rb}}{2}\right) + \gamma_{EV\,MAX} W\left(\frac{B_{RSF}}{2} - \frac{B}{2}\right) + \gamma_{DC\,MAX} W_{face}\left(B + \frac{b_{block}}{2} - \frac{B_{RSF}}{2}\right)$$

$_{DL}$+Q_{LL} =	-10.81 k-ft =	((1.25 x	1.91 ksf) +	(1.50 x	0.21 ksf) +	(1.75 x	1.29 ksf)) x 4.75 ft x	
		((2.375 ft x +	0.67 ft) -	(7 ft -	2.5 ft -	1.00 ft))		
Q_t+Q_{rb} =	34.66 k-ft =	((1.75 x	0.25 klf x	5.08 ft) +	(1.35 x	0.809 klf x	5.08 ft)) x	
		(7 ft -	2.54 ft)					
W =	54.32 k-ft =	1.35 x	22.99 klf x	(7 ft -	5.25 ft)			
W_{Face} =	-7.76 k-ft =	1.35 x	1.64 klf x	((2.5 ft +	1 ft) -	7 ft)		
Total =	70.40 k-ft/ ft							

Note that $M_{R,R}$ is taken about the bottom center of the width of the RSF.

Calculate Eccentricity, $e_{B,R}$

Per (1) Eq. 27

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R}$$

$e_{B,R}$ = 2.18 ft = (222.54 k-ft/ ft - 70.40 k-ft/ ft) / 69.88 klf

Calculate $\sigma_{v,base,R}$

Since $e_{B,R}$ = negative, set equal to zero in calcaulation of base bearing pressure.

Per (1) Eq. 25,

$$\sigma_{v,base,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}}$$

$\sigma_{v,base,R}$ = 7.24 ksf = 69.88 klf/ (14 ft - (2 x 2.18 ft)

Per bearing capacity calculation, factored bearing capacity = 12.71 ksf

OK

Calculate Internal Bearing Resistance

Per (1), Eq. 35

$$q_{n,an} = \left[0.7^{\left(\frac{S_v}{6d_{max}}\right)} \frac{T_f}{S_v}\right] K_{pr}$$

$q_{n,an}$ = nominal bearing resistance of the GRS abutment using the analytical method.
S_v = reinforcement spacing.
d_{max} = maximum grain size.
T_f = ultimate reinforcement strength.
K_{pr} = coefficient of passive earth pressure for the reinforced fill (calculated in equation 35).

K_{pr} = 5.83 = $\tan^2$(45 + (45/ 2))
$q_{n,an}$ = 27.36 ksf = [(0.7^ (0.67 ft/ (6 x 0.06 ft))) x (5.90 klf/ 0.67 in)] x 5.83
ϕ_{cap} = 0.45 (3) 4.3.7.2
$\phi_{cap} q_{n,an}$ = 12.31 ksf = 0.45 x 27.36 ksf

$$V_{applied,f} = \gamma_{DC\,MAX} q_{DL} + \gamma_{LL} q_{LL}$$

(1) Eq. 32

$V_{app,f}$ = 4.96 ksf = (1.25 x 1.91 ksf) + (1.50 x 0.21 ksf) + (1.75 x 1.29 ksf)

OK

Calculate and Check Deformations

Per (1), Eq. 37

$$q_{DL,allow\ @\varepsilon=1\%} = 0.2\left[0.7^{S_v/6d_{max}} \left(\frac{T_f}{S_v}\right) K_{pr}\right]$$

6^*d_{max} = 0.375 ft = 6 x 0.75 in x 0.083
T_f/S_v = 8.85 = 5.90 klf/ 0.67 ft
6^*d_{max} = 1.78 = 0.67 ft/ 0.375 ft

$q_{dl,\,all}$ = 5.47 ksf = 0.2 ((0.7^ 1.78 in) x 8.85 x 5.83 in)
q_{DL} = 1.91 ksf

OK

Calculate Lateral Strain

Per (1) 4.3.7.2.2, for the calculation of lateral strain, assume a vertical strain equal to 1% of the abutment height, H.

.01*H = 0.19 ft = 0.01 x 19.04 ft

$$D_L = \frac{2b_q D_v}{H}$$ (1) Eq. 38

D_L = 0.01 ft = (2 x (4.75 ft + 0.67 ft) x 0.02 ft)/ 19.04 ft
D_L = 0.11 in = 0.01 ft x 12

Per (1) 4.3.7.2.2, the total lateral strain should be limited to twice the vertical strain.

Strain = 0.38 ft = 2 x 0.19 ft

OK

Calculate Reinforcement Strength

q_{DC} = 1908.50 psf
q_{DW} = 208.23 psf
q_{LL} = 1294.82 psf
q_t = 250.00 psf
q_{rb} = 808.71 psf
W = 22990.80 psf
W_{RSF} = 4025.00 psf
W_{face} = 1643.33 psf

The evaulation of the abutment for the strength limit state is conducted according to (1) 4.3.7.3.1. and using the following equations:

Required Reinforcment:

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 40

Factored Total Lateral Pressure:

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f}$$ (3) Eq. 41

Factored Lateral Pressure due to weight of GRS:

$$\sigma_{h,W,f} = \gamma_{EH\ MAX}(\gamma_r z K_{ar})$$

Where:
$\gamma_{EH\ MAX}$ = maximum horizontal earth pressure load factor.
γ_r = unit weight of reinforced backfill.
z = depth from the top of the wall.
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

(3) Eq. 42

Factored Latreal Pressure due to equivalent bridge load:

$$\sigma_{h,bridge,f} = \frac{(\gamma_{DC\ MAX} q_{DL} + \gamma_{LL} q_{LL}) - (\gamma_{EH\ MAX} q_{rb} + \gamma_{LS} q_t)}{\pi}\left[\alpha_b + \sin(\alpha_b)\cos(\alpha_b) + 2\beta_b\right] K_{ar} \quad (43)$$

Where:
$\gamma_{DC\ MAX}$ = maximum DL load factor.
q_{DL} = superstructure DL pressure.
γ_{LL} = bridge LL surcharge load factor.
q_{LL} = bridge LL pressure.
q_{rb} = surcharge due to the structural backfill of the integrated approach (i.e., road base).
γ_{LS} = LL surcharge load factor.
q_t = roadway LL surcharge.
α_b = angle between wall face and projection of the midline of the bridge surcharge to the wall face found using equation 46 (see figure 23).
β_b = angle between the projections of the inner and outer edge lines of the bridge surcharge to the wall face found using equation 47 (see figure 23).

(3) Eq. 43

$$\sigma_{h,rb,f} = \gamma_{EH\,MAX} q_{rb} K_{ar}$$ (3) Eq. 44

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar}$$ (3) Eq. 45

$$\alpha_b = \tan^{-1}\left(\frac{b}{2z}\right) - \beta_b$$ (3) Eq. 46

$$\beta_b = \tan^{-1}\left(\frac{-b}{2z}\right)$$ (3) Eq. 47

Factored Reinforcement Strength:

$$T_{f,f} = \Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right) = 0.4T_f$$ (3) Eq. 48

Strength Limit State

z	$\sigma_{h,W,f}$	α_b	β_b	$\sigma_{h,bridge,f}$	$\sigma_{h,rb,f}$	$\sigma_{h,t,f}$	$\sigma_{h,f}$	$T_{req,f}$	$T_{f,f}$	
0.67 ft	0.020 ksf	2.59	-1.30	0.564 ksf	0.388 ksf	0.140 ksf	1.112 ksf	1.397 klf	2.36 klf	OK
1.33 ft	0.039 ksf	2.12	-1.06	0.538 ksf	0.388 ksf	0.140 ksf	1.106 ksf	1.390 klf	2.36 klf	OK
2.00 ft	0.059 ksf	1.74	-0.87	0.493 ksf	0.388 ksf	0.140 ksf	1.081 ksf	1.359 klf	2.36 klf	OK
2.67 ft	0.079 ksf	1.46	-0.73	0.443 ksf	0.388 ksf	0.140 ksf	1.050 ksf	1.320 klf	2.36 klf	OK
3.33 ft	0.099 ksf	1.24	-0.62	0.395 ksf	0.388 ksf	0.140 ksf	1.022 ksf	1.285 klf	2.36 klf	OK
4.00 ft	0.118 ksf	1.07	-0.54	0.353 ksf	0.388 ksf	0.140 ksf	1.000 ksf	1.256 klf	2.36 klf	OK
4.67 ft	0.138 ksf	0.94	-0.47	0.317 ksf	0.388 ksf	0.140 ksf	0.983 ksf	1.236 klf	2.36 klf	OK
5.33 ft	0.158 ksf	0.84	-0.42	0.286 ksf	0.388 ksf	0.140 ksf	0.972 ksf	1.222 klf	2.36 klf	OK
6.00 ft	0.178 ksf	0.75	-0.38	0.260 ksf	0.388 ksf	0.140 ksf	0.966 ksf	1.214 klf	2.36 klf	OK
6.67 ft	0.197 ksf	0.68	-0.34	0.238 ksf	0.388 ksf	0.140 ksf	0.964 ksf	1.212 klf	2.36 klf	OK
7.33 ft	0.217 ksf	0.63	-0.31	0.219 ksf	0.388 ksf	0.140 ksf	0.965 ksf	1.213 klf	2.36 klf	OK
8.00 ft	0.237 ksf	0.58	-0.29	0.203 ksf	0.388 ksf	0.140 ksf	0.968 ksf	1.217 klf	2.36 klf	OK
8.67 ft	0.257 ksf	0.53	-0.27	0.189 ksf	0.388 ksf	0.140 ksf	0.974 ksf	1.224 klf	2.36 klf	OK
9.33 ft	0.276 ksf	0.50	-0.25	0.177 ksf	0.388 ksf	0.140 ksf	0.981 ksf	1.233 klf	2.36 klf	OK
######	0.296 ksf	0.47	-0.23	0.166 ksf	0.388 ksf	0.140 ksf	0.990 ksf	1.244 klf	2.36 klf	OK
######	0.316 ksf	0.44	-0.22	0.156 ksf	0.388 ksf	0.140 ksf	1.000 ksf	1.257 klf	2.36 klf	OK
######	0.335 ksf	0.41	-0.21	0.147 ksf	0.388 ksf	0.140 ksf	1.011 ksf	1.271 klf	2.36 klf	OK
######	0.355 ksf	0.39	-0.20	0.140 ksf	0.388 ksf	0.140 ksf	1.023 ksf	1.286 klf	2.36 klf	OK
######	0.375 ksf	0.37	-0.19	0.133 ksf	0.388 ksf	0.140 ksf	1.036 ksf	1.302 klf	2.36 klf	OK
######	0.395 ksf	0.35	-0.18	0.126 ksf	0.388 ksf	0.140 ksf	1.049 ksf	1.319 klf	2.36 klf	OK
######	0.414 ksf	0.34	-0.17	0.120 ksf	0.388 ksf	0.140 ksf	1.063 ksf	1.336 klf	2.36 klf	OK
######	0.434 ksf	0.32	-0.16	0.115 ksf	0.388 ksf	0.140 ksf	1.078 ksf	1.354 klf	2.36 klf	OK
######	0.454 ksf	0.31	-0.15	0.110 ksf	0.388 ksf	0.140 ksf	1.093 ksf	1.373 klf	2.36 klf	OK
######	0.474 ksf	0.29	-0.15	0.106 ksf	0.388 ksf	0.140 ksf	1.108 ksf	1.392 klf	2.36 klf	OK
######	0.493 ksf	0.28	-0.14	0.102 ksf	0.388 ksf	0.140 ksf	1.123 ksf	1.412 klf	2.36 klf	OK
######	0.513 ksf	0.27	-0.14	0.098 ksf	0.388 ksf	0.140 ksf	1.139 ksf	1.432 klf	2.36 klf	OK
######	0.533 ksf	0.26	-0.13	0.094 ksf	0.388 ksf	0.140 ksf	1.155 ksf	1.452 klf	2.36 klf	OK
######	0.552 ksf	0.25	-0.13	0.091 ksf	0.388 ksf	0.140 ksf	1.172 ksf	1.473 klf	2.36 klf	OK
######	0.564 ksf	0.25	-0.12	0.089 ksf	0.388 ksf	0.140 ksf	1.181 ksf	1.485 klf	2.36 klf	OK

The evaulation of the abutment for the service limit state is conducted according to (1) 4.3.7.3.2. and using the following equations:

The nominal required reinforcement strength:

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 50

The lateral pressure within the GRS Abutment at a given dpeth and location:

$$\sigma_h = \sigma_{h,W} + \sigma_{h,bridge,eq} + \sigma_{h,rb} + \sigma_{h,t}$$ (3) Eq. 51

The lateral pressure due to the equivalent bridge load:

$$\sigma_{h,bridge,eq} = \frac{(q_{DL} + q_{LL}) - (q_{rb} + q_t)}{\pi}\left[\alpha_b + \sin(\alpha_b)\cos(\alpha_b + 2\beta_b)\right]K_{ar} \qquad (52)$$

Where:

q_{DL} = bridge DL pressure.
q_{LL} = bridge LL surcharge.
q_{rb} = surcharge due to the structural backfill of the integrated approach (i.e., road base).
q_t = roadway LL surcharge.

(3) Eq. 52

α_b = angle between wall face and projection of the midline of the bridge surcharge to the wall face found using equation 46 (see figure 23).
β_b = angle between the wall face and projection of the midline of the surcharge to the wall face found using equation 47 (see figure 23).
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

Lateral pressure due to the road base surcharge within GRS:

$$\sigma_{h,rb} = q_{rb} K_{ar}$$

(3) Eq. 53

Lateral pressure due to the traffic surchage within GRS:

$$\sigma_{h,t} = q_t K_{ar}$$

(3) Eq. 54

Service Limit State

z	$\sigma_{h,W}$	α_b	β_b	$\sigma_{h,bridge,eq}$	$\sigma_{h,rb}$	$\sigma_{h,t}$	σ_h	T_{req}	$T_{@\varepsilon = 2\%}$	
0.67 ft	0.013 ksf	2.59	-1.30	0.400 ksf	0.259 ksf	0.080 ksf	0.752 ksf	0.946 klf	1.25 klf	OK
1.33 ft	0.026 ksf	2.12	-1.06	0.382 ksf	0.259 ksf	0.080 ksf	0.747 ksf	0.939 klf	1.25 klf	OK
2.00 ft	0.039 ksf	1.74	-0.87	0.350 ksf	0.259 ksf	0.080 ksf	0.729 ksf	0.916 klf	1.25 klf	OK
2.67 ft	0.053 ksf	1.46	-0.73	0.315 ksf	0.259 ksf	0.080 ksf	0.706 ksf	0.888 klf	1.25 klf	OK
3.33 ft	0.066 ksf	1.24	-0.62	0.281 ksf	0.259 ksf	0.080 ksf	0.685 ksf	0.861 klf	1.25 klf	OK
4.00 ft	0.079 ksf	1.07	-0.54	0.251 ksf	0.259 ksf	0.080 ksf	0.668 ksf	0.840 klf	1.25 klf	OK
4.67 ft	0.092 ksf	0.94	-0.47	0.225 ksf	0.259 ksf	0.080 ksf	0.656 ksf	0.824 klf	1.25 klf	OK
5.33 ft	0.105 ksf	0.84	-0.42	0.203 ksf	0.259 ksf	0.080 ksf	0.647 ksf	0.814 klf	1.25 klf	OK
6.00 ft	0.118 ksf	0.75	-0.38	0.185 ksf	0.259 ksf	0.080 ksf	0.642 ksf	0.807 klf	1.25 klf	OK
6.67 ft	0.132 ksf	0.68	-0.34	0.169 ksf	0.259 ksf	0.080 ksf	0.640 ksf	0.804 klf	1.25 klf	OK
7.33 ft	0.145 ksf	0.63	-0.31	0.156 ksf	0.259 ksf	0.080 ksf	0.639 ksf	0.804 klf	1.25 klf	OK
8.00 ft	0.158 ksf	0.58	-0.29	0.144 ksf	0.259 ksf	0.080 ksf	0.641 ksf	0.806 klf	1.25 klf	OK
8.67 ft	0.171 ksf	0.53	-0.27	0.134 ksf	0.259 ksf	0.080 ksf	0.644 ksf	0.810 klf	1.25 klf	OK
9.33 ft	0.184 ksf	0.50	-0.25	0.125 ksf	0.259 ksf	0.080 ksf	0.649 ksf	0.815 klf	1.25 klf	OK
######	0.197 ksf	0.47	-0.23	0.118 ksf	0.259 ksf	0.080 ksf	0.654 ksf	0.822 klf	1.25 klf	OK
######	0.210 ksf	0.44	-0.22	0.111 ksf	0.259 ksf	0.080 ksf	0.660 ksf	0.830 klf	1.25 klf	OK
######	0.224 ksf	0.41	-0.21	0.105 ksf	0.259 ksf	0.080 ksf	0.667 ksf	0.839 klf	1.25 klf	OK
######	0.237 ksf	0.39	-0.20	0.099 ksf	0.259 ksf	0.080 ksf	0.675 ksf	0.848 klf	1.25 klf	OK
######	0.250 ksf	0.37	-0.19	0.094 ksf	0.259 ksf	0.080 ksf	0.683 ksf	0.858 klf	1.25 klf	OK
######	0.263 ksf	0.35	-0.18	0.090 ksf	0.259 ksf	0.080 ksf	0.692 ksf	0.869 klf	1.25 klf	OK
######	0.276 ksf	0.34	-0.17	0.000 ksf	0.259 ksf	0.080 ksf	0.615 ksf	0.773 klf	1.25 klf	OK
######	0.289 ksf	0.32	-0.16	0.000 ksf	0.259 ksf	0.080 ksf	0.628 ksf	0.790 klf	1.25 klf	OK
######	0.303 ksf	0.31	-0.15	0.000 ksf	0.259 ksf	0.080 ksf	0.641 ksf	0.806 klf	1.25 klf	OK
######	0.316 ksf	0.29	-0.15	0.000 ksf	0.259 ksf	0.080 ksf	0.655 ksf	0.823 klf	1.25 klf	OK
######	0.329 ksf	0.28	-0.14	0.000 ksf	0.259 ksf	0.080 ksf	0.668 ksf	0.839 klf	1.25 klf	OK
######	0.342 ksf	0.27	-0.14	0.000 ksf	0.259 ksf	0.080 ksf	0.681 ksf	0.856 klf	1.25 klf	OK
######	0.355 ksf	0.26	-0.13	0.000 ksf	0.259 ksf	0.080 ksf	0.694 ksf	0.872 klf	1.25 klf	OK
######	0.368 ksf	0.25	-0.13	0.000 ksf	0.259 ksf	0.080 ksf	0.707 ksf	0.889 klf	1.25 klf	OK
######	0.376 ksf	0.25	-0.12	0.063 ksf	0.259 ksf	0.080 ksf	0.778 ksf	0.978 klf	1.25 klf	OK

References:

1) GRS-IBS Interim Implementation Guide, FHWA, June 2018

2) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018

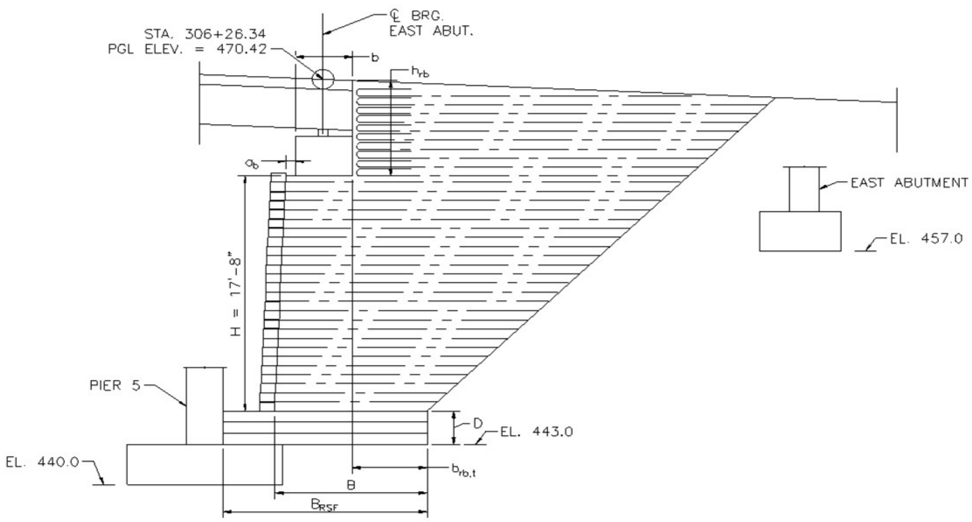
Material Properties

γ_f =	115.00 pcf	(Reinforced Fill)				
ϕ (Reinforced Soil) =	45	degrees				
K_a (Reinforced Soil) =	0.17 =		$\tan^2($	45 -	(45/	2))

Fill retained is in-situ soil.

γ_b =	125.00 pcf	(Retained Fill)				
ϕ (Existing Soil Above Footing) =	31.00	degrees				
K_{ab} =	0.32 =		$\tan^2($	45 -	(31/	2))

Geometry



Bridge Span Length =	85.94 ft			
Height of E. Abut. =	19.04 ft			
H =	19.04 ft			
Superstructure Depth =	4.39 ft			
h_{rb} =	7.03 ft			
B =	10.5 ft			
b =	4.75 ft			
Toe Length, X_{RSF} =	2.50 ft			
Total Width, B_{RSF} =	14 ft			
$b_{rb,t}$ =	5.08 ft =	10.5 ft -	0.67 ft -	4.75 ft
Depth of RSF, D =	2.50 ft			
Setback Distance, a_b =	0.67 ft			
L =	36.70 ft			
h_{block} =	8.00 in			
D_{block} =	12.00 in			
L_{block} =	18.00 in			
Weight =	85 lbs	per block		

Reinforcement

S_v =	0.67 ft
d_{max} =	0.75 in
T_f =	5.90 klf

Load Factors

Per (3) Table 3.4.1-1 and Table 3.4.1-2.

γ_p (DC, Min.) =	0.90	3) Table 3.4.1-2
γ_p (DC, Max.) =	1.25	3) Table 3.4.1-2
γ_p (DW, Min.) =	0.65	3) Table 3.4.1-3
γ_p (DW, Max.) =	1.50	3) Table 3.4.1-4
γ_p (EH, Max) =	1.50	3) Table 3.4.1-2
γ_p (EH, Min.) =	0.90	3) Table 3.4.1-2
γ_p (EV, Min.) =	1.00	3) Table 3.4.1-2
γ_p (EV, Max.) =	1.35	3) Table 3.4.1-2
Factor (LS) =	1.75	3) Table 3.4.1-1
Factor (LL) =	1.75	3) Table 3.4.1-1

Calculate Average Height of Precast Beam Seat

Min. Height of Cap =	1.50 ft			
Cross Slope =	0.0625			
Cap Length =	36.70 ft			
Max. Height =	3.79 ft =	1.50 ft +	(36.70 ft x	0.0625)
Average Height =	2.65 ft			

Calculate Vertical Loads and Applied Pressures

Calculate q_{DL}

$$q_{DL} = \frac{Q_{DL}}{bL}$$

	Total DC Rxn	No. of Girders	DC1	DC2
N. Ext.	47.53 k =	1 x	(42.24 k +	5.29 k)
Typ. Int.	147.12 k =	3 x	(43.75 k +	5.29 k)
S. Ext.	41.32 k =	1 x	(36.03 k +	5.29 k)
Total Girder DC Reaction =	235.97 k			

	Total DW Rxn	No. of Girders	DW
N. Ext.	7.26 k =	1 x	7.26 k)
Typ. Int.	21.78 k =	3 x	7.26 k)
S. Ext.	7.26 k =	1 x	7.26 k)
Total Girder DW Reaction =	36.30 k		

Assume a concrete end block integral with the steel beam that is 3'-1" high x 12" wide x full width of abutment.

End Block Weight =	16.97 k =	3.083 ft x	1.00 ft x	36.70 ft x	0.150 kcf

Assume concrete deck beyond CL of bearing is equal to 1.60 SF, per AutoCAD

Additional Deck Weight =	8.81 k =	1.60 ft² x	0.150 kcf x	36.70 ft

Assume that girder extends additional 2'-0" beyond the CL of Bearing

Top Flange =	0.10 ft² =	16 in x	0.875 in x	0.007
Web =	0.12 ft² =	34 in x	0.5 in x	0.007
Bottom Flange =	0.14 ft² =	16 in x	1.25 in x	0.007
Total Steel Area =	0.35 ft²			

Additional Weight of Steel =	1.74 k =	0.35 ft² x	0.49 kcf x	2.00 ft x	5 girders	
Weight of Beam Seat =	69.21 k =	2.65 ft x	4.75 ft x	36.70 ft x	0.15 kcf	
Total DC Reaction (Unfactored) =	332.70 k =	235.97 k +	16.97 k +	8.81 k +	1.74 k +	69.21 k
Total DW Reaction (Unfactored) =	36.30 k					

q_{DC} =	1908.50 psf =	(332.7 kips x	1000)/	(4.75 ft x	36.70 ft)
q_{DW} =	208.23 psf =	(36.3 kips x	1000)/	(4.75 ft x	36.70 ft)

Calculate q_{LL}

Calculate Live Load Reactions Per Barrel

Roadway Width = 27.00 ft
Number of Trucks = 2
Impact = 1.33

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

Reaction (Single Truck) = 64.18 kips = ((32 kips x 85.94 ft) + (32 kips x 71.94 ft) + (8 kips x 57.94 ft)) / 85.94 ft

Lane Load Reaction (Single Truck) = 27.5 kips = 0.5 x 0.64 klf x 85.94 ft

Reaction - 1 Lane Loaded = 135.43 kips = 1.2 x ((64.18 kips x 1.33) + 27.5)
Reaction - 2 Lanes Loaded = 225.72 kips = 1 x ((2 x 64.18 kips x 1.33) + (2 x 27.5 kips))
Max. Reaction = 225.72 kips

Bridge LL Reaction = 225.72 kips

$$q_{LL} = \frac{Q_{LL}}{bL}$$

q_{LL} = 1294.82 psf = (225.72 kips x 1000)/ (4.75 ft x 36.70 ft)

Calculate Traffic Surcharge, q_T

Per (3) 3.11.6.4-1, the equivalent height of soil acting as a surcharge load shall be determined as follows:

Table 3.11.6.4-1—Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic

Abutment Height (ft)	h_{eq} (ft)
5.0	4.0
10.0	3.0
≥20.0	2.0

Note that linear interpolation shall be used for intermediate wall heights.

H = 21.54 ft
h_{eq} = 2.00 ft
γ_r = 125.00 pcf

$$q_t = h_{eq}\gamma_b$$

q_T = 250.00 psf = 2.00 ft x 125.00 pcf

Calculate Road Base Surchage, q_{rb}

γ_{rb} = 115.00 pcf

$$q_{rb} = H_{rb}\gamma_{rb}$$

q_{rb} = 808.71 psf = 7.03 ft x 115.00 pcf

Calculate Weight of GRS Abutment

$$W = BH\gamma_r$$

H = 19.04 ft
B = 10.5 ft
γ_r = 115.00 pcf

W = 22990.80 plf = 19.04 ft x 10.5 ft x 115.00 pcf

Calculate Weight of RSF

$$W_{RSF} = B_{RSF} D_{RSF} \gamma_r$$

B = 14 ft
D = 2.5 ft
γ_r = 125.00 pcf

W_{RSF} = 4025.00 plf = 14 ft x 2.5 ft x 115.00 pcf

Calculate Weight of Facing

N_{block} = 28.56 = 19.04 ft/ 0.67 ft
N_{block} = 29

$$W_{face} = N_{block} \frac{W_{block}}{L_{block}}$$

W_{face} = 1643.33 plf = 29 x (85 lbs/ (18.00 in/ 12)

Summary of Applied Loads

q_{DC} = 1908.50 psf
q_{DW} = 208.23 psf
q_{LL} = 1294.82 psf
q_t = 250.00 psf
q_{rb} = 808.71 psf
W = 22990.80 plf
W_{RSF} = 4025.00 plf
W_{face} = 1643.33 plf

Check Beam Seat Pressure

Per (1) 4.3.5.4, the service bearing pressure should be targeted to around 4 ksf.

DC Reaction =	332.70 kips
DW Reaction =	36.30 kips
LL Reaction =	225.72 kips
Total =	594.72 kips

b = 4.75 ft
L = 36.70 ft

q_{seat} = 3.41 ksf = 594.72 kips/ (4.75 ft x 36.70 ft)

OK

Calculte Direct Sliding Effects at RSF/GRS Interface

Calculate Sliding per the (1) and (3) Strength I Load Combination

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2$$ (1) Eq. 9

$$F_{rb} = q_{rb} K_{ab} H$$ (1) Eq. 10

$$F_t = q_t K_{ab} H$$ (1) Eq. 11

$$F_R = \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t$$ (1) Eq. 12

F_b =	7252.67 lbs =	0.5 x	125.00 pcf x	0.32 x	19.04 ft ^	2
F_{rb} =	4928.85 lbs =	808.71 psf x	0.32 x	19.04 ft		
F_t =	1523.67 lbs =	250.00 psf	0.32 x	19.04 ft		

F_R = 20.94 klf = (1.5 x (7252.67 lbs + 4928.85 lbs) + (1.75 1523.67 lbs))/ 1000

$$W_{T,R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EV\,MIN}(q_{rb} b_{rb,t})$$ 4

$W_{T,R}$ = 37.38 klf = ((1.0 x 22990.80 plf) + (0.9 x 1908.50 psf x 4.75 ft) + (0.65 x 208.23 psf x 4.75 ft) +
(0.9 x 1643.33 plf) + (1.0 x 808.71 psf x 5.08 ft))/ 1000

Assume that μ = 2/3*tanϕ, per (1) C.1.6.1.1

μ = 0.67 = 0.67 x tan(45)

$$R_R = \Phi_\tau (W_{T,R}\mu)$$ (1) Eq. 13

R_R = 24.92 klf = 37.38 klf x 0.667

OK

Calculate Direct Sliding at Base of RSF

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2$$ (3) Eq. 17

$$F_{rb,RSF} = q_{rb} K_{ab}(H + D_{RSF})$$ (3) Eq. 18

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF})$$ (3) Eq. 19

$F_{b,RSF}$ = 9.28 klf = (0.5 x 125.00 pcf x 0.32 x (19.04 ft + 2.5)^ 2)/ 1000
$F_{rb,RSF}$ = 5.58 klf = 808.71 psf x 0.32 x (19.04 ft + 2.5 ft))/ 1000
$F_{t,RSF}$ = 1.72 klf = 250.00 psf x 0.32 x (19.04 ft + 2.5 ft))/ 1000

$$F_{R,RSF} = \gamma_{EH\,MAX}(F_{b,RSF} + F_{rb,RSF}) + \gamma_{LS} F_{t,RSF}$$ (1) Eq. 20

$F_{R,RSF}$ = 25.30 klf = 1.5 x (9.28 + 5.58) + (1.75 x 1.72 klf)

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\,MIN} W_{RSF}$$ (1) Eq. 22

$_{T,R,RSF}$ = 41.41 klf = 37.38 klf + (1.0 x 4.03 klf)

$$R_{R,RSF} = \Phi_{\tau}(W_{T,R,RSF}\mu_{RSF})$$

(1) Eq. 21

Note that ϕ = 1.0, per (1) 4.3.6.1.2

Per (2), and the previous calculation, the angle of internal friction beneath the RSF is equal to 32 degrees

$R_{R,\ RSF}$ = 25.87 klf = 1.0 x 41.41 klf x tan(32)

OK

Calculate External Bearing Resistance

Calculate V_R

Per (1) Eq. 26,

$$\sum V_R = \gamma_{EV\,MAX}(W) + \gamma_{EV\,MAX}(W_{RSF}) + \gamma_{DC\,MAX}(W_{face}) + \gamma_{LS}(q_t b_{rb,t}) + \gamma_{EH\,MAX}(q_{rb} b_{rb,t}) + \gamma_{DC\,MAX}(q_{DL} b) + \gamma_{LS}(q_{LL} b)$$

Note that $\gamma_{EV\ Max}$ is used in place of $\gamma_{EH\ Max}$ to factor q_{rb}*$b_{rb,t}$ since this is a summation of vertical reactions.

W =	31.04 klf =	1.35 x	22990.80 plf/	1000	
W_{RSF} =	5.43 klf =	1.35 x	4025.00 plf/	1000	
W_{FACE} =	2.05 klf =	1.25 x	1643.33 plf/	1000	
Q_t =	2.22 klf =	1.75 x	(250.00 psf x	5.08 ft)/	1000
Q_{rb} =	5.55 klf =	1.35 x	(808.71 psf x	5.08 ft)/	1000
Q_{DC} =	11.33 klf =	1.25 x	(1908.50 psf x	4.75 ft)/	1000
Q_{DW}=	1.48 klf =	1.5 x	(208.23 psf x	4.75 ft)/	1000
Q_{LL} =	10.76 klf =	1.75 x	(1294.82 psf	4.75 ft)/	1000
Total =	69.88 klf				

Calculate $M_{D,R}$

Per (1) Eq. 28

$$\sum M_{D,R} = \gamma_{EH\,MAX} F_{b,RSF}\left(\frac{H + D_{RSF}}{3}\right) + \gamma_{LS} F_{t,RSF}\left(\frac{H + D_{RSF}}{2}\right) + \gamma_{EH\,MAX} F_{rb,RSF}\left(\frac{H + D_{RSF}}{2}\right)$$

$F_{b,\ RSF}$ =	99.97 k-ft/ ft =	1.5 x	9.28 klf x	(0.33 x	(19.04 ft +	2.5 ft))
$F_{t,\ RSF}$ =	32.49 k-ft/ ft =	1.75 x	1.72 klf x	(0.50 x	(19.04 ft +	2.5 ft))
$F_{rb,\ RSF}$ =	90.08 k-ft/ ft =	1.5 x	5.58 klf x	(0.50 x	(19.04 ft +	2.5 ft))
Total =	222.54 k-ft/ ft					

Note that $M_{D,R}$ is taken about the bottom center of the width of the RSF.

Calculate $M_{R,R}$

Per (1) Eq. 29

$$\sum M_{R,R} = \left(\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b\right)\left[\left(\frac{b}{2} + a_b\right) - \left(\frac{B_{RSF}}{2} - x_{RSF} - b_{block}\right)\right] + \left(\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t}\right)\left(\frac{B_{RSF}}{2} - \frac{b_{rb}}{2}\right) + \gamma_{EV\,MAX} W\left(\frac{B_{RSF}}{2} - \frac{B}{2}\right) + \gamma_{DC\,MAX} W_{face}\left(B + \frac{b_{block}}{2} - \frac{B_{RSF}}{2}\right)$$

$_{L}+Q_{LL}$ =	-10.81 k-ft =	((1.25 x	1.91 ksf) +	(1.50 x	0.21 ksf) +	(1.75 x	1.29 ksf)) x 4.75 ft x
		((2.375 ft x +	0.67 ft) -	(7 ft -	2.5 ft -	1.00 ft))	
Q_t+Q_{rb} =	34.66 k-ft =	((1.75 x	0.25 klf x	5.08 ft) +	(1.35 x	0.809 klf x	5.08 ft)) x
		(7 ft -	2.54 ft)				
W =	54.32 k-ft =	1.35 x	22.99 klf x	(7 ft -	5.25 ft)		
W_{Face} =	-7.76 k-ft =	1.35 x	1.64 klf x	((2.5 ft +	1 ft) -	7 ft)	
Total =	70.40 k-ft/ ft						

Note that $M_{R,R}$ is taken about the bottom center of the width of the RSF.

Calculate Eccentricity, $e_{B,R}$

Per (1) Eq. 27

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R}$$

$e_{B,R}$ = 2.18 ft = (222.54 k-ft/ ft - 70.40 k-ft/ ft) / 69.88 klf

Calculate $\sigma_{v,base,R}$

Since $e_{B,R}$ = negative, set equal to zero in calcaulation of base bearing pressure.

Per (1) Eq. 25,

$$\sigma_{v,base,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}}$$

$\sigma_{v,base,R}$ = 7.24 ksf = 69.88 klf/ (14 ft - (2 x 2.18 ft)

Per bearing capacity calculation, factored bearing capacity = 12.71 ksf

OK

Calculate Internal Bearing Resistance

Per (1), Eq. 35

$$q_{n,an} = \left[0.7^{\left(\frac{S_v}{6d_{max}}\right)} \frac{T_f}{S_v}\right] K_{pr}$$

$q_{n,an}$ = nominal bearing resistance of the GRS abutment using the analytical method.
S_v = reinforcement spacing.
d_{max} = maximum grain size.
T_f = ultimate reinforcement strength.
K_{pr} = coefficient of passive earth pressure for the reinforced fill (calculated in equation 35).

K_{pr} = 5.83 = $\tan^2$(45 + (45/ 2))
$q_{n,an}$ = 27.36 ksf = [(0.7^ (0.67 ft/ (6 x 0.06 ft))) x (5.90 klf/ 0.67 in)] x 5.83
ϕ_{cap} = 0.45 (3) 4.3.7.2
$\phi_{cap}q_{n,an}$ = 12.31 ksf = 0.45 x 27.36 ksf

$$V_{applied,f} = \gamma_{DC\,MAX} q_{DL} + \gamma_{LL} q_{LL}$$

(1) Eq. 32

$V_{app,f}$ = 4.96 ksf = (1.25 x 1.91 ksf) + (1.50 x 0.21 ksf) + (1.75 x 1.29 ksf)

OK

Calculate and Check Deformations

Per (1), Eq. 37

$$q_{DL,allow\ @\varepsilon=1\%} = 0.2\left[0.7^{S_v/6d_{max}}\left(\frac{T_f}{S_v}\right)K_{pr}\right]$$

$6{*}d_{max}$ = 0.375 ft = 6 x 0.75 in x 0.083
T_f/S_v = 8.85 = 5.90 klf/ 0.67 ft
$6{*}d_{max}$ = 1.78 = 0.67 ft/ 0.375 ft

$q_{dl,\ all}$ = 5.47 ksf = 0.2 ((0.7^ 1.78 in) x 8.85 x 5.83 in)
q_{DL} = 1.91 ksf

OK

Calculate Lateral Strain

Per (1) 4.3.7.2.2, for the calculation of lateral strain, assume a vertical strain equal to 1% of the abutment height, H.

.01*H = 0.19 ft = 0.01 x 19.04 ft

$$D_L = \frac{2b_q D_v}{H}$$ (1) Eq. 38

D_L = 0.01 ft = (2 x (4.75 ft + 0.67 ft) x 0.02 ft)/ 19.04 ft
D_L = 0.11 in = 0.01 ft x 12

Per (1) 4.3.7.2.2, the total lateral strain should be limited to twice the vertical strain.

Strain = 0.38 ft = 2 x 0.19 ft

OK

Calculate Reinforcement Strength

q_{DC} = 1908.50 psf
q_{DW} = 208.23 psf
q_{LL} = 1294.82 psf
q_t = 250.00 psf
q_{rb} = 808.71 psf
W = 22990.80 psf
W_{RSF} = 4025.00 psf
W_{face} = 1643.33 psf

The evaulation of the abutment for the strength limit state is conducted according to (1) 4.3.7.3.1. and using the following equations:

Required Reinforcment:

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 40

Factored Total Lateral Pressure:

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f}$$ (3) Eq. 41

Factored Lateral Pressure due to weight of GRS:

$$\sigma_{h,W,f} = \gamma_{EH\ MAX}(\gamma_r z K_{ar})$$

Where:
$\gamma_{EH\ MAX}$ = maximum horizontal earth pressure load factor.
γ_r = unit weight of reinforced backfill.
z = depth from the top of the wall.
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

(3) Eq. 42

Factored Latreal Pressure due to equivalent bridge load:

$$\sigma_{h,bridge,f} = \frac{(\gamma_{DC\ MAX} q_{DL} + \gamma_{LL} q_{LL}) - (\gamma_{EH\ MAX} q_{rb} + \gamma_{LS} q_t)}{\pi}[\alpha_b + \sin(\alpha_b)\cos(\alpha_b) + 2\beta_b]K_{ar} \quad (43)$$

Where:
$\gamma_{DC\ MAX}$ = maximum DL load factor.
q_{DL} = superstructure DL pressure.
γ_{LL} = bridge LL surcharge load factor.
q_{LL} = bridge LL pressure.
q_{rb} = surcharge due to the structural backfill of the integrated approach (i.e., road base).
γ_{LS} = LL surcharge load factor.
q_t = roadway LL surcharge.
α_b = angle between wall face and projection of the midline of the bridge surcharge to the wall face found using equation 46 (see figure 23).
β_b = angle between the projections of the inner and outer edge lines of the bridge surcharge to the wall face found using equation 47 (see figure 23).

(3) Eq. 43

$$\sigma_{h,rb,f} = \gamma_{EH\,MAX} q_{rb} K_{ar}$$ (3) Eq. 44

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar}$$ (3) Eq. 45

$$\alpha_b = \tan^{-1}\left(\frac{b}{2z}\right) - \beta_b$$ (3) Eq. 46

$$\beta_b = \tan^{-1}\left(\frac{-b}{2z}\right)$$ (3) Eq. 47

Factored Reinforcement Strength:

$$T_{f,f} = \Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right) = 0.4T_f$$ (3) Eq. 48

Strength Limit State

z	$\sigma_{h,W,f}$	α_b	β_b	$\sigma_{h,bridge,f}$	$\sigma_{h,rb,f}$	$\sigma_{h,t,f}$	$\sigma_{h,f}$	$T_{req,f}$	$T_{f,f}$	
0.67 ft	0.020 ksf	2.59	-1.30	0.564 ksf	0.388 ksf	0.140 ksf	1.112 ksf	1.397 klf	2.36 klf	OK
1.33 ft	0.039 ksf	2.12	-1.06	0.538 ksf	0.388 ksf	0.140 ksf	1.106 ksf	1.390 klf	2.36 klf	OK
2.00 ft	0.059 ksf	1.74	-0.87	0.493 ksf	0.388 ksf	0.140 ksf	1.081 ksf	1.359 klf	2.36 klf	OK
2.67 ft	0.079 ksf	1.46	-0.73	0.443 ksf	0.388 ksf	0.140 ksf	1.050 ksf	1.320 klf	2.36 klf	OK
3.33 ft	0.099 ksf	1.24	-0.62	0.395 ksf	0.388 ksf	0.140 ksf	1.022 ksf	1.285 klf	2.36 klf	OK
4.00 ft	0.118 ksf	1.07	-0.54	0.353 ksf	0.388 ksf	0.140 ksf	1.000 ksf	1.256 klf	2.36 klf	OK
4.67 ft	0.138 ksf	0.94	-0.47	0.317 ksf	0.388 ksf	0.140 ksf	0.983 ksf	1.236 klf	2.36 klf	OK
5.33 ft	0.158 ksf	0.84	-0.42	0.286 ksf	0.388 ksf	0.140 ksf	0.972 ksf	1.222 klf	2.36 klf	OK
6.00 ft	0.178 ksf	0.75	-0.38	0.260 ksf	0.388 ksf	0.140 ksf	0.966 ksf	1.214 klf	2.36 klf	OK
6.67 ft	0.197 ksf	0.68	-0.34	0.238 ksf	0.388 ksf	0.140 ksf	0.964 ksf	1.212 klf	2.36 klf	OK
7.33 ft	0.217 ksf	0.63	-0.31	0.219 ksf	0.388 ksf	0.140 ksf	0.965 ksf	1.213 klf	2.36 klf	OK
8.00 ft	0.237 ksf	0.58	-0.29	0.203 ksf	0.388 ksf	0.140 ksf	0.968 ksf	1.217 klf	2.36 klf	OK
8.67 ft	0.257 ksf	0.53	-0.27	0.189 ksf	0.388 ksf	0.140 ksf	0.974 ksf	1.224 klf	2.36 klf	OK
9.33 ft	0.276 ksf	0.50	-0.25	0.177 ksf	0.388 ksf	0.140 ksf	0.981 ksf	1.233 klf	2.36 klf	OK
######	0.296 ksf	0.47	-0.23	0.166 ksf	0.388 ksf	0.140 ksf	0.990 ksf	1.244 klf	2.36 klf	OK
######	0.316 ksf	0.44	-0.22	0.156 ksf	0.388 ksf	0.140 ksf	1.000 ksf	1.257 klf	2.36 klf	OK
######	0.335 ksf	0.41	-0.21	0.147 ksf	0.388 ksf	0.140 ksf	1.011 ksf	1.271 klf	2.36 klf	OK
######	0.355 ksf	0.39	-0.20	0.140 ksf	0.388 ksf	0.140 ksf	1.023 ksf	1.286 klf	2.36 klf	OK
######	0.375 ksf	0.37	-0.19	0.133 ksf	0.388 ksf	0.140 ksf	1.036 ksf	1.302 klf	2.36 klf	OK
######	0.395 ksf	0.35	-0.18	0.126 ksf	0.388 ksf	0.140 ksf	1.049 ksf	1.319 klf	2.36 klf	OK
######	0.414 ksf	0.34	-0.17	0.120 ksf	0.388 ksf	0.140 ksf	1.063 ksf	1.336 klf	2.36 klf	OK
######	0.434 ksf	0.32	-0.16	0.115 ksf	0.388 ksf	0.140 ksf	1.078 ksf	1.354 klf	2.36 klf	OK
######	0.454 ksf	0.31	-0.15	0.110 ksf	0.388 ksf	0.140 ksf	1.093 ksf	1.373 klf	2.36 klf	OK
######	0.474 ksf	0.29	-0.15	0.106 ksf	0.388 ksf	0.140 ksf	1.108 ksf	1.392 klf	2.36 klf	OK
######	0.493 ksf	0.28	-0.14	0.102 ksf	0.388 ksf	0.140 ksf	1.123 ksf	1.412 klf	2.36 klf	OK
######	0.513 ksf	0.27	-0.14	0.098 ksf	0.388 ksf	0.140 ksf	1.139 ksf	1.432 klf	2.36 klf	OK
######	0.533 ksf	0.26	-0.13	0.094 ksf	0.388 ksf	0.140 ksf	1.155 ksf	1.452 klf	2.36 klf	OK
######	0.552 ksf	0.25	-0.13	0.091 ksf	0.388 ksf	0.140 ksf	1.172 ksf	1.473 klf	2.36 klf	OK
######	0.564 ksf	0.25	-0.12	0.089 ksf	0.388 ksf	0.140 ksf	1.181 ksf	1.485 klf	2.36 klf	OK

The evaulation of the abutment for the service limit state is conducted according to (1) 4.3.7.3.2. and using the following equations:

The nominal required reinforcement strength:

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 50

The lateral pressure within the GRS Abutment at a given dpeth and location:

$$\sigma_h = \sigma_{h,W} + \sigma_{h,bridge,eq} + \sigma_{h,rb} + \sigma_{h,t}$$ (3) Eq. 51

The lateral pressure due to the equivalent bridge load:

$$\sigma_{h,bridge,eq} = \frac{(q_{DL} + q_{LL}) - (q_{rb} + q_t)}{\pi}\left[\alpha_b + \sin(\alpha_b)\cos(\alpha_b + 2\beta_b)\right]K_{ar} \qquad (52)$$

Where:

q_{DL} = bridge DL pressure.
q_{LL} = bridge LL surcharge.
q_{rb} = surcharge due to the structural backfill of the integrated approach (i.e., road base).
q_t = roadway LL surcharge.

(3) Eq. 52

α_b = angle between wall face and projection of the midline of the bridge surcharge to the wall face found using equation 46 (see figure 23).
β_b = angle between the wall face and projection of the midline of the surcharge to the wall face found using equation 47 (see figure 23).
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

Lateral pressure due to the road base surcharge within GRS:

$$\sigma_{h,rb} = q_{rb} K_{ar}$$

(3) Eq. 53

Lateral pressure due to the traffic surchage within GRS:

$$\sigma_{h,t} = q_t K_{ar}$$

(3) Eq. 54

Service Limit State

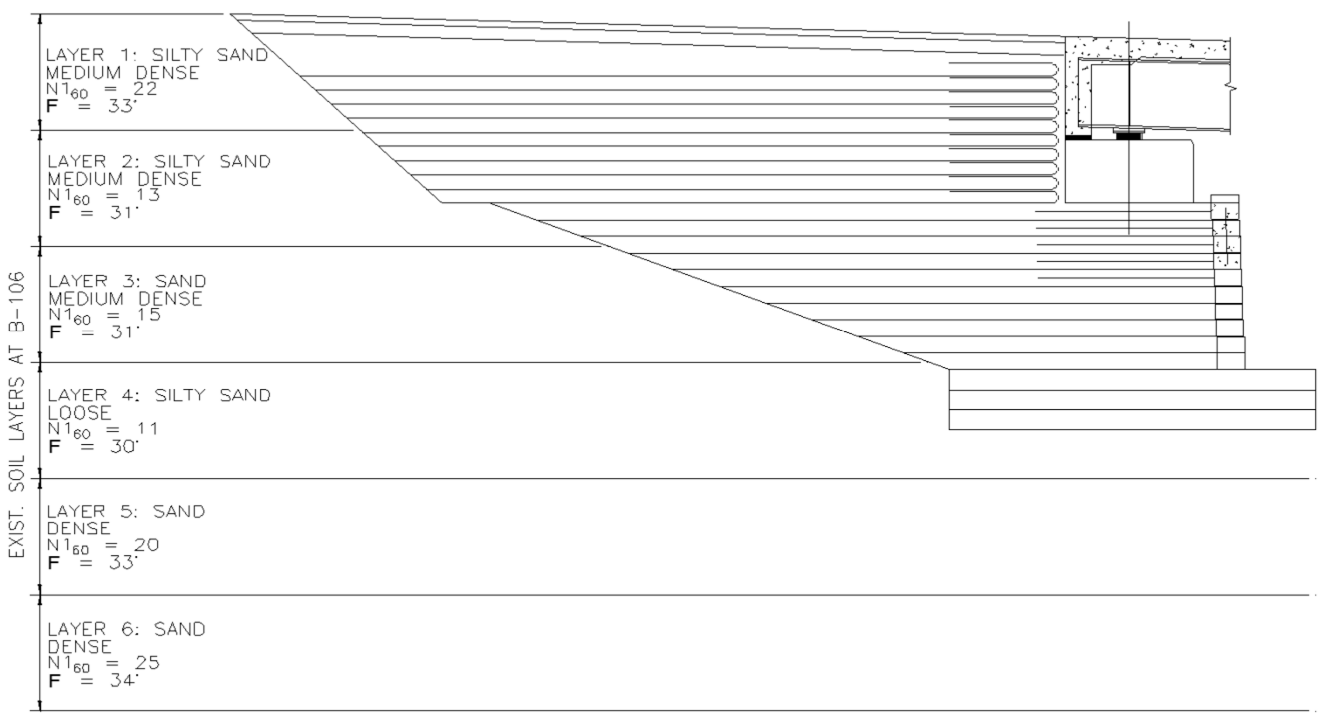
z	$\sigma_{h,W}$	α_b	β_b	$\sigma_{h,bridge,eq}$	$\sigma_{h,rb}$	$\sigma_{h,t}$	σ_h	T_{req}	$T_{@\varepsilon = 2\%}$	
0.67 ft	0.013 ksf	2.59	-1.30	0.400 ksf	0.259 ksf	0.080 ksf	0.752 ksf	0.946 klf	1.25 klf	OK
1.33 ft	0.026 ksf	2.12	-1.06	0.382 ksf	0.259 ksf	0.080 ksf	0.747 ksf	0.939 klf	1.25 klf	OK
2.00 ft	0.039 ksf	1.74	-0.87	0.350 ksf	0.259 ksf	0.080 ksf	0.729 ksf	0.916 klf	1.25 klf	OK
2.67 ft	0.053 ksf	1.46	-0.73	0.315 ksf	0.259 ksf	0.080 ksf	0.706 ksf	0.888 klf	1.25 klf	OK
3.33 ft	0.066 ksf	1.24	-0.62	0.281 ksf	0.259 ksf	0.080 ksf	0.685 ksf	0.861 klf	1.25 klf	OK
4.00 ft	0.079 ksf	1.07	-0.54	0.251 ksf	0.259 ksf	0.080 ksf	0.668 ksf	0.840 klf	1.25 klf	OK
4.67 ft	0.092 ksf	0.94	-0.47	0.225 ksf	0.259 ksf	0.080 ksf	0.656 ksf	0.824 klf	1.25 klf	OK
5.33 ft	0.105 ksf	0.84	-0.42	0.203 ksf	0.259 ksf	0.080 ksf	0.647 ksf	0.814 klf	1.25 klf	OK
6.00 ft	0.118 ksf	0.75	-0.38	0.185 ksf	0.259 ksf	0.080 ksf	0.642 ksf	0.807 klf	1.25 klf	OK
6.67 ft	0.132 ksf	0.68	-0.34	0.169 ksf	0.259 ksf	0.080 ksf	0.640 ksf	0.804 klf	1.25 klf	OK
7.33 ft	0.145 ksf	0.63	-0.31	0.156 ksf	0.259 ksf	0.080 ksf	0.639 ksf	0.804 klf	1.25 klf	OK
8.00 ft	0.158 ksf	0.58	-0.29	0.144 ksf	0.259 ksf	0.080 ksf	0.641 ksf	0.806 klf	1.25 klf	OK
8.67 ft	0.171 ksf	0.53	-0.27	0.134 ksf	0.259 ksf	0.080 ksf	0.644 ksf	0.810 klf	1.25 klf	OK
9.33 ft	0.184 ksf	0.50	-0.25	0.125 ksf	0.259 ksf	0.080 ksf	0.649 ksf	0.815 klf	1.25 klf	OK
######	0.197 ksf	0.47	-0.23	0.118 ksf	0.259 ksf	0.080 ksf	0.654 ksf	0.822 klf	1.25 klf	OK
######	0.210 ksf	0.44	-0.22	0.111 ksf	0.259 ksf	0.080 ksf	0.660 ksf	0.830 klf	1.25 klf	OK
######	0.224 ksf	0.41	-0.21	0.105 ksf	0.259 ksf	0.080 ksf	0.667 ksf	0.839 klf	1.25 klf	OK
######	0.237 ksf	0.39	-0.20	0.099 ksf	0.259 ksf	0.080 ksf	0.675 ksf	0.848 klf	1.25 klf	OK
######	0.250 ksf	0.37	-0.19	0.094 ksf	0.259 ksf	0.080 ksf	0.683 ksf	0.858 klf	1.25 klf	OK
######	0.263 ksf	0.35	-0.18	0.090 ksf	0.259 ksf	0.080 ksf	0.692 ksf	0.869 klf	1.25 klf	OK
######	0.276 ksf	0.34	-0.17	0.000 ksf	0.259 ksf	0.080 ksf	0.615 ksf	0.773 klf	1.25 klf	OK
######	0.289 ksf	0.32	-0.16	0.000 ksf	0.259 ksf	0.080 ksf	0.628 ksf	0.790 klf	1.25 klf	OK
######	0.303 ksf	0.31	-0.15	0.000 ksf	0.259 ksf	0.080 ksf	0.641 ksf	0.806 klf	1.25 klf	OK
######	0.316 ksf	0.29	-0.15	0.000 ksf	0.259 ksf	0.080 ksf	0.655 ksf	0.823 klf	1.25 klf	OK
######	0.329 ksf	0.28	-0.14	0.000 ksf	0.259 ksf	0.080 ksf	0.668 ksf	0.839 klf	1.25 klf	OK
######	0.342 ksf	0.27	-0.14	0.000 ksf	0.259 ksf	0.080 ksf	0.681 ksf	0.856 klf	1.25 klf	OK
######	0.355 ksf	0.26	-0.13	0.000 ksf	0.259 ksf	0.080 ksf	0.694 ksf	0.872 klf	1.25 klf	OK
######	0.368 ksf	0.25	-0.13	0.000 ksf	0.259 ksf	0.080 ksf	0.707 ksf	0.889 klf	1.25 klf	OK
######	0.376 ksf	0.25	-0.12	0.063 ksf	0.259 ksf	0.080 ksf	0.778 ksf	0.978 klf	1.25 klf	OK

Final Design - Abutment 4 - Settlement VTRANS D37 IM 091-1(68)

References:
(1) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018

The proposed Span 2 - East Abutment is assumed to be founded on the existing granular soil defined in boring B-106. A sketch of the existing soil layers in relation to the proposed abutment is shown below.

SPAN 2 – EAST ABUTMENT



Note that the proposed bottom of footing elevation for the Span 1 - West Abutment is 442.46 ft. Therefore, the footing will be founded on layers of dense silty sand.

Abutment Geometry

Footing Width, B =	14.00 ft			See "Abutment Design"
Abutment Length, L =	36.70 ft			See "Abutment Design"
e_B =	2.18 ft			See "Abutment Design"
B' =	9.65 ft =	14.00 ft -	(2.00 x	2.18 ft)

Final Design - Abutment 4 - Settlement VTRANS D37 IM 091-1(68)

Settlement Analyses

Settlment Analyses per (1) 10.6.2.4

Total settlement, including elastic, consolidation, and secondary components may be taken as:

$S_t = S_e + S_c + S_s$ (Ref 1 - Eq. 10.6.2.4.1-1)

For cohesionless soils, only elastic settlement is typically considered, therefore $S_t = S_e$

Elastic settlement can be approximated using either the elastic half-space method or the empirical Hough method. Both methods are calculated here for reference.

Elastic Half-Space Method:

$$S_e = \frac{[q_o(1-v^2)\sqrt{A'}]}{144 E_s \beta_z}$$ (Ref 1 - Eq. 10.6.2.4.2-1)

Where:

q_o = applied vertical stress (ksf)

A' = effective area of footing (ft^2)

E_s = Young's modulus of soil taken from Article 10.4.6.3

B_z = shape factor taken as specified in table 10.6.2.4.2-1

v = Poisson's Ratio, taken as specified in Article 10.4.6.3

P_{SS} =	369 kips	(total vertical load on abutment, Service 1. See "Abutment Design")				
P_W =	844 kips =	22991 plf x	36.70 ft	0.001		
P_{RSF} =	148 kips =	4025 plf x	36.70 ft	0.001		
P_{FACE} =	60 kips =	1643.3 plf x	36.70 ft	0.001		
P_{LL} =	226 kips					
P_{LS} =	47 kips =	250 plf x	5.08 ft x	36.70 ft x	0.001	
P =	1693 kips					

A' = B'L' =	354 ft² =	9.65 ft x	36.70 ft
q_o = P/A' =	4.8 ksf =	1693 kips/	354 ft²

E_s =	4.17 ksi	(1) Table C10.4.6.3-1, Loose/Medium Dense Sand
v =	0.28	(1) Table C10.4.6.3-1, Loose/Medium Dense Sand

Final Design - Abutment 4 - Settlement VTRANS D37 IM 091-1(68)

L/B = 3.80 = 36.70 ft/ 9.65 ft
Footing Type = Flexible
βz = 1.166

$$Se = 0.12 \text{ ft} = \frac{4.8 \text{ ksf} \quad (1 - \quad 0.08) \times \quad 18.81 \text{ ft}}{(144 \times \quad 4.17 \times \quad 1.166)}$$

Se = 1.42 in

While the elastic half space method should provide reasonably accurate results, calculate total settlement using the Hough method for comparison.
Empirical Hough Method:

$$S_e = \sum_{i=1}^{n} \Delta H_i \qquad \text{(Ref 1 - Eq. 10.6.2.4.2-2)}$$

Where: $$\Delta H_i = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right) \qquad \text{(Ref 1 - Eq. 10.6.2.4.2-3)}$$

n = number of soil layers within zone of stress influence of the footing
H_c = initial height of each layer I (ft)
C' = bearing capacity index from Figure 10.6.2.4.2-1
σ'_o = initial vertical effective stress at the midpoint of layer I (ksf)
$\Delta\sigma_v$ = increase in vertical stress at the midpoint of layer I (ksf)

Depth of bottom of footing below grade, d = 14.54 ft See Soil Properties - B106
Water Table Depth = 26 ft
Soil Unit Weight = 0.125 kcf
Water Unit Weight = 0.062 kcf

Increase in vertical stress at point directly below footing is equal to the applied vertical stress, qo (ksf). Increase in vertical stress at a depth of z below the bottom of footing are assumed equal to applied load P divided by effective area at point of interest calculated assuming a 2:1 distribution slope, therefore A' = (B'+z)(L'+z)

Layer	Depth to Bottom of Layer	H_c	Midpoint Depth Below Footing, z	Sample Midpoint	h_1	h_2
1	2.00 ft	2.00 ft	0.00 ft	1.00	1.00 ft	0.00 ft
2	8.00 ft	6.00 ft	0.00 ft	5.00 ft	5.00 ft	0.00 ft
3	12.00 ft	4.00 ft	0.00 ft	10.00 ft	10.00 ft	0.00 ft
4	16.00 ft	4.00 ft	0.73 ft	14.00 ft	14.00 ft	0.00 ft
5	21.00 ft	5.00 ft	3.96 ft	18.50 ft	18.50 ft	0.00 ft
6	30.50 ft	9.50 ft	11.21 ft	25.75 ft	25.75 ft	0.00 ft
7	40.50 ft	10.00 ft	20.96 ft	35.50 ft	26.00 ft	9.50 ft

Final Design - Abutment 4 - Settlement VTRANS D37 IM 091-1(68)

Layer	Depth to Bottom of Layer	σ'_v	A'	$\Delta\sigma_v$ (P/A')	N1 $_{60}$	*C'	ΔH
1	2.00 ft	0.125 ksf	354 ft²	4.8 ksf	21		-
2	8.00 ft	0.625 ksf	354 ft²	4.8 ksf	13		-
3	12.00 ft	1.250 ksf	354 ft²	4.8 ksf	15		-
4	16.00 ft	1.750 ksf	388 ft²	4.4 ksf	11	60	0.04 ft
5	21.00 ft	2.313 ksf	553 ft²	3.1 ksf	20	75	0.02 ft
6	30.50 ft	3.219 ksf	999 ft²	1.7 ksf	25	80	0.02 ft
7	40.50 ft	5.032 ksf	1765 ft²	1.0 ksf	36	110	0.01 ft
						Total =	0.09 ft
						Total =	1.07 in

*In Figure 10.6.2.4.2-1 Well graded silty sand and gravel

References:

1) GRS-IBS Interim Implementation Guide, FHWA, June 2018
(2) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018
(3) Geotechnical Data Report, Teracon Consultants Inc., March 27. 2019

Material Properties

γ_f = 115.00 pcf (Reinforced Fill)
ϕ (Reinforced Soil)= 45 degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β = 26.57 degrees

K_a (Reinforced Soil) = 0.22 = 0.894 x (0.894 - (0.800 - 0.5)^ 0.5) / (0.894 + (0.800 - 0.5)^ 0.5)

Fill retained is in-situ soil.

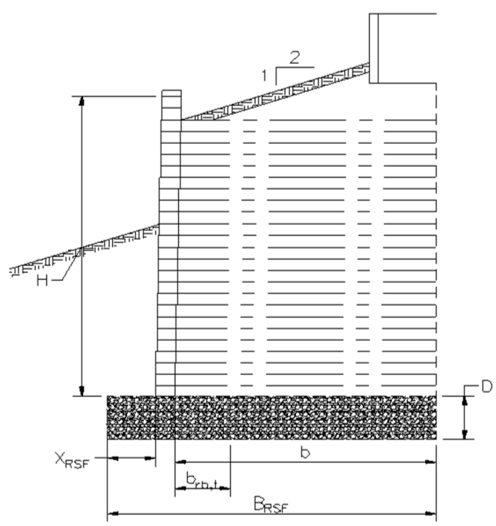
γ_b = 125.00 pcf (Retained Fill)
ϕ (Existing Filll)= 34.00 degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β = 26.57 degrees

K_a (Retained Fill) = 0.41 = 0.894 x (0.894 - (0.800 - 0.6873)^ 0.5) / (0.894 + (0.800 - 0.6873)^ 0.5)

Geometry



Depth of RSF, D = 2.5 ft
Top of RSF EL. = 458.96
Bottom of RSF EL. = 456.46 = 458.96 - 2.5 ft

Top of Wall EL. = 476.26
Wall Height = 17.30 ft 476.26 - 458.96
H = 17.30 ft

B = 9 ft
Toe Length, X_{RSF} = 2.5 ft
Total Width, B_{RSF} = 12.5 ft
$b_{rb,t}$ = 0.00 ft
Stepped Footing Length = 13.75 ft

h_{block} = 8.00 in
D_{block} = 12.00 in
L_{block} = 18.00 in
Weight = 85 lbs per block

Reinforcement

S_v = 0.67 ft
d_{max} = 0.75 in
T_f = 5.90 klf

Loading
Note that the superstructure dead loads, roadway bed loads, and traffic live loads are not applicable for this structure. No live load surcharge is applied.

Load Factors
Per (3) Table 3.4.1-1 and Table 3.4.1-2.

γ_p (DC, Min.) =	0.90	3) Table 3.4.1-2
γ_p (DC, Max.) =	1.25	3) Table 3.4.1-2
γ_p (EH, Max) =	1.50	3) Table 3.4.1-2
γ_p (EV, Min.) =	1.00	3) Table 3.4.1-2
γ_p (EV, Max.) =	1.35	3) Table 3.4.1-2
Factor (LS) =	1.75	3) Table 3.4.1-1

Calculate Vertical Loads and Applied Pressures
Calculate Traffic Surcharge, q_T

Due to wingwall layout geometry, assume that there is no live load surcharge acting on the wingwall.

h_{eq} = 0.00 ft
γ_r= 125.00 pcf

$$q_t = h_{eq}\gamma_b$$

q_T = 0.00 psf = 0.00 ft x 125.00 pcf

Calculate Weight of GRS Abutment

$$W = BH\gamma_r$$

H = 17.30 ft
B = 9 ft
γ_r= 115.00 pcf

W = 17907.39 plf = 17.30183 ft x 9 ft x 115.00 pcf

Calculate Weight of RSF

$$W_{RSF} = B_{RSF}D_{RSF}\gamma_r$$

B = 12.5 ft
D = 2.5 ft
γ_r= 125.00 pcf

W_{RSF} = 3593.75 plf = 12.5 ft x 2.5 ft x 115.00 pcf

Calculate Weight of Facing

N_{block} = 25.95 = 17.301827 ft/ 0.67 ft
N_{block} = 26

$$W_{face} = N_{block}\frac{W_{block}}{L_{block}}$$

W_{face} = 1473.33 plf = 26 x (85 lbs/ (18.00 in/ 12)

Calculate Weight of Surplus Retained Fill

Fill is retained at a 2H:1V slope

B = 9 ft
H_{max} = 4.50 ft = 9 ft/ 2
W_{SF} = 2531.25 plf = 0.5 x 9 ft x 4.50 ft x 125.00 pcf

FINAL DESIGN - GRS-IBS WINGWALL - ABUTMENT 1 - NWWW - STEP 1 VTRANS D37 IM 091-1(68)

Summary of Applied Loads

q_t =	0.00 psf
W =	17907.39 plf
W_{RSF} =	3593.75 plf
W_{face} =	1473.33 plf
W_{SF} =	2531.25 plf

Calculte Direct Sliding Effects at RSF/GRS Interface

Calculate Sliding per the (1) and (3) Strength I Load Combination

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2$$ (1) Eq. 9

$$F_t = q_t K_{ab} H$$ (1) Eq. 11

$$F_R = \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t$$ (1) Eq. 12

F_b = 7600.72 lbs = 0.5 x 125.00 pcf x 0.41 x 17.30183 ft^ 2

F_t = 0.00 lbs = 0.00 psf 0.41 x 17.301827 ft

F_R = 11.40 klf = (1.5 x 7600.72 lbs) + (1.75 x 0.00 lbs))/ 1000

$$W_{T,R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EV\,MIN}(q_{rb} b_{rb,t})$$ + $\gamma_{EV\,MIN} W_{SF}$ (1) Eq. 14

$W_{T,R}$ = 21.76 klf = ((1.0 x 17907.39 plf) + ((0.9 x 1473.33 plf))/ 1000) + (1.00 x 2.53 klf)

Assume that μ = 2/3*tanϕ, per (1) C.1.6.1.1

μ = 0.67 = 0.67 x tan(45)

$$R_R = \Phi_\tau (W_{T,R}\mu)$$ (1) Eq. 13

R_R = 14.51 klf = 21.765 klf x 0.667

OK

Calculate Direct Sliding at Base of RSF

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2$$ (3) Eq. 17

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF})$$ (3) Eq. 19

$F_{b,\,RSF}$ = 9.96 klf = (0.5 x 125.00 pcf x 0.41 x (17.30 ft + 2.5)^ 2)/ 1000

$F_{t,\,RSF}$ = 0.00 klf = 0.00 psf x 0.41 x (17.30 ft + 2.5 ft))/ 1000

$$F_{R,RSF} = \gamma_{EH\,MAX}(F_{b,RSF} + F_{rb,RSF}) + \gamma_{LS} F_{t,RSF}$$ (1) Eq. 20

$F_{R,RSF}$ = 14.93 klf = (1.5 x 9.96 klf) + (1.75 x 0.00 klf)

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\,MIN} W_{RSF}$$ (1) Eq. 22

$W_{T,R,RSF}$ = 25.36 klf = 21.76 klf + (1.0 x 3.59 klf)

$$R_{R,RSF} = \Phi_\tau (W_{T,R,RSF}\mu_{RSF})$$ (1) Eq. 21

Note that ϕ = 1.0, per (1) 4.3.6.1.2
Per (2), and the previous calculation, the angle of internal friction beneath the RSF is equal to 31 degrees

$R_{R,\,RSF}$ = 15.24 klf = 1.0 x 25.36 klf x tan(31)

OK

Calculate External Bearing Resistance

Calculate V_R

Per (1) Eq. 26,

$$\sum V_R = \gamma_{EV\,MAX}(W) + \gamma_{EV\,MAX}(W_{RSF}) + \gamma_{DC\,MAX}(W_{face}) + \gamma_{LS}(q_t b_{rb,t}) + \gamma_{EH\,MAX}(q_{rb} b_{rb,t}) + \gamma_{DC\,MAX}(q_{DL} b) + \gamma_{LS}(q_{LL} b)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\,MIN}$

W =	24.17 klf =	1.35 x	17907.39 plf/	1000		
W_{RSF} =	4.85 klf =	1.35 x	3593.75 plf/	1000		
W_{FACE} =	1.84 klf =	1.25 x	1473.33 plf/	1000		
W_{SF} =	3.42 klf =	1.35 x	2531.25 plf/	1000		
Q_t =	0.00 klf =	1.75 x	(0.00 psf x	0.00 ft)/	1000	
Total =	34.29 klf					

Calculate $M_{D,R}$

Per (1) Eq. 28

$$\sum M_{D,R} = \gamma_{EH\,MAX} F_{b,RSF}\left(\frac{H + D_{RSF}}{3}\right) + \gamma_{LS} F_{t,RSF}\left(\frac{H + D_{RSF}}{2}\right) + \gamma_{EH\,MAX} F_{rb,RSF}\left(\frac{H + D_{RSF}}{2}\right)$$

$F_{b,\,RSF}$ =	98.57 k-ft/ ft =	1.5 x	9.96 klf x	(0.33 x	(17.3018 ft +	2.5 ft))
$F_{t,\,RSF}$ =	0.00 k-ft/ ft =	1.75 x	0.00 klf x	(0.50 x	(17.3018 ft +	2.5 ft))
Total =	98.57 k-ft/ ft					

Note that $M_{D,R}$ is taken about the bottom center of the width of the RSF.

Calculate $M_{R,R}$

Per (1) Eq. 29

$$\sum M_{R,R} = (\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b)\left[\left(\frac{b}{2} + a_b\right) - \left(\frac{B_{RSF}}{2} - x_{RSF} - b_{block}\right)\right] + (\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t})\left(\frac{B_{RSF}}{2} - \frac{b_{rb}}{2}\right) + \gamma_{EV\,MAX} W\left(\frac{B_{RSF}}{2} - \frac{B}{2}\right) + \gamma_{DC\,MAX} W_{face}\left(B + \frac{b_{block}}{2} - \frac{B_{RSF}}{2}\right)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\,MAX}$. Resisting component = W_{SF}*((X_{RSF}+D_{block}+(2/3)*B)-BRSF/2)

W =	42.31 k-ft =	1.35 x	17.91 klf x	(6.25 ft -	4.5 ft)			
W_{Face} =	-6.46 k-ft =	1.35 x	1.47 klf x	((2.5 ft +	0.5 ft) -	6.25 ft)		
W_{SF} =	11.11 k-ft =	1.35 x	2.53 klf x	((2.5 ft +	1 ft +	(0.67 x	9 ft)) -	(12.5 ft/ 2))
Total =	46.95 k-ft/ ft							

Note that $M_{R,R}$ is taken about the bottom center of the width of the RSF.

Calculate Eccentricity, $e_{B,R}$

Per (1) Eq. 27

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R}$$

$e_{B,R}$ = 1.51 ft = $\dfrac{98.57 \text{ k-ft/ ft} - 46.95 \text{ k-ft/ ft}}{34.29 \text{ klf}}$

Calculate $\sigma_{v,base,R}$

Since $e_{B,R}$ = negative, set equal to zero in calcaulation of base bearing pressure.

Per (1) Eq. 25,

$$\sigma_{v,base,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}}$$

$\sigma_{v,base,R}$ = 3.61 ksf = 34.29 klf/ (12.5 ft - (2 x 1.51 ft)

Per abutment bearing capacity calculation, allowable bearing capacity = 9.95 ksf

OK

Calculate Reinforcement Strength

q_t = 0.00 psf

W = 17907.39 psf

W_{RSF} = 3593.75 psf

W_{face} = 1473.33 psf

The evaulation of the abutment for the strength limit state is conducted according to (1) 4.3.7.3.1. and using the following equations:

Required Reinforcment:

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 40

Factored Total Lateral Pressure:

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f}$$ (3) Eq. 41

Where:

$$\sigma_{h,W,f} = \gamma_{EH\ MAX}(\gamma_r z K_{ar})$$ (3) Eq. 42

Where:

$\gamma_{EH\ MAX}$ = maximum horizontal earth pressure load factor.
γ_r = unit weight of reinforced backfill.
z = depth from the top of the wall.
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar}$$ (3) Eq. 45

Note that $\sigma_{hbridge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Factored Reinforcement Strength:

$$T_{f,f} = \Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right) = 0.4T_f$$ (3) Eq. 48

Strength Limit State

z	$\sigma_{h,W,f}$	$\sigma_{h,t,f}$	$\sigma_{h,f}$	$T_{req,f}$	$T_{f,f}$	
0.67 ft	0.02 ksf	0.00 ksf	0.02 ksf	0.03 ksf	2.36 klf	OK
1.33 ft	0.05 ksf	0.00 ksf	0.05 ksf	0.06 ksf	2.36 klf	OK
2.00 ft	0.07 ksf	0.00 ksf	0.07 ksf	0.09 ksf	2.36 klf	OK
2.67 ft	0.10 ksf	0.00 ksf	0.10 ksf	0.12 ksf	2.36 klf	OK
3.33 ft	0.12 ksf	0.00 ksf	0.12 ksf	0.16 ksf	2.36 klf	OK
4.00 ft	0.15 ksf	0.00 ksf	0.15 ksf	0.19 ksf	2.36 klf	OK
4.67 ft	0.17 ksf	0.00 ksf	0.17 ksf	0.22 ksf	2.36 klf	OK
5.33 ft	0.20 ksf	0.00 ksf	0.20 ksf	0.25 ksf	2.36 klf	OK
6.00 ft	0.22 ksf	0.00 ksf	0.22 ksf	0.28 ksf	2.36 klf	OK
6.67 ft	0.25 ksf	0.00 ksf	0.25 ksf	0.31 ksf	2.36 klf	OK
7.33 ft	0.27 ksf	0.00 ksf	0.27 ksf	0.34 ksf	2.36 klf	OK
8.00 ft	0.30 ksf	0.00 ksf	0.30 ksf	0.37 ksf	2.36 klf	OK
8.67 ft	0.32 ksf	0.00 ksf	0.32 ksf	0.40 ksf	2.36 klf	OK
9.33 ft	0.35 ksf	0.00 ksf	0.35 ksf	0.44 ksf	2.36 klf	OK
10.00 ft	0.37 ksf	0.00 ksf	0.37 ksf	0.47 ksf	2.36 klf	OK
10.67 ft	0.40 ksf	0.00 ksf	0.40 ksf	0.50 ksf	2.36 klf	OK
11.33 ft	0.42 ksf	0.00 ksf	0.42 ksf	0.53 ksf	2.36 klf	OK
12.00 ft	0.45 ksf	0.00 ksf	0.45 ksf	0.56 ksf	2.36 klf	OK
12.67 ft	0.47 ksf	0.00 ksf	0.47 ksf	0.59 ksf	2.36 klf	OK
13.33 ft	0.49 ksf	0.00 ksf	0.49 ksf	0.62 ksf	2.36 klf	OK
14.00 ft	0.52 ksf	0.00 ksf	0.52 ksf	0.65 ksf	2.36 klf	OK
14.67 ft	0.54 ksf	0.00 ksf	0.54 ksf	0.68 ksf	2.36 klf	OK
15.33 ft	0.57 ksf	0.00 ksf	0.57 ksf	0.71 ksf	2.36 klf	OK
16.00 ft	0.59 ksf	0.00 ksf	0.59 ksf	0.75 ksf	2.36 klf	OK
16.67 ft	0.62 ksf	0.00 ksf	0.62 ksf	0.78 ksf	2.36 klf	OK
17.30 ft	0.64 ksf	0.00 ksf	0.64 ksf	0.81 ksf	2.36 klf	OK

The evaulation of the abutment for the service limit state is conducted according to (1) 4.3.7.3.2. and using the following equations:

The nominal required reinforcement strength:

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$

(3) Eq. 50

The lateral pressure within the GRS Abutment at a given dpeth and location:

$$\sigma_h = \sigma_{h,W} + \sigma_{h,bridge,eq} + \sigma_{h,rb} + \sigma_{h,t}$$

(3) Eq. 51

Where:

$$\sigma_{h,W} = \gamma_r z K_{ar}$$

(3) Eq. 2

$$\sigma_{h,t} = q_t K_{ar}$$

(3) Eq. 54

Note that $\sigma_{hbrdge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

FINAL DESIGN - GRS-IBS WINGWALL - ABUTMENT 1 - NWWW - STEP 1 VTRANS D37 IM 091-1(68)

Service Limit State

z	$\sigma_{h,W}$	$\sigma_{h,t}$	σ_h	T_{req}	$T_{@\varepsilon = 2\%}$	
0.67 ft	0.02 ksf	0.00 ksf	0.016 ksf	0.02 klf	1.25 klf	OK
1.33 ft	0.03 ksf	0.00 ksf	0.033 ksf	0.04 klf	1.25 klf	OK
2.00 ft	0.05 ksf	0.00 ksf	0.049 ksf	0.06 klf	1.25 klf	OK
2.67 ft	0.07 ksf	0.00 ksf	0.066 ksf	0.08 klf	1.25 klf	OK
3.33 ft	0.08 ksf	0.00 ksf	0.082 ksf	0.10 klf	1.25 klf	OK
4.00 ft	0.10 ksf	0.00 ksf	0.099 ksf	0.12 klf	1.25 klf	OK
4.67 ft	0.12 ksf	0.00 ksf	0.115 ksf	0.15 klf	1.25 klf	OK
5.33 ft	0.13 ksf	0.00 ksf	0.132 ksf	0.17 klf	1.25 klf	OK
6.00 ft	0.15 ksf	0.00 ksf	0.148 ksf	0.19 klf	1.25 klf	OK
6.67 ft	0.16 ksf	0.00 ksf	0.165 ksf	0.21 klf	1.25 klf	OK
7.33 ft	0.18 ksf	0.00 ksf	0.181 ksf	0.23 klf	1.25 klf	OK
8.00 ft	0.20 ksf	0.00 ksf	0.198 ksf	0.25 klf	1.25 klf	OK
8.67 ft	0.21 ksf	0.00 ksf	0.214 ksf	0.27 klf	1.25 klf	OK
9.33 ft	0.23 ksf	0.00 ksf	0.231 ksf	0.29 klf	1.25 klf	OK
10.00 ft	0.25 ksf	0.00 ksf	0.247 ksf	0.31 klf	1.25 klf	OK
10.67 ft	0.26 ksf	0.00 ksf	0.264 ksf	0.33 klf	1.25 klf	OK
11.33 ft	0.28 ksf	0.00 ksf	0.280 ksf	0.35 klf	1.25 klf	OK
12.00 ft	0.30 ksf	0.00 ksf	0.297 ksf	0.37 klf	1.25 klf	OK
12.67 ft	0.31 ksf	0.00 ksf	0.313 ksf	0.39 klf	1.25 klf	OK
13.33 ft	0.33 ksf	0.00 ksf	0.330 ksf	0.41 klf	1.25 klf	OK
14.00 ft	0.35 ksf	0.00 ksf	0.346 ksf	0.44 klf	1.25 klf	OK
14.67 ft	0.36 ksf	0.00 ksf	0.363 ksf	0.46 klf	1.25 klf	OK
15.33 ft	0.38 ksf	0.00 ksf	0.379 ksf	0.48 klf	1.25 klf	OK
16.00 ft	0.40 ksf	0.00 ksf	0.396 ksf	0.50 klf	1.25 klf	OK
16.67 ft	0.41 ksf	0.00 ksf	0.412 ksf	0.52 klf	1.25 klf	OK
17.30 ft	0.43 ksf	0.00 ksf	0.428 ksf	0.54 klf	1.25 klf	OK

FINAL DESIGN - GRS-IBS WINGWALL - ABUTMENT 1 - NWWW - STEP 2 VTRANS D37 IM 091-1(68)

References:

1) GRS-IBS Interim Implementation Guide, FHWA, June 2018
(2) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018
(3) Geotechnical Data Report, Teracon Consultants Inc., March 27. 2019

Material Properties

γ_f =	115.00 pcf	(Reinforced Fill)
ϕ (Reinforced Soil)=	45	degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β =	26.57	degrees

K_a (Reinforced Soil) = 0.22 = 0.894 x $\frac{0.894 - (0.800 - 0.5)\text{^ } 0.5}{0.894 + (0.800 - 0.5)\text{^ } 0.5}$

Fill retained is in-situ soil.

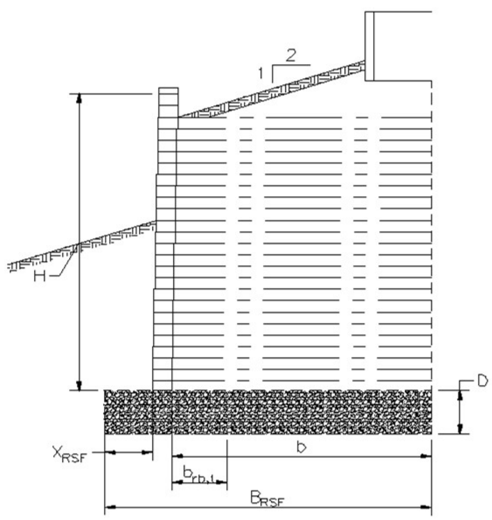
γ_b =	125.00 pcf	(Retained Fill)
ϕ (Existing Filll)=	34.00	degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β =	26.57	degrees

K_a (Retained Fill) = 0.41 = 0.894 x $\frac{0.894 - (0.800 - 0.6873)\text{^ } 0.5}{0.894 + (0.800 - 0.6873)\text{^ } 0.5}$

Geometry



Depth of RSF, D =	2.5 ft		
Top of RSF EL. =	456.46		
Bottom of RSF EL. =	453.96 =	456.46 -	2.5 ft
Top of Wall EL. =	479.78		
Wall Height =	23.32 ft	479.78 -	456.46
H =	23.32 ft		
B =	12 ft		
Toe Length, X_{RSF} =	2.5 ft		
Total Width, B_{RSF} =	15.5 ft		
$b_{rb,t}$ =	0.00 ft		
Stepped Footing Length =	13.75 ft		
h_{block} =	8.00 in		
D_{block} =	12.00 in		
L_{block} =	18.00 in		
Weight =	85 lbs	per block	

Reinforcement

S_v = 0.67 ft
d_{max} = 0.75 in
T_f = 5.90 klf

Loading
Note that the superstructure dead loads, roadway bed loads, and traffic live loads are not applicable for this structure. No live load surcharge is applied.

Load Factors
Per (3) Table 3.4.1-1 and Table 3.4.1-2.

γ_p (DC, Min.) =	0.90	3) Table 3.4.1-2
γ_p (DC, Max.) =	1.25	3) Table 3.4.1-2
γ_p (EH, Max) =	1.50	3) Table 3.4.1-2
γ_p (EV, Min.) =	1.00	3) Table 3.4.1-2
γ_p (EV, Max.) =	1.35	3) Table 3.4.1-2
Factor (LS) =	1.75	3) Table 3.4.1-1

Calculate Vertical Loads and Applied Pressures
Calculate Traffic Surcharge, q_T

Due to wingwall layout geometry, assume that there is no live load surcharge acting on the wingwall.

h_{eq} = 0.00 ft
γ_r = 125.00 pcf

$$q_t = h_{eq}\gamma_b$$

q_T = 0.00 psf = 0.00 ft x 125.00 pcf

Calculate Weight of GRS Abutment

$$W = BH\gamma_r$$

H = 23.32 ft
B = 12 ft
γ_r = 115.00 pcf

W = 32181.91 plf = 23.32022 ft x 12 ft x 115.00 pcf

Calculate Weight of RSF

$$W_{RSF} = B_{RSF}D_{RSF}\gamma_r$$

B = 15.5 ft
D = 2.5 ft
γ_r = 125.00 pcf

W_{RSF} = 4456.25 plf = 15.5 ft x 2.5 ft x 115.00 pcf

Calculate Weight of Facing

N_{block} = 34.98 = 23.320223 ft/ 0.67 ft
N_{block} = 35

$$W_{face} = N_{block}\frac{W_{block}}{L_{block}}$$

W_{face} = 1983.33 plf = 35 x (85 lbs/ (18.00 in/ 12)

Calculate Weight of Surplus Retained Fill

Fill is retained at a 2H:1V slope

B = 12 ft
H_{max} = 6.00 ft = 12 ft/ 2
W_{SF} = 4500.00 plf = 0.5 x 12 ft x 6.00 ft x 125.00 pcf

Summary of Applied Loads

q_t = 0.00 psf
W = 32181.91 plf
W_{RSF} = 4456.25 plf
W_{face} = 1983.33 plf
W_{SF} = 4500.00 plf

Calculte Direct Sliding Effects at RSF/GRS Interface

Calculate Sliding per the (1) and (3) Strength I Load Combination

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2$$ (1) Eq. 9

$$F_t = q_t K_{ab} H$$ (1) Eq. 11

$$F_R = \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t$$ (1) Eq. 12

F_b = 13808.18 lbs = 0.5 x 125.00 pcf x 0.41 x 23.32022 ft^ 2
F_t = 0.00 lbs = 0.00 psf 0.41 x 23.3202234 ft

F_R = 20.71 klf = (1.5 x 13808.18 lbs) + (1.75 x 0.00 lbs))/ 1000

$$W_{T,R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EV\,MIN}(q_{rb} b_{rb,t}) + \gamma_{EV\,MIN} W_{SF}$$ (1) Eq. 14

$W_{T,R}$ = 38.47 klf = ((1.0 x 32181.91 plf) + ((0.9 x 1983.33 plf))/ 1000) + (1.00 x 4.50 klf)

Assume that μ = 2/3*tanϕ, per (1) C.1.6.1.1

μ = 0.67 = 0.67 x tan(45)

$$R_R = \Phi_\tau (W_{T,R} \mu)$$ (1) Eq. 13

R_R = 25.64 klf = 38.467 klf x 0.667

OK

Calculate Direct Sliding at Base of RSF

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2$$ (3) Eq. 17

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF})$$ (3) Eq. 19

$F_{b,\ RSF}$ =	16.93 klf =	(0.5 x	125.00 pcf x	0.41 x	(23.32 ft +	2.5)^ 2)/	1000
$F_{t,\ RSF}$ =	0.00 klf =	0.00 psf x	0.41 x	(23.32 ft +	2.5 ft))/	1000	

$$F_{R,RSF} = \gamma_{EH\ MAX}(F_{b,RSF} + F_{rb,RSF}) + \gamma_{LS}F_{t,RSF}$$ (1) Eq. 20

$F_{R,RSF}$ = 25.39 klf = (1.5 x x 16.93 klf) + (1.75 x 0.00 klf)

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\ MIN}W_{RSF}$$ (1) Eq. 22

$W_{T,R,RSF}$ = 42.92 klf = 38.47 klf + (1.0 x 4.46 klf)

$$R_{R,RSF} = \Phi_\tau(W_{T,R,RSF}\mu_{RSF})$$ (1) Eq. 21

Note that ϕ = 1.0, per (1) 4.3.6.1.2
Per (2), and the previous calculation, the angle of internal friction beneath the RSF is equal to 31 degrees

$R_{R,\ RSF}$ = 25.79 klf = 1.0 x 42.92 klf x tan(31)

OK

Calculate External Bearing Resistance

Calculate V_R

Per (1) Eq. 26,

$$\sum V_R = \gamma_{EV\ MAX}(W) + \gamma_{EV\ MAX}(W_{RSF}) + \gamma_{DC\ MAX}(W_{face}) + \gamma_{LS}(q_t b_{rb,t}) + \gamma_{EH\ MAX}(q_{rb} b_{rb,t}) + \gamma_{DC\ MAX}(q_{DL} b) + \gamma_{LS}(q_{LL} b)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\ MIN}$

W =	43.45 klf =	1.35 x	32181.91 plf/	1000	
W_{RSF} =	6.02 klf =	1.35 x	4456.25 plf/	1000	
W_{FACE} =	2.48 klf =	1.25 x	1983.33 plf/	1000	
W_{SF} =	6.08 klf =	1.35 x	4500.00 plf/	1000	
Q_t =	0.00 klf =	1.75 x	(0.00 psf x	0.00 ft)/	1000
Total =	58.02 klf				

Calculate $M_{D,R}$

Per (1) Eq. 28

$$\sum M_{D,R} = \gamma_{EH\ MAX}F_{b,RSF}\left(\frac{H + D_{RSF}}{3}\right) + \gamma_{LS}F_{t,RSF}\left(\frac{H + D_{RSF}}{2}\right) + \gamma_{EH\ MAX}F_{rb,RSF}\left(\frac{H + D_{RSF}}{2}\right)$$

$F_{b,\ RSF}$ =	218.54 k-ft/ ft =	1.5 x	16.93 klf x	(0.33 x	(23.3202 ft +	2.5 ft))
$F_{t,\ RSF}$ =	0.00 k-ft/ ft =	1.75 x	0.00 klf x	(0.50 x	(23.3202 ft +	2.5 ft))
Total =	218.54 k-ft/ ft					

Note that $M_{D,R}$ is taken about the bottom center of the width of the RSF.

Calculate $M_{R,R}$

Per (1) Eq. 29

$$\sum M_{R,R} = (\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b)\left[\left(\frac{b}{2} + a_b\right) - \left(\frac{B_{RSF}}{2} - x_{RSF} - b_{block}\right)\right] + \left(\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t}\right)\left(\frac{B_{RSF}}{2} - \frac{b_{rb}}{2}\right) + \gamma_{EV\,MAX} W\left(\frac{B_{RSF}}{2} - \frac{B}{2}\right) + \gamma_{DC\,MAX} W_{face}\left(B + \frac{b_{block}}{2} - \frac{B_{RSF}}{2}\right)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\,MAX}$. Resisting component = W_{SF}*((X_{RSF}+D_{block}+(2/3)*B)-B_{RSF}/2)

W =	76.03 k-ft =	1.35 x	32.18 klf x	(7.75 ft -	6 ft)			
W_{Face} =	-12.72 k-ft =	1.35 x	1.98 klf x	((2.5 ft +	0.5 ft) -	7.75 ft)		
W_{SF} =	22.78 k-ft =	1.35 x	4.50 klf x	((2.5 ft +	1 ft +	(0.67 x	12 ft)) -	(15.5 ft/ 2))
Total =	86.09 k-ft/ ft							

Note that $M_{R,R}$ is taken about the bottom center of the width of the RSF.

Calculate Eccentricity, $e_{B,R}$

Per (1) Eq. 27

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R}$$

$e_{B,R}$ = 2.28 ft = $\frac{218.54 \text{ k-ft/ ft} - 86.09 \text{ k-ft/ ft}}{58.02 \text{ klf}}$

Calculate $\sigma_{v,base,R}$

Since $e_{B,R}$ = negative, set equal to zero in calcaulation of base bearing pressure.

Per (1) Eq. 25,

$$\sigma_{v,base,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}}$$

$\sigma_{v,base,R}$ = 5.31 ksf = 58.02 klf/ (15.5 ft - (2 x 2.28 ft)

Per bearing capacity calculation, allowable bearing capacity = 9.95 ksf

OK

Calculate Reinforcement Strength

q_t = 0.00 psf
W = 32181.91 psf
W_{RSF} = 4456.25 psf
W_{face} = 1983.33 psf

The evaulation of the abutment for the strength limit state is conducted according to (1) 4.3.7.3.1. and using the following equations:

Required Reinforcment:

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$

(3) Eq. 40

Factored Total Lateral Pressure:

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f}$$

(3) Eq. 41

Where:

$$\sigma_{h,W,f} = \gamma_{EH\,MAX}(\gamma_r z K_{ar})$$

Where:

$\gamma_{EH\,MAX}$ = maximum horizontal earth pressure load factor.
γ_r = unit weight of reinforced backfill.
z = depth from the top of the wall.
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

(3) Eq. 42

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar}$$

(3) Eq. 45

Note that $\sigma_{hbridge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Factored Reinforcement Strength:

$$T_{f,f} = \Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right) = 0.4T_f$$

(3) Eq. 48

Strength Limit State

z	$\sigma_{h,W,f}$	$\sigma_{h,t,f}$	$\sigma_{h,f}$	$T_{req,f}$	$T_{f,f}$	
0.67 ft	0.02 ksf	0.00 ksf	0.02 ksf	0.03 ksf	2.36 klf	OK
1.33 ft	0.05 ksf	0.00 ksf	0.05 ksf	0.06 ksf	2.36 klf	OK
2.00 ft	0.07 ksf	0.00 ksf	0.07 ksf	0.09 ksf	2.36 klf	OK
2.67 ft	0.10 ksf	0.00 ksf	0.10 ksf	0.12 ksf	2.36 klf	OK
3.33 ft	0.12 ksf	0.00 ksf	0.12 ksf	0.16 ksf	2.36 klf	OK
4.00 ft	0.15 ksf	0.00 ksf	0.15 ksf	0.19 ksf	2.36 klf	OK
4.67 ft	0.17 ksf	0.00 ksf	0.17 ksf	0.22 ksf	2.36 klf	OK
5.33 ft	0.20 ksf	0.00 ksf	0.20 ksf	0.25 ksf	2.36 klf	OK
6.00 ft	0.22 ksf	0.00 ksf	0.22 ksf	0.28 ksf	2.36 klf	OK
6.67 ft	0.25 ksf	0.00 ksf	0.25 ksf	0.31 ksf	2.36 klf	OK
7.33 ft	0.27 ksf	0.00 ksf	0.27 ksf	0.34 ksf	2.36 klf	OK
8.00 ft	0.30 ksf	0.00 ksf	0.30 ksf	0.37 ksf	2.36 klf	OK
8.67 ft	0.32 ksf	0.00 ksf	0.32 ksf	0.40 ksf	2.36 klf	OK
9.33 ft	0.35 ksf	0.00 ksf	0.35 ksf	0.44 ksf	2.36 klf	OK
10.00 ft	0.37 ksf	0.00 ksf	0.37 ksf	0.47 ksf	2.36 klf	OK
10.67 ft	0.40 ksf	0.00 ksf	0.40 ksf	0.50 ksf	2.36 klf	OK
11.33 ft	0.42 ksf	0.00 ksf	0.42 ksf	0.53 ksf	2.36 klf	OK
12.00 ft	0.45 ksf	0.00 ksf	0.45 ksf	0.56 ksf	2.36 klf	OK
12.67 ft	0.47 ksf	0.00 ksf	0.47 ksf	0.59 ksf	2.36 klf	OK
13.33 ft	0.49 ksf	0.00 ksf	0.49 ksf	0.62 ksf	2.36 klf	OK
14.00 ft	0.52 ksf	0.00 ksf	0.52 ksf	0.65 ksf	2.36 klf	OK
14.67 ft	0.54 ksf	0.00 ksf	0.54 ksf	0.68 ksf	2.36 klf	OK
15.33 ft	0.57 ksf	0.00 ksf	0.57 ksf	0.71 ksf	2.36 klf	OK
16.00 ft	0.59 ksf	0.00 ksf	0.59 ksf	0.75 ksf	2.36 klf	OK
16.67 ft	0.62 ksf	0.00 ksf	0.62 ksf	0.78 ksf	2.36 klf	OK
17.33 ft	0.64 ksf	0.00 ksf	0.64 ksf	0.81 ksf	2.36 klf	OK
18.00 ft	0.67 ksf	0.00 ksf	0.67 ksf	0.84 ksf	2.36 klf	OK
18.67 ft	0.69 ksf	0.00 ksf	0.69 ksf	0.87 ksf	2.36 klf	OK
19.33 ft	0.72 ksf	0.00 ksf	0.72 ksf	0.90 ksf	2.36 klf	OK
20.00 ft	0.74 ksf	0.00 ksf	0.74 ksf	0.93 ksf	2.36 klf	OK
20.67 ft	0.77 ksf	0.00 ksf	0.77 ksf	0.96 ksf	2.36 klf	OK
21.33 ft	0.79 ksf	0.00 ksf	0.79 ksf	0.99 ksf	2.36 klf	OK
22.00 ft	0.82 ksf	0.00 ksf	0.82 ksf	1.03 ksf	2.36 klf	OK
22.67 ft	0.84 ksf	0.00 ksf	0.84 ksf	1.06 ksf	2.36 klf	OK
23.32 ft	0.86 ksf	0.00 ksf	0.86 ksf	1.09 ksf	2.36 klf	OK

The evaulation of the abutment for the service limit state is conducted according to (1) 4.3.7.3.2. and using the following equations:

The nominal required reinforcement strength:

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 50

The lateral pressure within the GRS Abutment at a given dpeth and location:

$$\sigma_h = \sigma_{h,W} + \sigma_{h,bridge,eq} + \sigma_{h,rb} + \sigma_{h,t}$$ (3) Eq. 51

Where:

$$\sigma_{h,W} = \gamma_r z K_{ar}$$ (3) Eq. 2

$$\sigma_{h,t} = q_t K_{ar}$$ (3) Eq. 54

Note that $\sigma_{hbridge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Service Limit State

z	$\sigma_{h,W}$	$\sigma_{h,t}$	σ_h	T_{req}	$T_{@\varepsilon = 2\%}$	
0.67 ft	0.02 ksf	0.00 ksf	0.016 ksf	0.02 klf	1.25 klf	OK
1.33 ft	0.03 ksf	0.00 ksf	0.033 ksf	0.04 klf	1.25 klf	OK
2.00 ft	0.05 ksf	0.00 ksf	0.049 ksf	0.06 klf	1.25 klf	OK
2.67 ft	0.07 ksf	0.00 ksf	0.066 ksf	0.08 klf	1.25 klf	OK
3.33 ft	0.08 ksf	0.00 ksf	0.082 ksf	0.10 klf	1.25 klf	OK
4.00 ft	0.10 ksf	0.00 ksf	0.099 ksf	0.12 klf	1.25 klf	OK
4.67 ft	0.12 ksf	0.00 ksf	0.115 ksf	0.15 klf	1.25 klf	OK
5.33 ft	0.13 ksf	0.00 ksf	0.132 ksf	0.17 klf	1.25 klf	OK
6.00 ft	0.15 ksf	0.00 ksf	0.148 ksf	0.19 klf	1.25 klf	OK
6.67 ft	0.16 ksf	0.00 ksf	0.165 ksf	0.21 klf	1.25 klf	OK
7.33 ft	0.18 ksf	0.00 ksf	0.181 ksf	0.23 klf	1.25 klf	OK
8.00 ft	0.20 ksf	0.00 ksf	0.198 ksf	0.25 klf	1.25 klf	OK
8.67 ft	0.21 ksf	0.00 ksf	0.214 ksf	0.27 klf	1.25 klf	OK
9.33 ft	0.23 ksf	0.00 ksf	0.231 ksf	0.29 klf	1.25 klf	OK
10.00 ft	0.25 ksf	0.00 ksf	0.247 ksf	0.31 klf	1.25 klf	OK
10.67 ft	0.26 ksf	0.00 ksf	0.264 ksf	0.33 klf	1.25 klf	OK
11.33 ft	0.28 ksf	0.00 ksf	0.280 ksf	0.35 klf	1.25 klf	OK
12.00 ft	0.30 ksf	0.00 ksf	0.297 ksf	0.37 klf	1.25 klf	OK
12.67 ft	0.31 ksf	0.00 ksf	0.313 ksf	0.39 klf	1.25 klf	OK
13.33 ft	0.33 ksf	0.00 ksf	0.330 ksf	0.41 klf	1.25 klf	OK
14.00 ft	0.35 ksf	0.00 ksf	0.346 ksf	0.44 klf	1.25 klf	OK
14.67 ft	0.36 ksf	0.00 ksf	0.363 ksf	0.46 klf	1.25 klf	OK
15.33 ft	0.38 ksf	0.00 ksf	0.379 ksf	0.48 klf	1.25 klf	OK
16.00 ft	0.40 ksf	0.00 ksf	0.396 ksf	0.50 klf	1.25 klf	OK
16.67 ft	0.41 ksf	0.00 ksf	0.412 ksf	0.52 klf	1.25 klf	OK
17.33 ft	0.43 ksf	0.00 ksf	0.429 ksf	0.54 klf	1.25 klf	OK
18.00 ft	0.45 ksf	0.00 ksf	0.445 ksf	0.56 klf	1.25 klf	OK
18.67 ft	0.46 ksf	0.00 ksf	0.462 ksf	0.58 klf	1.25 klf	OK
19.33 ft	0.48 ksf	0.00 ksf	0.478 ksf	0.60 klf	1.25 klf	OK
20.00 ft	0.49 ksf	0.00 ksf	0.495 ksf	0.62 klf	1.25 klf	OK
20.67 ft	0.51 ksf	0.00 ksf	0.511 ksf	0.64 klf	1.25 klf	OK
21.33 ft	0.53 ksf	0.00 ksf	0.528 ksf	0.66 klf	1.25 klf	OK
22.00 ft	0.54 ksf	0.00 ksf	0.544 ksf	0.68 klf	1.25 klf	OK
22.67 ft	0.56 ksf	0.00 ksf	0.561 ksf	0.70 klf	1.25 klf	OK
23.32 ft	0.58 ksf	0.00 ksf	0.577 ksf	0.72 klf	1.25 klf	OK

References:

1) GRS-IBS Interim Implementation Guide, FHWA, June 2018
(2) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018
(3) Geotechnical Data Report, Teracon Consultants Inc., March 27. 2019

Material Properties

γ_f = 115.00 pcf (Reinforced Fill)
ϕ (Reinforced Soil) = 45 degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β = 26.57 degrees

K_a (Reinforced Soil) =	0.22 =	0.894 x	0.894 -	(0.800 -	0.5)^ 0.5
			0.894 +	(0.800 -	0.5)^ 0.5

Fill retained is in-situ soil.

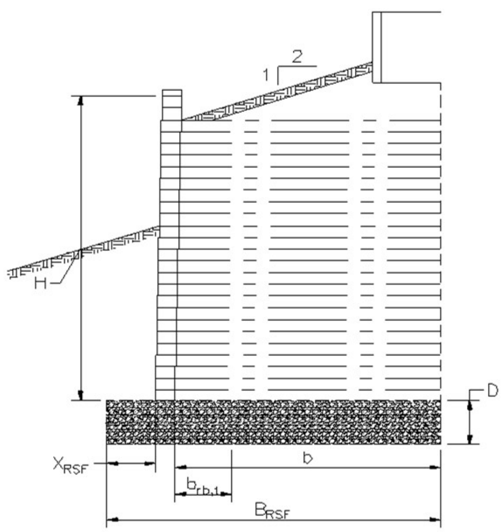
γ_b = 125.00 pcf (Retained Fill)
ϕ (Existing Filll) = 34.00 degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β = 26.57 degrees

K_a (Retained Fill) =	0.41 =	0.894 x	0.894 -	(0.800 -	0.6873)^ 0.5
			0.894 +	(0.800 -	0.6873)^ 0.5

Geometry



Depth of RSF, D =	2.5 ft		
Top of RSF EL. =	456.46		
Bottom of RSF EL. =	453.96 =	456.46 -	2.5 ft
Top of Wall EL. =	476.28		
Wall Height =	19.82 ft	476.28 -	456.46
H =	19.82 ft		
B =	10.25 ft		
Toe Length, X_{RSF} =	2.5 ft		
Total Width, B_{RSF} =	13.75 ft		
$b_{rb,t}$ =	0.00 ft		
Stepped Footing Length =	16.50 ft		
h_{block} =	8.00 in		
D_{block} =	12.00 in		
L_{block} =	18.00 in		
Weight =	85 lbs	per block	

Reinforcement

S_v = 0.67 ft
d_{max} = 0.75 in
T_f = 5.90 klf

Loading
Note that the superstructure dead loads, roadway bed loads, and traffic live loads are not applicable for this structure. No live load surcharge is applied.

Load Factors
Per (3) Table 3.4.1-1 and Table 3.4.1-2.

γ_p (DC, Min.) =	0.90	3) Table 3.4.1-2
γ_p (DC, Max.) =	1.25	3) Table 3.4.1-2
γ_p (EH, Max) =	1.50	3) Table 3.4.1-2
γ_p (EV, Min.) =	1.00	3) Table 3.4.1-2
γ_p (EV, Max.) =	1.35	3) Table 3.4.1-2
Factor (LS) =	1.75	3) Table 3.4.1-1

Calculate Vertical Loads and Applied Pressures
Calculate Traffic Surcharge, q_T

Due to wingwall layout geometry, assume that there is no live load surcharge acting on the wingwall.

h_{eq} = 0.00 ft
γ_r= 125.00 pcf

$$q_t = h_{eq}\gamma_b$$

q_T = 0.00 psf = 0.00 ft x 125.00 pcf

Calculate Weight of GRS Abutment

$$W = BH\gamma_r$$

H = 19.82 ft
B = 10.25 ft
γ_r= 115.00 pcf

W = 23367.54 plf = 19.824 ft x 10.25 ft x 115.00 pcf

Calculate Weight of RSF

$$W_{RSF} = B_{RSF}D_{RSF}\gamma_r$$

B = 13.75 ft
D = 2.5 ft
γ_r= 125.00 pcf

W_{RSF} = 3953.13 plf = 13.75 ft x 2.5 ft x 115.00 pcf

Calculate Weight of Facing

N_{block} = 29.74 = 19.824 ft/ 0.67 ft
N_{block} = 30

$$W_{face} = N_{block}\frac{W_{block}}{L_{block}}$$

W_{face} = 1700.00 plf = 30 x (85 lbs/ (18.00 in/ 12)

Calculate Weight of Surplus Retained Fill

Fill is retained at a 2H:1V slope

B = 10.25 ft
H_{max} = 5.13 ft = 10.25 ft/ 2
W_{SF} = 3283.20 plf = 0.5 x 10.25 ft x 5.13 ft x 125.00 pcf

Summary of Applied Loads

q_t = 0.00 psf
W = 23367.54 plf
W_{RSF} = 3953.13 plf
W_{face} = 1700.00 plf
W_{SF} = 3283.20 plf

Calculte Direct Sliding Effects at RSF/GRS Interface

Calculate Sliding per the (1) and (3) Strength I Load Combination

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2$$ (1) Eq. 9

$$F_t = q_t K_{ab} H$$ (1) Eq. 11

$$F_R = \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t$$ (1) Eq. 12

F_b = 9978.23 lbs = 0.5 x 125.00 pcf x 0.41 x 19.824 ft^ 2
F_t = 0.00 lbs = 0.00 psf 0.41 x 19.824 ft

F_R = 14.97 klf = (1.5 x 9978.23 lbs) + (1.75 x 0.00 lbs))/ 1000

$$W_{T,R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EV\,MIN}(q_{rb} b_{rb,t}) + \gamma_{EV\,MIN} W_{SF}$$ (1) Eq. 14

$W_{T,R}$ = 28.18 klf = ((1.0 x 23367.54 plf) + ((0.9 x 1700.00 plf))/ 1000) + (1.00 x 3.28 klf)

Assume that μ = 2/3*tanϕ, per (1) C.1.6.1.1

μ = 0.67 = 0.67 x tan(45)

$$R_R = \Phi_\tau (W_{T,R}\mu)$$ (1) Eq. 13

R_R = 18.79 klf = 28.181 klf x 0.667

OK

Calculate Direct Sliding at Base of RSF

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2$$ (3) Eq. 17

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF})$$ (3) Eq. 19

$F_{b,\ RSF}$ =	12.65 klf =	(0.5 x	125.00 pcf x	0.41 x	(19.82 ft +	2.5)^ 2)/	1000
$F_{t,\ RSF}$ =	0.00 klf =	0.00 psf x	0.41 x	(19.82 ft +	2.5 ft))/	1000	

$$F_{R,RSF} = \gamma_{EH\ MAX}(F_{b,RSF} + F_{rb,RSF}) + \gamma_{LS}F_{t,RSF}$$ (1) Eq. 20

$F_{R,RSF}$ = 18.98 klf = (1.5 x x 12.65 klf) + (1.75 x 0.00 klf)

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\ MIN}W_{RSF}$$ (1) Eq. 22

$W_{T,R,RSF}$ = 32.13 klf = 28.18 klf + (1.0 x 3.95 klf)

$$R_{R,RSF} = \Phi_\tau(W_{T,R,RSF}\mu_{RSF})$$ (1) Eq. 21

Note that ϕ = 1.0, per (1) 4.3.6.1.2
Per (2), and the previous calculation, the angle of internal friction beneath the RSF is equal to 31 degrees

$R_{R,\ RSF}$ = 19.31 klf = 1.0 x 32.13 klf x tan(31)

OK

Calculate External Bearing Resistance

Calculate V_R

Per (1) Eq. 26,

$$\sum V_R = \gamma_{EV\ MAX}(W) + \gamma_{EV\ MAX}(W_{RSF}) + \gamma_{DC\ MAX}(W_{face}) + \gamma_{LS}(q_t b_{rb,t}) + \gamma_{EH\ MAX}(q_{rb} b_{rb,t}) + \gamma_{DC\ MAX}(q_{DL} b) + \gamma_{LS}(q_{LL} b)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\ MIN}$

W =	31.55 klf =	1.35 x	23367.54 plf/	1000	
W_{RSF} =	5.34 klf =	1.35 x	3953.13 plf/	1000	
W_{FACE} =	2.13 klf =	1.25 x	1700.00 plf/	1000	
W_{SF} =	4.43 klf =	1.35 x	3283.20 plf/	1000	
Q_t =	0.00 klf =	1.75 x	(0.00 psf x	0.00 ft)/	1000
Total =	43.44 klf				

Calculate $M_{D,R}$

Per (1) Eq. 28

$$\sum M_{D,R} = \gamma_{EH\ MAX}F_{b,RSF}\left(\frac{H + D_{RSF}}{3}\right) + \gamma_{LS}F_{t,RSF}\left(\frac{H + D_{RSF}}{2}\right) + \gamma_{EH\ MAX}F_{rb,RSF}\left(\frac{H + D_{RSF}}{2}\right)$$

$F_{b,\ RSF}$ =	141.24 k-ft/ ft =	1.5 x	12.65 klf x	(0.33 x	(19.824 ft +	2.5 ft))
$F_{t,\ RSF}$ =	0.00 k-ft/ ft =	1.75 x	0.00 klf x	(0.50 x	(19.824 ft +	2.5 ft))
Total =	141.24 k-ft/ ft					

Note that $M_{D,R}$ is taken about the bottom center of the width of the RSF.

Calculate $M_{R,R}$

Per (1) Eq. 29

$$\sum M_{R,R} = (\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b)\left[\left(\frac{b}{2} + a_b\right) - \left(\frac{B_{RSF}}{2} - x_{RSF} - b_{block}\right)\right] + \left(\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t}\right)\left(\frac{B_{RSF}}{2} - \frac{b_{rb}}{2}\right) + \gamma_{EV\,MAX} W\left(\frac{B_{RSF}}{2} - \frac{B}{2}\right) + \gamma_{DC\,MAX} W_{face}\left(B + \frac{b_{block}}{2} - \frac{B_{RSF}}{2}\right)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\ MAX}$. Resisting component = W_{SF}*((X_{RSF}+D_{block}+(2/3)*B)-BRSF/2)

W =	55.21 k-ft =	1.35 x	23.37 klf x	(6.875 ft -	5.125 ft)			
W_{Face} =	-8.89 k-ft =	1.35 x	1.70 klf x	((2.5 ft +	0.5 ft) -	6.875 ft)		
W_{SF} =	15.33 k-ft =	1.35 x	3.28 klf x	((2.5 ft +	1 ft +	(0.67 x	10.25 ft)) -	(13.75 ft/ 2))
Total =	61.64 k-ft/ ft							

Note that $M_{R,R}$ is taken about the bottom center of the width of the RSF.

Calculate Eccentricity, $e_{B,R}$

Per (1) Eq. 27

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R}$$

$e_{B,R}$ = 1.83 ft = $\frac{141.24 \text{ k-ft/ ft} - 61.64 \text{ k-ft/ ft}}{43.44 \text{ klf}}$

Calculate $\sigma_{v,base,R}$

Since $e_{B,R}$ = negative, set equal to zero in calcaulation of base bearing pressure.

Per (1) Eq. 25,

$$\sigma_{v,base,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}}$$

$\sigma_{v,base,R}$ = 4.31 ksf = 43.44 klf/ (13.75 ft - (2 x 1.83 ft)

Per abutment bearing capacity calculation, allowable bearing capacity = 9.95 ksf

OK

Calculate Reinforcement Strength

q_t = 0.00 psf
W = 23367.54 psf
W_{RSF} = 3953.13 psf
W_{face} = 1700.00 psf

The evaulation of the abutment for the strength limit state is conducted according to (1) 4.3.7.3.1. and using the following equations:

Required Reinforcment:

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 40

Factored Total Lateral Pressure:

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f}$$ (3) Eq. 41

Where:

$$\sigma_{h,W,f} = \gamma_{EH\ MAX}(\gamma_r z K_{ar})$$

Where:
$\gamma_{EH\ MAX}$ = maximum horizontal earth pressure load factor.
γ_r = unit weight of reinforced backfill.
z = depth from the top of the wall.
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

(3) Eq. 42

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar}$$ (3) Eq. 45

Note that $\sigma_{hbridge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Factored Reinforcement Strength:

$$T_{f,f} = \Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right) = 0.4T_f$$ (3) Eq. 48

Strength Limit State

z	$\sigma_{h,W,f}$	$\sigma_{h,t,f}$	$\sigma_{h,f}$	$T_{req,f}$	$T_{f,f}$	
0.67 ft	0.02 ksf	0.00 ksf	0.02 ksf	0.03 ksf	2.36 klf	OK
1.33 ft	0.05 ksf	0.00 ksf	0.05 ksf	0.06 ksf	2.36 klf	OK
2.00 ft	0.07 ksf	0.00 ksf	0.07 ksf	0.09 ksf	2.36 klf	OK
2.67 ft	0.10 ksf	0.00 ksf	0.10 ksf	0.12 ksf	2.36 klf	OK
3.33 ft	0.12 ksf	0.00 ksf	0.12 ksf	0.16 ksf	2.36 klf	OK
4.00 ft	0.15 ksf	0.00 ksf	0.15 ksf	0.19 ksf	2.36 klf	OK
4.67 ft	0.17 ksf	0.00 ksf	0.17 ksf	0.22 ksf	2.36 klf	OK
5.33 ft	0.20 ksf	0.00 ksf	0.20 ksf	0.25 ksf	2.36 klf	OK
6.00 ft	0.22 ksf	0.00 ksf	0.22 ksf	0.28 ksf	2.36 klf	OK
6.67 ft	0.25 ksf	0.00 ksf	0.25 ksf	0.31 ksf	2.36 klf	OK
7.33 ft	0.27 ksf	0.00 ksf	0.27 ksf	0.34 ksf	2.36 klf	OK
8.00 ft	0.30 ksf	0.00 ksf	0.30 ksf	0.37 ksf	2.36 klf	OK
8.67 ft	0.32 ksf	0.00 ksf	0.32 ksf	0.40 ksf	2.36 klf	OK
9.33 ft	0.35 ksf	0.00 ksf	0.35 ksf	0.44 ksf	2.36 klf	OK
10.00 ft	0.37 ksf	0.00 ksf	0.37 ksf	0.47 ksf	2.36 klf	OK
10.67 ft	0.40 ksf	0.00 ksf	0.40 ksf	0.50 ksf	2.36 klf	OK
11.33 ft	0.42 ksf	0.00 ksf	0.42 ksf	0.53 ksf	2.36 klf	OK
12.00 ft	0.45 ksf	0.00 ksf	0.45 ksf	0.56 ksf	2.36 klf	OK
12.67 ft	0.47 ksf	0.00 ksf	0.47 ksf	0.59 ksf	2.36 klf	OK
13.33 ft	0.49 ksf	0.00 ksf	0.49 ksf	0.62 ksf	2.36 klf	OK
14.00 ft	0.52 ksf	0.00 ksf	0.52 ksf	0.65 ksf	2.36 klf	OK
14.67 ft	0.54 ksf	0.00 ksf	0.54 ksf	0.68 ksf	2.36 klf	OK
15.33 ft	0.57 ksf	0.00 ksf	0.57 ksf	0.71 ksf	2.36 klf	OK
16.00 ft	0.59 ksf	0.00 ksf	0.59 ksf	0.75 ksf	2.36 klf	OK
16.67 ft	0.62 ksf	0.00 ksf	0.62 ksf	0.78 ksf	2.36 klf	OK
17.33 ft	0.64 ksf	0.00 ksf	0.64 ksf	0.81 ksf	2.36 klf	OK
18.00 ft	0.67 ksf	0.00 ksf	0.67 ksf	0.84 ksf	2.36 klf	OK
18.67 ft	0.69 ksf	0.00 ksf	0.69 ksf	0.87 ksf	2.36 klf	OK
19.33 ft	0.72 ksf	0.00 ksf	0.72 ksf	0.90 ksf	2.36 klf	OK
19.82 ft	0.74 ksf	0.00 ksf	0.74 ksf	0.92 ksf	2.36 klf	OK

The evaulation of the abutment for the service limit state is conducted according to (1) 4.3.7.3.2. and using the following equations:

The nominal required reinforcement strength:

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 50

The lateral pressure within the GRS Abutment at a given dpeth and location:

$$\sigma_h = \sigma_{h,W} + \sigma_{h,bridge,eq} + \sigma_{h,rb} + \sigma_{h,t}$$ (3) Eq. 51

Where:

$$\sigma_{h,W} = \gamma_r z K_{ar}$$ (3) Eq. 2

$$\sigma_{h,t} = q_t K_{ar}$$ (3) Eq. 54

Note that $\sigma_{hbrdge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Service Limit State

z	$\sigma_{h,W}$	$\sigma_{h,t}$	σ_h	T_{req}	$T_{@\varepsilon = 2\%}$	
0.67 ft	0.02 ksf	0.00 ksf	0.016 ksf	0.02 klf	1.25 klf	OK
1.33 ft	0.03 ksf	0.00 ksf	0.033 ksf	0.04 klf	1.25 klf	OK
2.00 ft	0.05 ksf	0.00 ksf	0.049 ksf	0.06 klf	1.25 klf	OK
2.67 ft	0.07 ksf	0.00 ksf	0.066 ksf	0.08 klf	1.25 klf	OK
3.33 ft	0.08 ksf	0.00 ksf	0.082 ksf	0.10 klf	1.25 klf	OK
4.00 ft	0.10 ksf	0.00 ksf	0.099 ksf	0.12 klf	1.25 klf	OK
4.67 ft	0.12 ksf	0.00 ksf	0.115 ksf	0.15 klf	1.25 klf	OK
5.33 ft	0.13 ksf	0.00 ksf	0.132 ksf	0.17 klf	1.25 klf	OK
6.00 ft	0.15 ksf	0.00 ksf	0.148 ksf	0.19 klf	1.25 klf	OK
6.67 ft	0.16 ksf	0.00 ksf	0.165 ksf	0.21 klf	1.25 klf	OK
7.33 ft	0.18 ksf	0.00 ksf	0.181 ksf	0.23 klf	1.25 klf	OK
8.00 ft	0.20 ksf	0.00 ksf	0.198 ksf	0.25 klf	1.25 klf	OK
8.67 ft	0.21 ksf	0.00 ksf	0.214 ksf	0.27 klf	1.25 klf	OK
9.33 ft	0.23 ksf	0.00 ksf	0.231 ksf	0.29 klf	1.25 klf	OK
10.00 ft	0.25 ksf	0.00 ksf	0.247 ksf	0.31 klf	1.25 klf	OK
10.67 ft	0.26 ksf	0.00 ksf	0.264 ksf	0.33 klf	1.25 klf	OK
11.33 ft	0.28 ksf	0.00 ksf	0.280 ksf	0.35 klf	1.25 klf	OK
12.00 ft	0.30 ksf	0.00 ksf	0.297 ksf	0.37 klf	1.25 klf	OK
12.67 ft	0.31 ksf	0.00 ksf	0.313 ksf	0.39 klf	1.25 klf	OK
13.33 ft	0.33 ksf	0.00 ksf	0.330 ksf	0.41 klf	1.25 klf	OK
14.00 ft	0.35 ksf	0.00 ksf	0.346 ksf	0.44 klf	1.25 klf	OK
14.67 ft	0.36 ksf	0.00 ksf	0.363 ksf	0.46 klf	1.25 klf	OK
15.33 ft	0.38 ksf	0.00 ksf	0.379 ksf	0.48 klf	1.25 klf	OK
16.00 ft	0.40 ksf	0.00 ksf	0.396 ksf	0.50 klf	1.25 klf	OK
16.67 ft	0.41 ksf	0.00 ksf	0.412 ksf	0.52 klf	1.25 klf	OK
17.33 ft	0.43 ksf	0.00 ksf	0.429 ksf	0.54 klf	1.25 klf	OK
18.00 ft	0.45 ksf	0.00 ksf	0.445 ksf	0.56 klf	1.25 klf	OK
18.67 ft	0.46 ksf	0.00 ksf	0.462 ksf	0.58 klf	1.25 klf	OK
19.33 ft	0.48 ksf	0.00 ksf	0.478 ksf	0.60 klf	1.25 klf	OK
19.82 ft	0.49 ksf	0.00 ksf	0.490 ksf	0.62 klf	1.25 klf	OK

References:

1) GRS-IBS Interim Implementation Guide, FHWA, June 2018
(2) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018
(3) Geotechnical Data Report, Teracon Consultants Inc., March 27. 2019

Material Properties

γ_f =	115.00 pcf	(Reinforced Fill)
ϕ (Reinforced Soil) =	45	degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β = 26.57 degrees

K_a (Reinforced Soil) = 0.22 = 0.894 x $\frac{0.894 - (0.800 - 0.5)^\wedge 0.5}{0.894 + (0.800 - 0.5)^\wedge 0.5}$

Fill retained is in-situ soil.

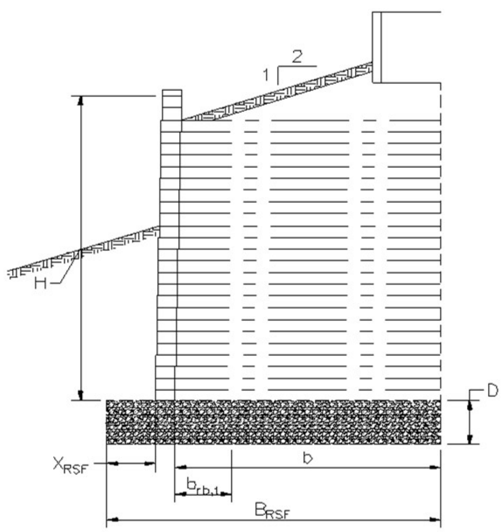
γ_b =	125.00 pcf	(Retained Fill)
ϕ (Existing Filll) =	34.00	degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β = 26.57 degrees

K_a (Retained Fill) = 0.41 = 0.894 x $\frac{0.894 - (0.800 - 0.6873)^\wedge 0.5}{0.894 + (0.800 - 0.6873)^\wedge 0.5}$

Geometry



Depth of RSF, D =	2.5 ft		
Top of RSF EL. =	456.46		
Bottom of RSF EL. =	453.96 =	456.46 -	2.5 ft
Top of Wall EL. =	476.17		
Wall Height =	19.71 ft	476.17 -	456.46
H =	19.71 ft		
B =	9.75 ft		
Toe Length, X_{RSF} =	2.5 ft		
Total Width, B_{RSF} =	13.25 ft		
$b_{rb,t}$ =	0.00 ft		
Stepped Footing Length =	16.00 ft		
h_{block} =	8.00 in		
D_{block} =	12.00 in		
L_{block} =	18.00 in		
Weight =	85 lbs	per block	

Reinforcement

S_v =	0.67 ft
d_{max} =	0.50 in
T_f =	5.90 klf

Loading
Note that the superstructure dead loads, roadway bed loads, and traffic live loads are not applicable for this structure. No live load surcharge is applied.

Load Factors
Per (3) Table 3.4.1-1 and Table 3.4.1-2.

γ_p (DC, Min.) =	0.90	3) Table 3.4.1-2
γ_p (DC, Max.) =	1.25	3) Table 3.4.1-2
γ_p (EH, Max) =	1.50	3) Table 3.4.1-2
γ_p (EV, Min.) =	1.00	3) Table 3.4.1-2
γ_p (EV, Max.) =	1.35	3) Table 3.4.1-2
Factor (LS) =	1.75	3) Table 3.4.1-1

Calculate Vertical Loads and Applied Pressures
Calculate Traffic Surcharge, q_T

Due to wingwall layout geometry, assume that there is no live load surcharge acting on the wingwall.

h_{eq} = 0.00 ft
γ_r= 125.00 pcf

$$q_t = h_{eq}\gamma_b$$

q_T = 0.00 psf = 0.00 ft x 125.00 pcf

Calculate Weight of GRS Abutment

$$W = BH\gamma_r$$

H = 19.71 ft
B = 9.75 ft
γ_r= 115.00 pcf

W = 22101.71 plf = 19.71167 ft x 9.75 ft x 115.00 pcf

Calculate Weight of RSF

$$W_{RSF} = B_{RSF}D_{RSF}\gamma_r$$

B = 13.25 ft
D = 2.5 ft
γ_r= 125.00 pcf

W_{RSF} = 3809.38 plf = 13.25 ft x 2.5 ft x 115.00 pcf

Calculate Weight of Facing

N_{block} = 29.57 = 19.711674 ft/ 0.67 ft
N_{block} = 30

$$W_{face} = N_{block}\frac{W_{block}}{L_{block}}$$

W_{face} = 1700.00 plf = 30 x (85 lbs/ (18.00 in/ 12)

Calculate Weight of Surplus Retained Fill

Fill is retained at a 2H:1V slope

B =	9.75 ft					
H_{max} =	4.88 ft =	9.75 ft/ 2				
W_{SF} =	2970.70 plf =	0.5 x	9.75 ft x	4.88 ft x	125.00 pcf	

Summary of Applied Loads

q_t =	0.00 psf
W =	22101.71 plf
W_{RSF} =	3809.38 plf
W_{face} =	1700.00 plf
W_{SF} =	2970.70 plf

Calculte Direct Sliding Effects at RSF/GRS Interface

Calculate Sliding per the (1) and (3) Strength I Load Combination

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2$$ (1) Eq. 9

$$F_t = q_t K_{ab} H$$ (1) Eq. 11

$$F_R = \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t$$ (1) Eq. 12

F_b =	9865.48 lbs =	0.5 x	125.00 pcf x	0.41 x	19.71167 ft^ 2	
F_t =	0.00 lbs =	0.00 psf	0.41 x	19.7116738 ft		
F_R =	14.80 klf =	(1.5 x	9865.48 lbs) +	(1.75 x	0.00 lbs))/	1000

$$W_{T,R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EV\,MIN}(q_{rb} b_{rb,t})$$ + $\gamma_{EV\,MIN} W_{SF}$ (1) Eq. 14

$W_{T,R}$ = 26.60 klf = ((1.0 x 22101.71 plf) + ((0.9 x 1700.00 plf))/ 1000) + (1.00 x 2.97 klf)

Assume that μ = 2/3*tanϕ, per (1) C.1.6.1.1

μ = 0.67 = 0.67 x tan(45)

$$R_R = \Phi_\tau (W_{T,R}\mu)$$ (1) Eq. 13

R_R = 17.73 klf = 26.602 klf x 0.667

OK

Calculate Direct Sliding at Base of RSF

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2$$ (3) Eq. 17

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF})$$ (3) Eq. 19

$F_{b,\ RSF}$ =	12.53 klf =	(0.5 x	125.00 pcf x	0.41 x	(19.71 ft +	2.5)^ 2)/	1000	
$F_{t,\ RSF}$ =	0.00 klf =	0.00 psf x	0.41 x	(19.71 ft +	2.5 ft))/	1000		

$$F_{R,RSF} = \gamma_{EH\,MAX}\left(F_{b,RSF} + F_{rb,RSF}\right) + \gamma_{LS}F_{t,RSF}$$ (1) Eq. 20

$F_{R,RSF}$ = 18.79 klf = (1.5 x x 12.53 klf) + (1.75 x 0.00 klf)

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\,MIN}W_{RSF}$$ (1) Eq. 22

$W_{T,R,RSF}$ = 30.41 klf = 26.60 klf + (1.0 x 3.81 klf)

$$R_{R,RSF} = \Phi_\tau(W_{T,R,RSF}\mu_{RSF})$$ (1) Eq. 21

Note that ϕ = 1.0, per (1) 4.3.6.1.2
Per (2), and the previous calculation, the angle of internal friction beneath the RSF is equal to 32 degrees

$R_{R,\ RSF}$ = 19.00 klf = 1.0 x 30.41 klf x tan(32)

OK

Calculate External Bearing Resistance

Calculate V_R

Per (1) Eq. 26,

$$\sum V_R = \gamma_{EV\,MAX}(W) + \gamma_{EV\,MAX}(W_{RSF}) + \gamma_{DC\,MAX}(W_{face}) + \gamma_{LS}(q_t b_{rb,t}) + \gamma_{EH\,MAX}(q_{rb} b_{rb,t}) + \gamma_{DC\,MAX}(q_{DL} b) + \gamma_{LS}(q_{LL} b)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\ MIN}$

W =	29.84 klf =	1.35 x	22101.71 plf/	1000	
W_{RSF} =	5.14 klf =	1.35 x	3809.38 plf/	1000	
W_{FACE} =	2.13 klf =	1.25 x	1700.00 plf/	1000	
W_{SF} =	4.01 klf =	1.35 x	2970.70 plf/	1000	
Q_t =	0.00 klf =	1.75 x	(0.00 psf x	0.00 ft)/	1000
Total =	41.12 klf				

Calculate $M_{D,R}$

Per (1) Eq. 28

$$\sum M_{D,R} = \gamma_{EH\,MAX} F_{b,RSF}\left(\frac{H + D_{RSF}}{3}\right) + \gamma_{LS} F_{t,RSF}\left(\frac{H + D_{RSF}}{2}\right) + \gamma_{EH\,MAX} F_{rb,RSF}\left(\frac{H + D_{RSF}}{2}\right)$$

$F_{b,\,RSF}$ =	139.12 k-ft/ ft =	1.5 x	12.53 klf x	(0.33 x	(19.7117 ft +	2.5 ft))
$F_{t,\,RSF}$ =	0.00 k-ft/ ft =	1.75 x	0.00 klf x	(0.50 x	(19.7117 ft +	2.5 ft))
Total =	139.12 k-ft/ ft					

Note that $M_{D,R}$ is taken about the bottom center of the width of the RSF.

Calculate $M_{R,R}$

Per (1) Eq. 29

$$\sum M_{R,R} = \left(\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b\right)\left[\left(\frac{b}{2} + a_b\right) - \left(\frac{B_{RSF}}{2} - x_{RSF} - b_{block}\right)\right] + \left(\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t}\right)\left(\frac{B_{RSF}}{2} - \frac{b_{rb}}{2}\right) + \gamma_{EV\,MAX} W\left(\frac{B_{RSF}}{2} - \frac{B}{2}\right) + \gamma_{DC\,MAX} W_{face}\left(B + \frac{b_{block}}{2} - \frac{B_{RSF}}{2}\right)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\,MAX}$. Resisting component = W_{SF}*((X_{RSF}+D_{block}+(2/3)*B)-BRSF/2)

W =	52.22 k-ft =	1.35 x	22.10 klf x	(6.625 ft -	4.875 ft)				
W_{Face} =	-8.32 k-ft =	1.35 x	1.70 klf x	((2.5 ft +	0.5 ft) -	6.625 ft)			
W_{SF} =	13.54 k-ft =	1.35 x	2.97 klf x	((2.5 ft +	1 ft +	(0.67 x	9.75 ft)) -	(13.25 ft/ 2))	
Total =	57.43 k-ft/ ft								

Note that $M_{R,R}$ is taken about the bottom center of the width of the RSF.

Calculate Eccentricity, $e_{B,R}$

Per (1) Eq. 27

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R}$$

$e_{B,R}$ = 1.99 ft = (139.12 k-ft/ ft - 57.43 k-ft/ ft) / 41.12 klf

Calculate $\sigma_{v,base,R}$

Since $e_{B,R}$ = negative, set equal to zero in calcaulation of base bearing pressure.

Per (1) Eq. 25,

$$\sigma_{v,base,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}}$$

$\sigma_{v,base,R}$ = 4.43 ksf = 41.12 klf/ (13.25 ft - (2 x 1.99 ft)

Per abutment bearing capacity calculation, allowable bearing capacity = 11.02 ksf

OK

Calculate Reinforcement Strength

q_t = 0.00 psf
W = 22101.71 psf
W_{RSF} = 3809.38 psf
W_{face} = 1700.00 psf

The evaulation of the abutment for the strength limit state is conducted according to (1) 4.3.7.3.1. and using the following equations:

Required Reinforcment:

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$

(3) Eq. 40

Factored Total Lateral Pressure:

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f}$$

(3) Eq. 41

Where:

$$\sigma_{h,W,f} = \gamma_{EH\ MAX}\left(\gamma_r z K_{ar}\right)$$

Where:
$\gamma_{EH\ MAX}$ = maximum horizontal earth pressure load factor.
γ_r = unit weight of reinforced backfill.
z = depth from the top of the wall.
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

(3) Eq. 42

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar}$$

(3) Eq. 45

Note that $\sigma_{h,bridge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Factored Reinforcement Strength:

$$T_{f,f} = \Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right) = 0.4T_f$$

(3) Eq. 48

Strength Limit State

z	$\sigma_{h,W,f}$	$\sigma_{h,t,f}$	$\sigma_{h,f}$	$T_{req,f}$	$T_{f,f}$	
0.67 ft	0.02 ksf	0.00 ksf	0.02 ksf	0.04 ksf	2.36 klf	OK
1.33 ft	0.05 ksf	0.00 ksf	0.05 ksf	0.09 ksf	2.36 klf	OK
2.00 ft	0.07 ksf	0.00 ksf	0.07 ksf	0.13 ksf	2.36 klf	OK
2.67 ft	0.10 ksf	0.00 ksf	0.10 ksf	0.17 ksf	2.36 klf	OK
3.33 ft	0.12 ksf	0.00 ksf	0.12 ksf	0.21 ksf	2.36 klf	OK
4.00 ft	0.15 ksf	0.00 ksf	0.15 ksf	0.26 ksf	2.36 klf	OK
4.67 ft	0.17 ksf	0.00 ksf	0.17 ksf	0.30 ksf	2.36 klf	OK
5.33 ft	0.20 ksf	0.00 ksf	0.20 ksf	0.34 ksf	2.36 klf	OK
6.00 ft	0.22 ksf	0.00 ksf	0.22 ksf	0.38 ksf	2.36 klf	OK
6.67 ft	0.25 ksf	0.00 ksf	0.25 ksf	0.43 ksf	2.36 klf	OK
7.33 ft	0.27 ksf	0.00 ksf	0.27 ksf	0.47 ksf	2.36 klf	OK
8.00 ft	0.30 ksf	0.00 ksf	0.30 ksf	0.51 ksf	2.36 klf	OK
8.67 ft	0.32 ksf	0.00 ksf	0.32 ksf	0.55 ksf	2.36 klf	OK
9.33 ft	0.35 ksf	0.00 ksf	0.35 ksf	0.60 ksf	2.36 klf	OK
10.00 ft	0.37 ksf	0.00 ksf	0.37 ksf	0.64 ksf	2.36 klf	OK
10.67 ft	0.40 ksf	0.00 ksf	0.40 ksf	0.68 ksf	2.36 klf	OK
11.33 ft	0.42 ksf	0.00 ksf	0.42 ksf	0.73 ksf	2.36 klf	OK
12.00 ft	0.45 ksf	0.00 ksf	0.45 ksf	0.77 ksf	2.36 klf	OK
12.67 ft	0.47 ksf	0.00 ksf	0.47 ksf	0.81 ksf	2.36 klf	OK
13.33 ft	0.49 ksf	0.00 ksf	0.49 ksf	0.85 ksf	2.36 klf	OK
14.00 ft	0.52 ksf	0.00 ksf	0.52 ksf	0.90 ksf	2.36 klf	OK
14.67 ft	0.54 ksf	0.00 ksf	0.54 ksf	0.94 ksf	2.36 klf	OK
15.33 ft	0.57 ksf	0.00 ksf	0.57 ksf	0.98 ksf	2.36 klf	OK
16.00 ft	0.59 ksf	0.00 ksf	0.59 ksf	1.02 ksf	2.36 klf	OK
16.67 ft	0.62 ksf	0.00 ksf	0.62 ksf	1.07 ksf	2.36 klf	OK
17.33 ft	0.64 ksf	0.00 ksf	0.64 ksf	1.11 ksf	2.36 klf	OK
18.00 ft	0.67 ksf	0.00 ksf	0.67 ksf	1.15 ksf	2.36 klf	OK
18.67 ft	0.69 ksf	0.00 ksf	0.69 ksf	1.19 ksf	2.36 klf	OK
19.33 ft	0.72 ksf	0.00 ksf	0.72 ksf	1.24 ksf	2.36 klf	OK
19.71 ft	0.73 ksf	0.00 ksf	0.73 ksf	1.26 ksf	2.36 klf	OK

The evaulation of the abutment for the service limit state is conducted according to (1) 4.3.7.3.2. and using the following equations:

The nominal required reinforcement strength:

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 50

The lateral pressure within the GRS Abutment at a given dpeth and location:

$$\sigma_h = \sigma_{h,W} + \sigma_{h,bridge,eq} + \sigma_{h,rb} + \sigma_{h,t}$$ (3) Eq. 51

Where:

$$\sigma_{h,W} = \gamma_r z K_{ar}$$ (3) Eq. 2

$$\sigma_{h,t} = q_t K_{ar}$$ (3) Eq. 54

Note that $\sigma_{hbrdge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

FINAL DESIGN - GRS-IBS WINGWALL - ABUTMENT 2 - NEWW - STEP 1 VTRANS D37 IM 091-1(68)

Service Limit State

z	$\sigma_{h,W}$	$\sigma_{h,t}$	σ_h	T_{req}	$T_{@\epsilon = 2\%}$	
0.67 ft	0.02 ksf	0.00 ksf	0.016 ksf	0.03 klf	1.5 klf	OK
1.33 ft	0.03 ksf	0.00 ksf	0.033 ksf	0.06 klf	1.5 klf	OK
2.00 ft	0.05 ksf	0.00 ksf	0.049 ksf	0.09 klf	1.5 klf	OK
2.67 ft	0.07 ksf	0.00 ksf	0.066 ksf	0.11 klf	1.5 klf	OK
3.33 ft	0.08 ksf	0.00 ksf	0.082 ksf	0.14 klf	1.5 klf	OK
4.00 ft	0.10 ksf	0.00 ksf	0.099 ksf	0.17 klf	1.5 klf	OK
4.67 ft	0.12 ksf	0.00 ksf	0.115 ksf	0.20 klf	1.5 klf	OK
5.33 ft	0.13 ksf	0.00 ksf	0.132 ksf	0.23 klf	1.5 klf	OK
6.00 ft	0.15 ksf	0.00 ksf	0.148 ksf	0.26 klf	1.5 klf	OK
6.67 ft	0.16 ksf	0.00 ksf	0.165 ksf	0.28 klf	1.5 klf	OK
7.33 ft	0.18 ksf	0.00 ksf	0.181 ksf	0.31 klf	1.5 klf	OK
8.00 ft	0.20 ksf	0.00 ksf	0.198 ksf	0.34 klf	1.5 klf	OK
8.67 ft	0.21 ksf	0.00 ksf	0.214 ksf	0.37 klf	1.5 klf	OK
9.33 ft	0.23 ksf	0.00 ksf	0.231 ksf	0.40 klf	1.5 klf	OK
10.00 ft	0.25 ksf	0.00 ksf	0.247 ksf	0.43 klf	1.5 klf	OK
10.67 ft	0.26 ksf	0.00 ksf	0.264 ksf	0.46 klf	1.5 klf	OK
11.33 ft	0.28 ksf	0.00 ksf	0.280 ksf	0.48 klf	1.5 klf	OK
12.00 ft	0.30 ksf	0.00 ksf	0.297 ksf	0.51 klf	1.5 klf	OK
12.67 ft	0.31 ksf	0.00 ksf	0.313 ksf	0.54 klf	1.5 klf	OK
13.33 ft	0.33 ksf	0.00 ksf	0.330 ksf	0.57 klf	1.5 klf	OK
14.00 ft	0.35 ksf	0.00 ksf	0.346 ksf	0.60 klf	1.5 klf	OK
14.67 ft	0.36 ksf	0.00 ksf	0.363 ksf	0.63 klf	1.5 klf	OK
15.33 ft	0.38 ksf	0.00 ksf	0.379 ksf	0.65 klf	1.5 klf	OK
16.00 ft	0.40 ksf	0.00 ksf	0.396 ksf	0.68 klf	1.5 klf	OK
16.67 ft	0.41 ksf	0.00 ksf	0.412 ksf	0.71 klf	1.5 klf	OK
17.33 ft	0.43 ksf	0.00 ksf	0.429 ksf	0.74 klf	1.5 klf	OK
18.00 ft	0.45 ksf	0.00 ksf	0.445 ksf	0.77 klf	1.5 klf	OK
18.67 ft	0.46 ksf	0.00 ksf	0.462 ksf	0.80 klf	1.5 klf	OK
19.33 ft	0.48 ksf	0.00 ksf	0.478 ksf	0.83 klf	1.5 klf	OK
19.71 ft	0.49 ksf	0.00 ksf	0.487 ksf	0.84 klf	1.5 klf	OK

FINAL DESIGN - GRS-IBS WINGWALL - ABUTMENT 2 - SEWW - STEP 1 | VTRANS D37 IM 091-1(68)

References:

1) GRS-IBS Interim Implementation Guide, FHWA, June 2018
(2) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018
(3) Geotechnical Data Report, Teracon Consultants Inc., March 27. 2019

Material Properties

γ_f =	115.00 pcf	(Reinforced Fill)
ϕ (Reinforced Soil) =	45	degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β =	26.57	degrees

K_a (Reinforced Soil) = 0.22 = 0.894 x $\dfrac{0.894 - (0.800 - 0.5)\verb|^| 0.5}{0.894 + (0.800 - 0.5)\verb|^| 0.5}$

Fill retained is in-situ soil.

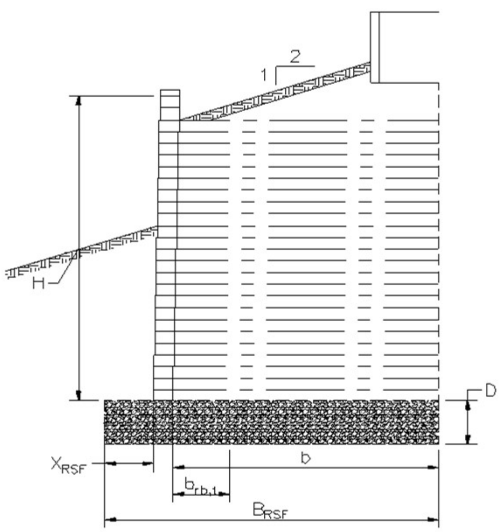
γ_b =	125.00 pcf	(Retained Fill)
ϕ (Existing Filll) =	34.00	degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β =	26.57	degrees

K_a (Retained Fill) = 0.41 = 0.894 x $\dfrac{0.894 - (0.800 - 0.6873)\verb|^| 0.5}{0.894 + (0.800 - 0.6873)\verb|^| 0.5}$

Geometry



Depth of RSF, D =	2.5 ft		
Top of RSF EL. =	458.96		
Bottom of RSF EL. =	456.46 =	458.96 -	2.5 ft
Top of Wall EL. =	472.41		
Wall Height =	13.45 ft	472.41 -	458.96
H =	13.45 ft		
B =	6.75 ft		
Toe Length, X_{RSF} =	2.5 ft		
Total Width, B_{RSF} =	10.25 ft		
$b_{rb,t}$ =	0.00 ft		
Stepped Footing Length =	11.75 ft		
h_{block} =	8.00 in		
D_{block} =	12.00 in		
L_{block} =	18.00 in		
Weight =	85 lbs	per block	

Reinforcement

S_v =	0.67 ft
d_{max} =	0.75 in
T_f =	5.90 klf

Loading

Note that the superstructure dead loads, roadway bed loads, and traffic live loads are not applicable for this structure. No live load surcharge is applied.

Load Factors

Per (3) Table 3.4.1-1 and Table 3.4.1-2.

γ_p (DC, Min.) =	0.90	3) Table 3.4.1-2
γ_p (DC, Max.) =	1.25	3) Table 3.4.1-2
γ_p (EH, Max) =	1.50	3) Table 3.4.1-2
γ_p (EV, Min.) =	1.00	3) Table 3.4.1-2
γ_p (EV, Max.) =	1.35	3) Table 3.4.1-2
Factor (LS) =	1.75	3) Table 3.4.1-1

Calculate Vertical Loads and Applied Pressures

Calculate Traffic Surcharge, q_T

Due to wingwall layout geometry, assume that there is no live load surcharge acting on the wingwall.

h_{eq} = 0.00 ft

γ_r= 125.00 pcf

$$q_t = h_{eq}\gamma_b$$

q_T = 0.00 psf = 0.00 ft x 125.00 pcf

Calculate Weight of GRS Abutment

$$W = BH\gamma_r$$

H = 13.45 ft

B = 6.75 ft

γ_r= 115.00 pcf

W = 10439.54 plf = 13.44868 ft x 6.75 ft x 115.00 pcf

Calculate Weight of RSF

$$W_{RSF} = B_{RSF} D_{RSF} \gamma_r$$

B = 10.25 ft

D = 2.5 ft

γ_r= 125.00 pcf

W_{RSF} = 2946.88 plf = 10.25 ft x 2.5 ft x 115.00 pcf

Calculate Weight of Facing

N_{block} = 20.17 = 13.44868 ft/ 0.67 ft

N_{block} = 21

$$W_{face} = N_{block}\frac{W_{block}}{L_{block}}$$

W_{face} = 1190.00 plf = 21 x (85 lbs/ (18.00 in/ 12)

Calculate Weight of Surplus Retained Fill

Fill is retained at a 2H:1V slope

B = 6.75 ft
H_{max} = 3.38 ft = 6.75 ft/ 2
W_{SF} = 1423.83 plf = 0.5 x 6.75 ft x 3.38 ft x 125.00 pcf

Summary of Applied Loads

q_t = 0.00 psf
W = 10439.54 plf
W_{RSF} = 2946.88 plf
W_{face} = 1190.00 plf
W_{SF} = 1423.83 plf

Calculte Direct Sliding Effects at RSF/GRS Interface

Calculate Sliding per the (1) and (3) Strength I Load Combination

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2$$ (1) Eq. 9

$$F_t = q_t K_{ab} H$$ (1) Eq. 11

$$F_R = \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t$$ (1) Eq. 12

F_b = 4592.30 lbs = 0.5 x 125.00 pcf x 0.41 x 13.44868 ft^ 2
F_t = 0.00 lbs = 0.00 psf 0.41 x 13.4486799 ft

F_R = 6.89 klf = (1.5 x 4592.30 lbs) + (1.75 x 0.00 lbs))/ 1000

$$W_{T,R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EV\,MIN}(q_{rb} b_{rb,t}) + \gamma_{EV\,MIN} W_{SF}$$ (1) Eq. 14

$W_{T,R}$ = 12.93 klf = ((1.0 x 10439.54 plf) + ((0.9 x 1190.00 plf))/ 1000) + (1.00 x 1.42 klf)

Assume that μ = 2/3*tanϕ, per (1) C.1.6.1.1

μ = 0.67 = 0.67 x tan(45)

$$R_R = \Phi_\tau (W_{T,R}\mu)$$ (1) Eq. 13

R_R = 8.62 klf = 12.934 klf x 0.667

OK

Calculate Direct Sliding at Base of RSF

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2$$ (3) Eq. 17

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF})$$ (3) Eq. 19

$F_{b,\,RSF}$ =	6.46 klf =	(0.5 x	125.00 pcf x	0.41 x	(13.45 ft +	2.5)^ 2)/	1000
$F_{t,\,RSF}$ =	0.00 klf =	0.00 psf x	0.41 x	(13.45 ft +	2.5 ft))/	1000	

$$F_{R,RSF} = \gamma_{EH\,MAX}(F_{b,RSF} + F_{rb,RSF}) + \gamma_{LS} F_{t,RSF}$$ (1) Eq. 20

$F_{R,RSF}$ = 9.69 klf = (1.5 x x 6.46 klf) + (1.75 x 0.00 klf)

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\,MIN} W_{RSF}$$ (1) Eq. 22

$W_{T,R,RSF}$ = 15.88 klf = 12.93 klf + (1.0 x 2.95 klf)

$$R_{R,RSF} = \Phi_\tau(W_{T,R,RSF}\,\mu_{RSF})$$ (1) Eq. 21

Note that ϕ = 1.0, per (1) 4.3.6.1.2
Per (2), and the previous calculation, the angle of internal friction beneath the RSF is equal to 32 degrees

$R_{R,\,RSF}$ = 9.92 klf = 1.0 x 15.88 klf x tan(32)

OK

Calculate External Bearing Resistance

Calculate V_R

Per (1) Eq. 26,

$$\sum V_R = \gamma_{EV\,MAX}(W) + \gamma_{EV\,MAX}(W_{RSF}) + \gamma_{DC\,MAX}(W_{face}) + \gamma_{LS}(q_t b_{rb,t}) + \gamma_{EH\,MAX}(q_{rb} b_{rb,t}) + \gamma_{DC\,MAX}(q_{DL} b) + \gamma_{LS}(q_{LL} b)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\,MIN}$

W =	14.09 klf =	1.35 x	10439.54 plf/	1000	
W_{RSF} =	3.98 klf =	1.35 x	2946.88 plf/	1000	
W_{FACE} =	1.49 klf =	1.25 x	1190.00 plf/	1000	
W_{SF} =	1.92 klf =	1.35 x	1423.83 plf/	1000	
Q_t =	0.00 klf =	1.75 x	(0.00 psf x	0.00 ft)/	1000
Total =	21.48 klf				

Calculate $M_{D,R}$

Per (1) Eq. 28

$$\sum M_{D,R} = \gamma_{EH\,MAX} F_{b,RSF}\left(\frac{H + D_{RSF}}{3}\right) + \gamma_{LS} F_{t,RSF}\left(\frac{H + D_{RSF}}{2}\right) + \gamma_{EH\,MAX} F_{rb,RSF}\left(\frac{H + D_{RSF}}{2}\right)$$

$F_{b,\,RSF}$ =	51.50 k-ft/ ft =	1.5 x	6.46 klf x	(0.33 x	(13.4487 ft +	2.5 ft))
$F_{t,\,RSF}$ =	0.00 k-ft/ ft =	1.75 x	0.00 klf x	(0.50 x	(13.4487 ft +	2.5 ft))
Total =	51.50 k-ft/ ft					

Note that $M_{D,R}$ is taken about the bottom center of the width of the RSF.

Calculate $M_{R,R}$

Per (1) Eq. 29

$$\sum M_{R,R} = (\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b)\left[\left(\frac{b}{2} + a_b\right) - \left(\frac{B_{RSF}}{2} - x_{RSF} - b_{block}\right)\right] + \left(\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t}\right)\left(\frac{B_{RSF}}{2} - \frac{b_{rb}}{2}\right) + \gamma_{EV\,MAX} W\left(\frac{B_{RSF}}{2} - \frac{B}{2}\right) + \gamma_{DC\,MAX} W_{face}\left(B + \frac{b_{block}}{2} - \frac{B_{RSF}}{2}\right)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\ MAX}$. Resisting component = W_{SF}*((X_{RSF}+D_{block}+(2/3)*B)-BRSF/2)

W =	24.66 k-ft =	1.35 x	10.44 klf x	(5.125 ft -	3.375 ft)				
W_{Face} =	-3.41 k-ft =	1.35 x	1.19 klf x	((2.5 ft +	0.5 ft) -	5.125 ft)			
W_{SF} =	5.53 k-ft =	1.35 x	1.42 klf x	((2.5 ft +	1 ft +		(0.67 x	6.75 ft)) -	(10.25 ft/ 2))
Total =	26.78 k-ft/ ft								

Note that $M_{R,R}$ is taken about the bottom center of the width of the RSF.

Calculate Eccentricity, $e_{B,R}$

Per (1) Eq. 27

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R}$$

$e_{B,R}$ = 1.15 ft = (51.50 k-ft/ ft - 26.78 k-ft/ ft) / 21.48 klf

Calculate $\sigma_{v,base,R}$

Since $e_{B,R}$ = negative, set equal to zero in calcaulation of base bearing pressure.

Per (1) Eq. 25,

$$\sigma_{v,base,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}}$$

$\sigma_{v,base,R}$ = 2.70 ksf = 21.48 klf/ (10.25 ft - (2 x 1.15 ft)

Per abutment bearing capacity calculation, allowable bearing capacity = 11.02 ksf

OK

Calculate Reinforcement Strength

q_t = 0.00 psf
W = 10439.54 psf
W_{RSF} = 2946.88 psf
W_{face} = 1190.00 psf

The evaulation of the abutment for the strength limit state is conducted according to (1) 4.3.7.3.1. and using the following equations:

Required Reinforcment:

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 40

Factored Total Lateral Pressure:

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f}$$ (3) Eq. 41

Where:

$$\sigma_{h,W,f} = \gamma_{EH\,MAX}(\gamma_r z K_{ar})$$

Where:
$\gamma_{EH\,MAX}$ = maximum horizontal earth pressure load factor.
γ_r = unit weight of reinforced backfill.
z = depth from the top of the wall.
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

(3) Eq. 42

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar}$$ (3) Eq. 45

Note that $\sigma_{hbridge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Factored Reinforcement Strength:

$$T_{f,f} = \Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right) = 0.4T_f$$ (3) Eq. 48

Strength Limit State

z	$\sigma_{h,W,f}$	$\sigma_{h,t,f}$	$\sigma_{h,f}$	$T_{req,f}$	$T_{f,f}$	
0.67 ft	0.02 ksf	0.00 ksf	0.02 ksf	0.03 ksf	2.36 klf	OK
1.33 ft	0.05 ksf	0.00 ksf	0.05 ksf	0.06 ksf	2.36 klf	OK
2.00 ft	0.07 ksf	0.00 ksf	0.07 ksf	0.09 ksf	2.36 klf	OK
2.67 ft	0.10 ksf	0.00 ksf	0.10 ksf	0.12 ksf	2.36 klf	OK
3.33 ft	0.12 ksf	0.00 ksf	0.12 ksf	0.16 ksf	2.36 klf	OK
4.00 ft	0.15 ksf	0.00 ksf	0.15 ksf	0.19 ksf	2.36 klf	OK
4.67 ft	0.17 ksf	0.00 ksf	0.17 ksf	0.22 ksf	2.36 klf	OK
5.33 ft	0.20 ksf	0.00 ksf	0.20 ksf	0.25 ksf	2.36 klf	OK
6.00 ft	0.22 ksf	0.00 ksf	0.22 ksf	0.28 ksf	2.36 klf	OK
6.67 ft	0.25 ksf	0.00 ksf	0.25 ksf	0.31 ksf	2.36 klf	OK
7.33 ft	0.27 ksf	0.00 ksf	0.27 ksf	0.34 ksf	2.36 klf	OK
8.00 ft	0.30 ksf	0.00 ksf	0.30 ksf	0.37 ksf	2.36 klf	OK
8.67 ft	0.32 ksf	0.00 ksf	0.32 ksf	0.40 ksf	2.36 klf	OK
9.33 ft	0.35 ksf	0.00 ksf	0.35 ksf	0.44 ksf	2.36 klf	OK
10.00 ft	0.37 ksf	0.00 ksf	0.37 ksf	0.47 ksf	2.36 klf	OK
10.67 ft	0.40 ksf	0.00 ksf	0.40 ksf	0.50 ksf	2.36 klf	OK
11.33 ft	0.42 ksf	0.00 ksf	0.42 ksf	0.53 ksf	2.36 klf	OK
12.00 ft	0.45 ksf	0.00 ksf	0.45 ksf	0.56 ksf	2.36 klf	OK
12.67 ft	0.47 ksf	0.00 ksf	0.47 ksf	0.59 ksf	2.36 klf	OK
13.45 ft	0.50 ksf	0.00 ksf	0.50 ksf	0.63 ksf	2.36 klf	OK

The evaulation of the abutment for the service limit state is conducted according to (1) 4.3.7.3.2. and using the following equations:

The nominal required reinforcement strength:

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 50

The lateral pressure within the GRS Abutment at a given dpeth and location:

$$\sigma_h = \sigma_{h,W} + \sigma_{h,bridge,eq} + \sigma_{h,rb} + \sigma_{h,t}$$ (3) Eq. 51

Where:

$$\sigma_{h,W} = \gamma_r z K_{ar}$$ (3) Eq. 2

$$\sigma_{h,t} = q_t K_{ar}$$ (3) Eq. 54

Note that $\sigma_{hbridge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Service Limit State

z	$\sigma_{h,W}$	$\sigma_{h,t}$	σ_h	T_{req}	$T_{@\varepsilon = 2\%}$	
0.67 ft	0.02 ksf	0.00 ksf	0.016 ksf	0.02 klf	1.25 klf	OK
1.33 ft	0.03 ksf	0.00 ksf	0.033 ksf	0.04 klf	1.25 klf	OK
2.00 ft	0.05 ksf	0.00 ksf	0.049 ksf	0.06 klf	1.25 klf	OK
2.67 ft	0.07 ksf	0.00 ksf	0.066 ksf	0.08 klf	1.25 klf	OK
3.33 ft	0.08 ksf	0.00 ksf	0.082 ksf	0.10 klf	1.25 klf	OK
4.00 ft	0.10 ksf	0.00 ksf	0.099 ksf	0.12 klf	1.25 klf	OK
4.67 ft	0.12 ksf	0.00 ksf	0.115 ksf	0.15 klf	1.25 klf	OK
5.33 ft	0.13 ksf	0.00 ksf	0.132 ksf	0.17 klf	1.25 klf	OK
6.00 ft	0.15 ksf	0.00 ksf	0.148 ksf	0.19 klf	1.25 klf	OK
6.67 ft	0.16 ksf	0.00 ksf	0.165 ksf	0.21 klf	1.25 klf	OK
7.33 ft	0.18 ksf	0.00 ksf	0.181 ksf	0.23 klf	1.25 klf	OK
8.00 ft	0.20 ksf	0.00 ksf	0.198 ksf	0.25 klf	1.25 klf	OK
8.67 ft	0.21 ksf	0.00 ksf	0.214 ksf	0.27 klf	1.25 klf	OK
9.33 ft	0.23 ksf	0.00 ksf	0.231 ksf	0.29 klf	1.25 klf	OK
10.00 ft	0.25 ksf	0.00 ksf	0.247 ksf	0.31 klf	1.25 klf	OK
10.67 ft	0.26 ksf	0.00 ksf	0.264 ksf	0.33 klf	1.25 klf	OK
11.33 ft	0.28 ksf	0.00 ksf	0.280 ksf	0.35 klf	1.25 klf	OK
12.00 ft	0.30 ksf	0.00 ksf	0.297 ksf	0.37 klf	1.25 klf	OK
12.67 ft	0.31 ksf	0.00 ksf	0.313 ksf	0.39 klf	1.25 klf	OK
13.45 ft	0.33 ksf	0.00 ksf	0.333 ksf	0.42 klf	1.25 klf	OK

FINAL DESIGN - GRS-IBS WINGWALL - ABUTMENT 2 - SEWW - STEP 2 — VTRANS D37 IM 091-1(68)

References:

1) GRS-IBS Interim Implementation Guide, FHWA, June 2018
(2) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018
(3) Geotechnical Data Report, Teracon Consultants Inc., March 27. 2019

Material Properties

γ_f = 115.00 pcf (Reinforced Fill)
ϕ (Reinforced Soil) = 45 degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β = 26.57 degrees

K_a (Reinforced Soil) = 0.22 = 0.894 x (0.894 - (0.800 - 0.5)^ 0.5) / (0.894 + (0.800 - 0.5)^ 0.5)

Fill retained is in-situ soil.

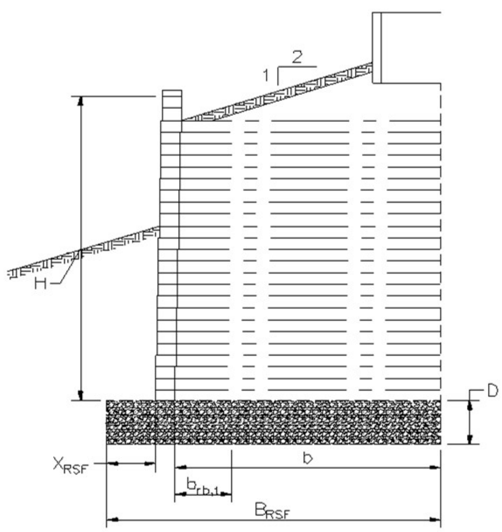
γ_b = 125.00 pcf (Retained Fill)
ϕ (Existing Filll) = 34.00 degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β = 26.57 degrees

K_a (Retained Fill) = 0.41 = 0.894 x (0.894 - (0.800 - 0.6873)^ 0.5) / (0.894 + (0.800 - 0.6873)^ 0.5)

Geometry



Depth of RSF, D = 2.5 ft
Top of RSF EL. = 456.46
Bottom of RSF EL. = 453.96 = 456.46 - 2.5 ft

Top of Wall EL. = 476.27
Wall Height = 19.81 ft = 476.27 - 456.46
H = 19.81 ft

B = 9.75 ft
Toe Length, X_{RSF} = 2.5 ft
Total Width, B_{RSF} = 13.25 ft
$b_{rb,t}$ = 0.00 ft
Stepped Footing Length = 11.75 ft

h_{block} = 8.00 in
D_{block} = 12.00 in
L_{block} = 18.00 in
Weight = 85 lbs per block

Reinforcement

S_v = 0.67 ft
d_{max} = 0.75 in
T_f = 5.90 klf

Loading
Note that the superstructure dead loads, roadway bed loads, and traffic live loads are not applicable for this structure. No live load surcharge is applied.

Load Factors
Per (3) Table 3.4.1-1 and Table 3.4.1-2.

γ_p (DC, Min.) =	0.90	3) Table 3.4.1-2
γ_p (DC, Max.) =	1.25	3) Table 3.4.1-2
γ_p (EH, Max) =	1.50	3) Table 3.4.1-2
γ_p (EV, Min.) =	1.00	3) Table 3.4.1-2
γ_p (EV, Max.) =	1.35	3) Table 3.4.1-2
Factor (LS) =	1.75	3) Table 3.4.1-1

Calculate Vertical Loads and Applied Pressures
Calculate Traffic Surcharge, q_T

Due to wingwall layout geometry, assume that there is no live load surcharge acting on the wingwall.

h_{eq} = 0.00 ft
γ_r= 125.00 pcf

$$q_t = h_{eq}\gamma_b$$

q_T = 0.00 psf = 0.00 ft x 125.00 pcf

Calculate Weight of GRS Abutment

$$W = BH\gamma_r$$

H = 19.81 ft
B = 9.75 ft
γ_r= 115.00 pcf

W = 22213.36 plf = 19.81125 ft x 9.75 ft x 115.00 pcf

Calculate Weight of RSF

$$W_{RSF} = B_{RSF}D_{RSF}\gamma_r$$

B = 13.25 ft
D = 2.5 ft
γ_r= 125.00 pcf

W_{RSF} = 3809.38 plf = 13.25 ft x 2.5 ft x 115.00 pcf

Calculate Weight of Facing

N_{block} = 29.72 = 19.811248 ft/ 0.67 ft
N_{block} = 30

$$W_{face} = N_{block}\frac{W_{block}}{L_{block}}$$

W_{face} = 1700.00 plf = 30 x (85 lbs/ (18.00 in/ 12)

Calculate Weight of Surplus Retained Fill

Fill is retained at a 2H:1V slope

B = 9.75 ft
H_{max} = 4.88 ft = 9.75 ft/ 2
W_{SF} = 2970.70 plf = 0.5 x 9.75 ft x 4.88 ft x 125.00 pcf

Summary of Applied Loads

q_t = 0.00 psf
W = 22213.36 plf
W_{RSF} = 3809.38 plf
W_{face} = 1700.00 plf
W_{SF} = 2970.70 plf

Calculte Direct Sliding Effects at RSF/GRS Interface

Calculate Sliding per the (1) and (3) Strength I Load Combination

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2$$ (1) Eq. 9

$$F_t = q_t K_{ab} H$$ (1) Eq. 11

$$F_R = \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t$$ (1) Eq. 12

F_b = 9965.40 lbs = 0.5 x 125.00 pcf x 0.41 x 19.81125 ft^ 2
F_t = 0.00 lbs = 0.00 psf 0.41 x 19.8112477 ft

F_R = 14.95 klf = (1.5 x 9965.40 lbs) + (1.75 x 0.00 lbs))/ 1000

$$W_{T,R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EV\,MIN}(q_{rb} b_{rb,t}) + \gamma_{EV\,MIN} W_{SF}$$ (1) Eq. 14

$W_{T,R}$ = 26.71 klf = ((1.0 x 22213.36 plf) + ((0.9 x 1700.00 plf))/ 1000) + (1.00 x 2.97 klf)

Assume that μ = 2/3*tanϕ, per (1) C.1.6.1.1

μ = 0.67 = 0.67 x tan(45)

$$R_R = \Phi_\tau (W_{T,R}\mu)$$ (1) Eq. 13

R_R = 17.81 klf = 26.714 klf x 0.667

OK

Calculate Direct Sliding at Base of RSF

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2$$ (3) Eq. 17

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF})$$ (3) Eq. 19

$F_{b,\ RSF}$ =	12.64 klf =	(0.5 x	125.00 pcf x	0.41 x	(19.81 ft +	2.5)^ 2)/	1000
$F_{t,\ RSF}$ =	0.00 klf =	0.00 psf x	0.41 x	(19.81 ft +	2.5 ft))/	1000	

$$F_{R,RSF} = \gamma_{EH\ MAX}(F_{b,RSF} + F_{rb,RSF}) + \gamma_{LS} F_{t,RSF}$$ (1) Eq. 20

$F_{R,RSF}$ = 18.96 klf = (1.5 x x 12.64 klf) + (1.75 x 0.00 klf)

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\ MIN} W_{RSF}$$ (1) Eq. 22

$W_{T,R,RSF}$ = 30.52 klf = 26.71 klf + (1.0 x 3.81 klf)

$$R_{R,RSF} = \Phi_\tau(W_{T,R,RSF}\mu_{RSF})$$ (1) Eq. 21

Note that ϕ = 1.0, per (1) 4.3.6.1.2
Per (2), and the previous calculation, the angle of internal friction beneath the RSF is equal to 32 degrees

$R_{R,\ RSF}$ = 19.07 klf = 1.0 x 30.52 klf x tan(32)

OK

Calculate External Bearing Resistance

Calculate V_R

Per (1) Eq. 26,

$$\sum V_R = \gamma_{EV\ MAX}(W) + \gamma_{EV\ MAX}(W_{RSF}) + \gamma_{DC\ MAX}(W_{face}) + \gamma_{LS}(q_t b_{rb,t}) + \gamma_{EH\ MAX}(q_{rb} b_{rb,t}) + \gamma_{DC\ MAX}(q_{DL} b) + \gamma_{LS}(q_{LL} b)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\ MIN}$

W =	29.99 klf =	1.35 x	22213.36 plf/	1000	
W_{RSF} =	5.14 klf =	1.35 x	3809.38 plf/	1000	
W_{FACE} =	2.13 klf =	1.25 x	1700.00 plf/	1000	
W_{SF} =	4.01 klf =	1.35 x	2970.70 plf/	1000	
Q_t =	0.00 klf =	1.75 x	(0.00 psf x	0.00 ft)/	1000
Total =	41.27 klf				

Calculate $M_{D,R}$

Per (1) Eq. 28

$$\sum M_{D,R} = \gamma_{EH\ MAX} F_{b,RSF}\left(\frac{H + D_{RSF}}{3}\right) + \gamma_{LS} F_{t,RSF}\left(\frac{H + D_{RSF}}{2}\right) + \gamma_{EH\ MAX} F_{rb,RSF}\left(\frac{H + D_{RSF}}{2}\right)$$

$F_{b,\ RSF}$ =	141.00 k-ft/ ft =	1.5 x	12.64 klf x	(0.33 x	(19.8112 ft +	2.5 ft))
$F_{t,\ RSF}$ =	0.00 k-ft/ ft =	1.75 x	0.00 klf x	(0.50 x	(19.8112 ft +	2.5 ft))
Total =	141.00 k-ft/ ft					

Note that $M_{D,R}$ is taken about the bottom center of the width of the RSF.

Calculate $M_{R,R}$

Per (1) Eq. 29

$$\sum M_{R,R} = \left(\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b\right)\left[\left(\frac{b}{2} + a_b\right) - \left(\frac{B_{RSF}}{2} - x_{RSF} - b_{block}\right)\right] + \left(\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t}\right)\left(\frac{B_{RSF}}{2} - \frac{b_{rb}}{2}\right) + \gamma_{EV\,MAX} W\left(\frac{B_{RSF}}{2} - \frac{B}{2}\right) + \gamma_{DC\,MAX} W_{face}\left(B + \frac{b_{block}}{2} - \frac{B_{RSF}}{2}\right)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\,MAX}$. Resisting component = W_{SF}*((X_{RSF}+D_{block}+(2/3)*B)-BRSF/2)

W =	52.48 k-ft =	1.35 x	22.21 klf x	(6.625 ft -	4.875 ft)				
W_{Face} =	-8.32 k-ft =	1.35 x	1.70 klf x	((2.5 ft +	0.5 ft) -	6.625 ft)			
W_{SF} =	13.54 k-ft =	1.35 x	2.97 klf x	((2.5 ft +	1 ft +		(0.67 x	9.75 ft)) -	(13.25 ft/ 2))
Total =	57.69 k-ft/ ft								

Note that $M_{R,R}$ is taken about the bottom center of the width of the RSF.

Calculate Eccentricity, $e_{B,R}$

Per (1) Eq. 27

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R}$$

$e_{B,R}$ = 2.02 ft = $\frac{141.00 \text{ k-ft/ ft} - 57.69 \text{ k-ft/ ft}}{41.27 \text{ klf}}$

Calculate $\sigma_{v,base,R}$

Since $e_{B,R}$ = negative, set equal to zero in calcaulation of base bearing pressure.

Per (1) Eq. 25,

$$\sigma_{v,base,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}}$$

$\sigma_{v,base,R}$ = 4.48 ksf = 41.27 klf/ (13.25 ft - (2 x 2.02 ft)

Per abutment bearing capacity calculation, allowable bearing capacity = 11.02 ksf

OK

Calculate Reinforcement Strength

q_t = 0.00 psf
W = 22213.36 psf
W_{RSF} = 3809.38 psf
W_{face} = 1700.00 psf

The evaulation of the abutment for the strength limit state is conducted according to (1) 4.3.7.3.1. and using the following equations:

Required Reinforcment:

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$

(3) Eq. 40

Factored Total Lateral Pressure:

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f}$$

(3) Eq. 41

Where:

$$\sigma_{h,W,f} = \gamma_{EH\,MAX}(\gamma_r z K_{ar})$$

Where:
$\gamma_{EH\,MAX}$ = maximum horizontal earth pressure load factor.
γ_r = unit weight of reinforced backfill.
z = depth from the top of the wall.
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

(3) Eq. 42

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar}$$

(3) Eq. 45

Note that $\sigma_{hbridge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Factored Reinforcement Strength:

$$T_{f,f} = \Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right) = 0.4T_f$$

(3) Eq. 48

Strength Limit State

z	$\sigma_{h,W,f}$	$\sigma_{h,t,f}$	$\sigma_{h,f}$	$T_{req,f}$	$T_{f,f}$	
0.67 ft	0.02 ksf	0.00 ksf	0.02 ksf	0.03 ksf	2.36 klf	OK
1.33 ft	0.05 ksf	0.00 ksf	0.05 ksf	0.06 ksf	2.36 klf	OK
2.00 ft	0.07 ksf	0.00 ksf	0.07 ksf	0.09 ksf	2.36 klf	OK
2.67 ft	0.10 ksf	0.00 ksf	0.10 ksf	0.12 ksf	2.36 klf	OK
3.33 ft	0.12 ksf	0.00 ksf	0.12 ksf	0.16 ksf	2.36 klf	OK
4.00 ft	0.15 ksf	0.00 ksf	0.15 ksf	0.19 ksf	2.36 klf	OK
4.67 ft	0.17 ksf	0.00 ksf	0.17 ksf	0.22 ksf	2.36 klf	OK
5.33 ft	0.20 ksf	0.00 ksf	0.20 ksf	0.25 ksf	2.36 klf	OK
6.00 ft	0.22 ksf	0.00 ksf	0.22 ksf	0.28 ksf	2.36 klf	OK
6.67 ft	0.25 ksf	0.00 ksf	0.25 ksf	0.31 ksf	2.36 klf	OK
7.33 ft	0.27 ksf	0.00 ksf	0.27 ksf	0.34 ksf	2.36 klf	OK
8.00 ft	0.30 ksf	0.00 ksf	0.30 ksf	0.37 ksf	2.36 klf	OK
8.67 ft	0.32 ksf	0.00 ksf	0.32 ksf	0.40 ksf	2.36 klf	OK
9.33 ft	0.35 ksf	0.00 ksf	0.35 ksf	0.44 ksf	2.36 klf	OK
10.00 ft	0.37 ksf	0.00 ksf	0.37 ksf	0.47 ksf	2.36 klf	OK
10.67 ft	0.40 ksf	0.00 ksf	0.40 ksf	0.50 ksf	2.36 klf	OK
11.33 ft	0.42 ksf	0.00 ksf	0.42 ksf	0.53 ksf	2.36 klf	OK
12.00 ft	0.45 ksf	0.00 ksf	0.45 ksf	0.56 ksf	2.36 klf	OK
12.67 ft	0.47 ksf	0.00 ksf	0.47 ksf	0.59 ksf	2.36 klf	OK
13.33 ft	0.49 ksf	0.00 ksf	0.49 ksf	0.62 ksf	2.36 klf	OK
14.00 ft	0.52 ksf	0.00 ksf	0.52 ksf	0.65 ksf	2.36 klf	OK
14.67 ft	0.54 ksf	0.00 ksf	0.54 ksf	0.68 ksf	2.36 klf	OK
15.33 ft	0.57 ksf	0.00 ksf	0.57 ksf	0.71 ksf	2.36 klf	OK
16.00 ft	0.59 ksf	0.00 ksf	0.59 ksf	0.75 ksf	2.36 klf	OK
16.67 ft	0.62 ksf	0.00 ksf	0.62 ksf	0.78 ksf	2.36 klf	OK
17.33 ft	0.64 ksf	0.00 ksf	0.64 ksf	0.81 ksf	2.36 klf	OK
18.00 ft	0.67 ksf	0.00 ksf	0.67 ksf	0.84 ksf	2.36 klf	OK
18.67 ft	0.69 ksf	0.00 ksf	0.69 ksf	0.87 ksf	2.36 klf	OK
19.33 ft	0.72 ksf	0.00 ksf	0.72 ksf	0.90 ksf	2.36 klf	OK
19.81 ft	0.73 ksf	0.00 ksf	0.73 ksf	0.92 ksf	2.36 klf	OK

The evaulation of the abutment for the service limit state is conducted according to (1) 4.3.7.3.2. and using the following equations:

The nominal required reinforcement strength:

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 50

The lateral pressure within the GRS Abutment at a given dpeth and location:

$$\sigma_h = \sigma_{h,W} + \sigma_{h,bridge,eq} + \sigma_{h,rb} + \sigma_{h,t}$$ (3) Eq. 51

Where:

$$\sigma_{h,W} = \gamma_r z K_{ar}$$ (3) Eq. 2

$$\sigma_{h,t} = q_t K_{ar}$$ (3) Eq. 54

Note that $\sigma_{hbridge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Service Limit State

z	$\sigma_{h,W}$	$\sigma_{h,t}$	σ_h	T_{req}	$T_{@\varepsilon = 2\%}$	
0.67 ft	0.02 ksf	0.00 ksf	0.016 ksf	0.02 klf	1.25 klf	OK
1.33 ft	0.03 ksf	0.00 ksf	0.033 ksf	0.04 klf	1.25 klf	OK
2.00 ft	0.05 ksf	0.00 ksf	0.049 ksf	0.06 klf	1.25 klf	OK
2.67 ft	0.07 ksf	0.00 ksf	0.066 ksf	0.08 klf	1.25 klf	OK
3.33 ft	0.08 ksf	0.00 ksf	0.082 ksf	0.10 klf	1.25 klf	OK
4.00 ft	0.10 ksf	0.00 ksf	0.099 ksf	0.12 klf	1.25 klf	OK
4.67 ft	0.12 ksf	0.00 ksf	0.115 ksf	0.15 klf	1.25 klf	OK
5.33 ft	0.13 ksf	0.00 ksf	0.132 ksf	0.17 klf	1.25 klf	OK
6.00 ft	0.15 ksf	0.00 ksf	0.148 ksf	0.19 klf	1.25 klf	OK
6.67 ft	0.16 ksf	0.00 ksf	0.165 ksf	0.21 klf	1.25 klf	OK
7.33 ft	0.18 ksf	0.00 ksf	0.181 ksf	0.23 klf	1.25 klf	OK
8.00 ft	0.20 ksf	0.00 ksf	0.198 ksf	0.25 klf	1.25 klf	OK
8.67 ft	0.21 ksf	0.00 ksf	0.214 ksf	0.27 klf	1.25 klf	OK
9.33 ft	0.23 ksf	0.00 ksf	0.231 ksf	0.29 klf	1.25 klf	OK
10.00 ft	0.25 ksf	0.00 ksf	0.247 ksf	0.31 klf	1.25 klf	OK
10.67 ft	0.26 ksf	0.00 ksf	0.264 ksf	0.33 klf	1.25 klf	OK
11.33 ft	0.28 ksf	0.00 ksf	0.280 ksf	0.35 klf	1.25 klf	OK
12.00 ft	0.30 ksf	0.00 ksf	0.297 ksf	0.37 klf	1.25 klf	OK
12.67 ft	0.31 ksf	0.00 ksf	0.313 ksf	0.39 klf	1.25 klf	OK
13.33 ft	0.33 ksf	0.00 ksf	0.330 ksf	0.41 klf	1.25 klf	OK
14.00 ft	0.35 ksf	0.00 ksf	0.346 ksf	0.44 klf	1.25 klf	OK
14.67 ft	0.36 ksf	0.00 ksf	0.363 ksf	0.46 klf	1.25 klf	OK
15.33 ft	0.38 ksf	0.00 ksf	0.379 ksf	0.48 klf	1.25 klf	OK
16.00 ft	0.40 ksf	0.00 ksf	0.396 ksf	0.50 klf	1.25 klf	OK
16.67 ft	0.41 ksf	0.00 ksf	0.412 ksf	0.52 klf	1.25 klf	OK
17.33 ft	0.43 ksf	0.00 ksf	0.429 ksf	0.54 klf	1.25 klf	OK
18.00 ft	0.45 ksf	0.00 ksf	0.445 ksf	0.56 klf	1.25 klf	OK
18.67 ft	0.46 ksf	0.00 ksf	0.462 ksf	0.58 klf	1.25 klf	OK
19.33 ft	0.48 ksf	0.00 ksf	0.478 ksf	0.60 klf	1.25 klf	OK
19.81 ft	0.49 ksf	0.00 ksf	0.490 ksf	0.62 klf	1.25 klf	OK

FINAL DESIGN - GRS-IBS WINGWALL - ABUTMENT 3 - NWWW - STEP 1 VTRANS D37 IM 091-1(68)

References:

1) GRS-IBS Interim Implementation Guide, FHWA, June 2018
(2) AAASHTO LRFD Bridge Design, 8th Edition with Errata, 2018
(3) Geotechnical Data Report, Teracon Consultants Inc., March 27. 2019

Material Properties

γ_f = 115.00 pcf (Reinforced Fill)
ϕ (Reinforced Soil) = 45 degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β = 26.57 degrees

K_a (Reinforced Soil) = 0.22 = 0.894 x (0.894 - (0.800 - 0.5)^ 0.5) / (0.894 + (0.800 - 0.5)^ 0.5)

Fill retained is in-situ soil.

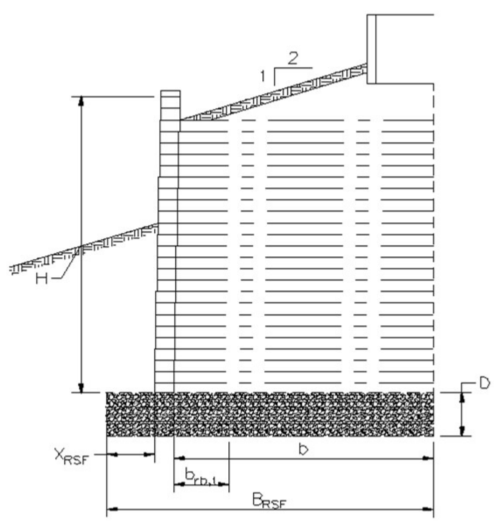
γ_b = 125.00 pcf (Retained Fill)
ϕ (Existing Filll) = 34.00 degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β = 26.57 degrees

K_a (Retained Fill) = 0.41 = 0.894 x (0.894 - (0.800 - 0.6873)^ 0.5) / (0.894 + (0.800 - 0.6873)^ 0.5)

Geometry



Depth of RSF, D = 2.5 ft
Top of RSF EL. = 453.46
Bottom of RSF EL. = 450.96 = 453.46 - 2.5 ft

Top of Wall EL. = 466.46
Wall Height = 13.00 ft 466.46 - 453.46
H = 13.00 ft

B = 6.75 ft
Toe Length, X_{RSF} = 2.5 ft
Total Width, B_{RSF} = 10.25 ft
$b_{rb,t}$ = 0.00 ft
Stepped Footing Length = 14.00 ft

h_{block} = 8.00 in
D_{block} = 12.00 in
L_{block} = 18.00 in
Weight = 85 lbs per block

Reinforcement

S_v =	0.67 ft
d_{max} =	0.75 in
T_f =	5.90 klf

Loading
Note that the superstructure dead loads, roadway bed loads, and traffic live loads are not applicable for this structure. No live load surcharge is applied.

Load Factors
Per (3) Table 3.4.1-1 and Table 3.4.1-2.

γ_p (DC, Min.) =	0.90	3) Table 3.4.1-2
γ_p (DC, Max.) =	1.25	3) Table 3.4.1-2
γ_p (EH, Max) =	1.50	3) Table 3.4.1-2
γ_p (EV, Min.) =	1.00	3) Table 3.4.1-2
γ_p (EV, Max.) =	1.35	3) Table 3.4.1-2
Factor (LS) =	1.75	3) Table 3.4.1-1

Calculate Vertical Loads and Applied Pressures
Calculate Traffic Surcharge, q_T

Due to wingwall layout geometry, assume that there is no live load surcharge acting on the wingwall.

h_{eq} = 0.00 ft
γ_r= 125.00 pcf

$$q_t = h_{eq}\gamma_b$$

q_T = 0.00 psf = 0.00 ft x 125.00 pcf

Calculate Weight of GRS Abutment

$$W = BH\gamma_r$$

H = 13.00 ft
B = 6.75 ft
γ_r= 115.00 pcf

W = 10088.43 plf = 12.99637 ft x 6.75 ft x 115.00 pcf

Calculate Weight of RSF

$$W_{RSF} = B_{RSF}D_{RSF}\gamma_r$$

B = 10.25 ft
D = 2.5 ft
γ_r= 125.00 pcf

W_{RSF} = 2946.88 plf = 10.25 ft x 2.5 ft x 115.00 pcf

Calculate Weight of Facing

N_{block} = 19.49 = 12.996366 ft/ 0.67 ft
N_{block} = 20

$$W_{face} = N_{block}\frac{W_{block}}{L_{block}}$$

W_{face} = 1133.33 plf = 20 x (85 lbs/ (18.00 in/ 12)

Calculate Weight of Surplus Retained Fill

Fill is retained at a 2H:1V slope

B = 6.75 ft
H_{max} = 3.38 ft = 6.75 ft/ 2
W_{SF} = 1423.83 plf = 0.5 x 6.75 ft x 3.38 ft x 125.00 pcf

Summary of Applied Loads

q_t =	0.00	psf
W =	10088.43	plf
W_{RSF} =	2946.88	plf
W_{face} =	1133.33	plf
W_{SF} =	1423.83	plf

Calculte Direct Sliding Effects at RSF/GRS Interface

Calculate Sliding per the (1) and (3) Strength I Load Combination

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2$$ (1) Eq. 9

$$F_t = q_t K_{ab} H$$ (1) Eq. 11

$$F_R = \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t$$ (1) Eq. 12

F_b = 4288.59 lbs = 0.5 x 125.00 pcf x 0.41 x 12.99637 ft^ 2
F_t = 0.00 lbs = 0.00 psf 0.41 x 12.9963659 ft

F_R = 6.43 klf = (1.5 x 4288.59 lbs) + (1.75 x 0.00 lbs))/ 1000

$$W_{T,R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EV\,MIN}(q_{rb} b_{rb,t}) + \gamma_{EV\,MIN} W_{SF}$$ (1) Eq. 14

$W_{T,R}$ = 12.53 klf = ((1.0 x 10088.43 plf) + ((0.9 x 1133.33 plf))/ 1000) + (1.00 x 1.42 klf)

Assume that μ = 2/3*tanϕ, per (1) C.1.6.1.1

μ = 0.67 = 0.67 x tan(45)

$$R_R = \Phi_\tau (W_{T,R}\mu)$$ (1) Eq. 13

R_R = 8.35 klf = 12.532 klf x 0.667

OK

Calculate Direct Sliding at Base of RSF

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2$$ (3) Eq. 17

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF})$$ (3) Eq. 19

$F_{b,\ RSF}$ =	6.10 klf =	(0.5 x	125.00 pcf x	0.41 x	(13.00 ft +	2.5)^ 2)/	1000
$F_{t,\ RSF}$ =	0.00 klf =	0.00 psf x	0.41 x	(13.00 ft +	2.5 ft))/	1000	

$$F_{R,RSF} = \gamma_{EH\ MAX}(F_{b,RSF} + F_{rb,RSF}) + \gamma_{LS}F_{t,RSF}$$ (1) Eq. 20

$F_{R,RSF}$ = 9.15 klf = (1.5 x x 6.10 klf) + (1.75 x 0.00 klf)

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\ MIN}W_{RSF}$$ (1) Eq. 22

$W_{T,R,RSF}$ = 15.48 klf = 12.53 klf + (1.0 x 2.95 klf)

$$R_{R,RSF} = \Phi_\tau(W_{T,R,RSF}\mu_{RSF})$$ (1) Eq. 21

Note that ϕ = 1.0, per (1) 4.3.6.1.2
Per (2), and the previous calculation, the angle of internal friction beneath the RSF is equal to 31 degrees

$R_{R,\ RSF}$ = 9.30 klf = 1.0 x 15.48 klf x tan(31)

OK

Calculate External Bearing Resistance

Calculate V_R

Per (1) Eq. 26,

$$\sum V_R = \gamma_{EV\ MAX}(W) + \gamma_{EV\ MAX}(W_{RSF}) + \gamma_{DC\ MAX}(W_{face}) + \gamma_{LS}(q_t b_{rb,t}) + \gamma_{EH\ MAX}(q_{rb} b_{rb,t}) + \gamma_{DC\ MAX}(q_{DL} b) + \gamma_{LS}(q_{LL} b)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\ MIN}$

W =	13.62 klf =	1.35 x	10088.43 plf/	1000	
W_{RSF} =	3.98 klf =	1.35 x	2946.88 plf/	1000	
W_{FACE} =	1.42 klf =	1.25 x	1133.33 plf/	1000	
W_{SF} =	1.92 klf =	1.35 x	1423.83 plf/	1000	
Q_t =	0.00 klf =	1.75 x	(0.00 psf x	0.00 ft)/	1000
Total =	20.94 klf				

Calculate $M_{D,R}$

Per (1) Eq. 28

$$\sum M_{D,R} = \gamma_{EH\ MAX}F_{b,RSF}\left(\frac{H + D_{RSF}}{3}\right) + \gamma_{LS}F_{t,RSF}\left(\frac{H + D_{RSF}}{2}\right) + \gamma_{EH\ MAX}F_{rb,RSF}\left(\frac{H + D_{RSF}}{2}\right)$$

$F_{b,\ RSF}$ =	47.24 k-ft/ ft =	1.5 x	6.10 klf x	(0.33 x	(12.9964 ft +	2.5 ft))
$F_{t,\ RSF}$ =	0.00 k-ft/ ft =	1.75 x	0.00 klf x	(0.50 x	(12.9964 ft +	2.5 ft))
Total =	47.24 k-ft/ ft					

Note that $M_{D,R}$ is taken about the bottom center of the width of the RSF.

Calculate $M_{R,R}$

Per (1) Eq. 29

$$\sum M_{R,R} = (\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b)\left[\left(\frac{b}{2} + a_b\right) - \left(\frac{B_{RSF}}{2} - x_{RSF} - b_{block}\right)\right] + \left(\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t}\right)\left(\frac{B_{RSF}}{2} - \frac{b_{rb}}{2}\right) + \gamma_{EV\,MAX} W\left(\frac{B_{RSF}}{2} - \frac{B}{2}\right) + \gamma_{DC\,MAX} W_{face}\left(B + \frac{b_{block}}{2} - \frac{B_{RSF}}{2}\right)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\ MAX}$. Resisting component = W_{SF}*((X_{RSF}+D_{block}+(2/3)*B)-BRSF/2)

W =	23.83 k-ft =	1.35 x	10.09 klf x	(5.125 ft -	3.375 ft)				
W_{Face} =	-3.25 k-ft =	1.35 x	1.13 klf x	((2.5 ft +	0.5 ft) -	5.125 ft)			
W_{SF} =	5.53 k-ft =	1.35 x	1.42 klf x	((2.5 ft +	1 ft +		(0.67 x	6.75 ft)) -	(10.25 ft/ 2))
Total =	26.11 k-ft/ ft								

Note that $M_{R,R}$ is taken about the bottom center of the width of the RSF.

Calculate Eccentricity, $e_{B,R}$

Per (1) Eq. 27

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R}$$

$$e_{B,R} = 1.01\text{ ft} = \frac{47.24\text{ k-ft/ ft} - 26.11\text{ k-ft/ ft}}{20.94\text{ klf}}$$

Calculate $\sigma_{v,base,R}$

Since $e_{B,R}$ = negative, set equal to zero in calcaulation of base bearing pressure.

Per (1) Eq. 25,

$$\sigma_{v,base,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}}$$

$\sigma_{v,base,R}$ = 2.54 ksf = 20.94 klf/ (10.25 ft - (2 x 1.01 ft)

Per abutment bearing capacity calculation, allowable bearing capacity = 9.15 ksf

OK

Calculate Reinforcement Strength

q_t =	0.00 psf
W =	10088.43 psf
W_{RSF} =	2946.88 psf
W_{face} =	1133.33 psf

The evaulation of the abutment for the strength limit state is conducted according to (1) 4.3.7.3.1. and using the following equations:

Required Reinforcment:

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}} \right] S_v$$ (3) Eq. 40

Factored Total Lateral Pressure:

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f}$$ (3) Eq. 41

Where:

$$\sigma_{h,W,f} = \gamma_{EH\ MAX}(\gamma_r z K_{ar})$$ (3) Eq. 42

Where:
$\gamma_{EH\ MAX}$ = maximum horizontal earth pressure load factor.
γ_r = unit weight of reinforced backfill.
z = depth from the top of the wall.
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar}$$ (3) Eq. 45

Note that $\sigma_{hbrdge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Factored Reinforcement Strength:

$$T_{f,f} = \Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right) = 0.4T_f$$ (3) Eq. 48

Strength Limit State

z	$\sigma_{h,W,f}$	$\sigma_{h,t,f}$	$\sigma_{h,f}$	$T_{req,f}$	$T_{f,f}$	
0.67 ft	0.02 ksf	0.00 ksf	0.02 ksf	0.03 ksf	2.36 klf	OK
1.33 ft	0.05 ksf	0.00 ksf	0.05 ksf	0.06 ksf	2.36 klf	OK
2.00 ft	0.07 ksf	0.00 ksf	0.07 ksf	0.09 ksf	2.36 klf	OK
2.67 ft	0.10 ksf	0.00 ksf	0.10 ksf	0.12 ksf	2.36 klf	OK
3.33 ft	0.12 ksf	0.00 ksf	0.12 ksf	0.16 ksf	2.36 klf	OK
4.00 ft	0.15 ksf	0.00 ksf	0.15 ksf	0.19 ksf	2.36 klf	OK
4.67 ft	0.17 ksf	0.00 ksf	0.17 ksf	0.22 ksf	2.36 klf	OK
5.33 ft	0.20 ksf	0.00 ksf	0.20 ksf	0.25 ksf	2.36 klf	OK
6.00 ft	0.22 ksf	0.00 ksf	0.22 ksf	0.28 ksf	2.36 klf	OK
6.67 ft	0.25 ksf	0.00 ksf	0.25 ksf	0.31 ksf	2.36 klf	OK
7.33 ft	0.27 ksf	0.00 ksf	0.27 ksf	0.34 ksf	2.36 klf	OK
8.00 ft	0.30 ksf	0.00 ksf	0.30 ksf	0.37 ksf	2.36 klf	OK
8.67 ft	0.32 ksf	0.00 ksf	0.32 ksf	0.40 ksf	2.36 klf	OK
9.33 ft	0.35 ksf	0.00 ksf	0.35 ksf	0.44 ksf	2.36 klf	OK
10.00 ft	0.37 ksf	0.00 ksf	0.37 ksf	0.47 ksf	2.36 klf	OK
10.67 ft	0.40 ksf	0.00 ksf	0.40 ksf	0.50 ksf	2.36 klf	OK
11.33 ft	0.42 ksf	0.00 ksf	0.42 ksf	0.53 ksf	2.36 klf	OK
12.00 ft	0.45 ksf	0.00 ksf	0.45 ksf	0.56 ksf	2.36 klf	OK
13.00 ft	0.48 ksf	0.00 ksf	0.48 ksf	0.61 ksf	2.36 klf	OK

The evaulation of the abutment for the service limit state is conducted according to (1) 4.3.7.3.2. and using the following equations:

The nominal required reinforcement strength:

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}} \right] S_v$$ (3) Eq. 50

The lateral pressure within the GRS Abutment at a given dpeth and location:

$$\sigma_h = \sigma_{h,W} + \sigma_{h,bridge,eq} + \sigma_{h,rb} + \sigma_{h,t}$$ (3) Eq. 51

Where:

$$\sigma_{h,W} = \gamma_r z K_{ar}$$ (3) Eq. 2

$$\sigma_{h,t} = q_t K_{ar}$$ (3) Eq. 54

Note that $\sigma_{h,bridge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Service Limit State

z	$\sigma_{h,W}$	$\sigma_{h,t}$	σ_h	T_{req}	$T_{@\varepsilon = 2\%}$	
0.67 ft	0.02 ksf	0.00 ksf	0.016 ksf	0.02 klf	1.25 klf	OK
1.33 ft	0.03 ksf	0.00 ksf	0.033 ksf	0.04 klf	1.25 klf	OK
2.00 ft	0.05 ksf	0.00 ksf	0.049 ksf	0.06 klf	1.25 klf	OK
2.67 ft	0.07 ksf	0.00 ksf	0.066 ksf	0.08 klf	1.25 klf	OK
3.33 ft	0.08 ksf	0.00 ksf	0.082 ksf	0.10 klf	1.25 klf	OK
4.00 ft	0.10 ksf	0.00 ksf	0.099 ksf	0.12 klf	1.25 klf	OK
4.67 ft	0.12 ksf	0.00 ksf	0.115 ksf	0.15 klf	1.25 klf	OK
5.33 ft	0.13 ksf	0.00 ksf	0.132 ksf	0.17 klf	1.25 klf	OK
6.00 ft	0.15 ksf	0.00 ksf	0.148 ksf	0.19 klf	1.25 klf	OK
6.67 ft	0.16 ksf	0.00 ksf	0.165 ksf	0.21 klf	1.25 klf	OK
7.33 ft	0.18 ksf	0.00 ksf	0.181 ksf	0.23 klf	1.25 klf	OK
8.00 ft	0.20 ksf	0.00 ksf	0.198 ksf	0.25 klf	1.25 klf	OK
8.67 ft	0.21 ksf	0.00 ksf	0.214 ksf	0.27 klf	1.25 klf	OK
9.33 ft	0.23 ksf	0.00 ksf	0.231 ksf	0.29 klf	1.25 klf	OK
10.00 ft	0.25 ksf	0.00 ksf	0.247 ksf	0.31 klf	1.25 klf	OK
10.67 ft	0.26 ksf	0.00 ksf	0.264 ksf	0.33 klf	1.25 klf	OK
11.33 ft	0.28 ksf	0.00 ksf	0.280 ksf	0.35 klf	1.25 klf	OK
12.00 ft	0.30 ksf	0.00 ksf	0.297 ksf	0.37 klf	1.25 klf	OK
13.00 ft	0.32 ksf	0.00 ksf	0.321 ksf	0.40 klf	1.25 klf	OK

FINAL DESIGN - GRS-IBS WINGWALL -ABUTMENT 3 - NWWW - STEP 2 VTRANS D37 IM 091-1(68)

References:

1) GRS-IBS Interim Implementation Guide, FHWA, June 2018
(2) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018
(3) Geotechnical Data Report, Teracon Consultants Inc., March 27. 2019

Material Properties

γ_f =	115.00 pcf	(Reinforced Fill)
ϕ (Reinforced Soil) =	45	degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β =	26.57	degrees

K_a (Reinforced Soil) = 0.22 = 0.894 x (0.894 - (0.800 - 0.5)^ 0.5) / (0.894 + (0.800 - 0.5)^ 0.5)

Fill retained is in-situ soil.

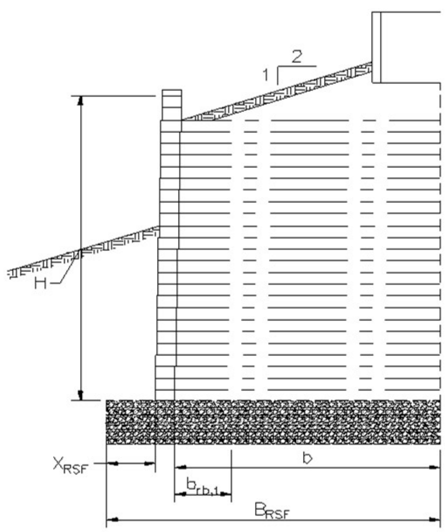
γ_b =	125.00 pcf	(Retained Fill)
ϕ (Existing Filll) =	34.00	degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β =	26.57	degrees

K_a (Retained Fill) = 0.41 = 0.894 x (0.894 - (0.800 - 0.6873)^ 0.5) / (0.894 + (0.800 - 0.6873)^ 0.5)

Geometry



Depth of RSF, D =	2.5 ft		
Top of RSF EL. =	450.96		
Bottom of RSF EL. =	448.46 =	450.96 -	2.5 ft
Top of Wall EL. =	470.27		
Wall Height =	19.31 ft	470.27 -	450.96
H =	19.31 ft		
B =	10 ft		
Toe Length, X_{RSF} =	2.5 ft		
Total Width, B_{RSF} =	13.5 ft		
$b_{rb,t}$ =	0.00 ft		
Stepped Footing Length =	14.00 ft		
h_{block} =	8.00 in		
D_{block} =	12.00 in		
L_{block} =	18.00 in		
Weight =	85 lbs	per block	

Reinforcement

S_v = 0.67 ft
d_{max} = 0.75 in
T_f = 5.90 klf

Loading
Note that the superstructure dead loads, roadway bed loads, and traffic live loads are not applicable for this structure. No live load surcharge is applied.

Load Factors
Per (3) Table 3.4.1-1 and Table 3.4.1-2.

γ_p (DC, Min.) =	0.90	3) Table 3.4.1-2
γ_p (DC, Max.) =	1.25	3) Table 3.4.1-2
γ_p (EH, Max) =	1.50	3) Table 3.4.1-2
γ_p (EV, Min.) =	1.00	3) Table 3.4.1-2
γ_p (EV, Max.) =	1.35	3) Table 3.4.1-2
Factor (LS) =	1.75	3) Table 3.4.1-1

Calculate Vertical Loads and Applied Pressures
Calculate Traffic Surcharge, q_T

Due to wingwall layout geometry, assume that there is no live load surcharge acting on the wingwall.

h_{eq} = 0.00 ft
γ_r= 125.00 pcf

$$q_t = h_{eq}\gamma_b$$

q_T = 0.00 psf = 0.00 ft x 125.00 pcf

Calculate Weight of GRS Abutment

$$W = BH\gamma_r$$

H = 19.31 ft
B = 10 ft
γ_r= 115.00 pcf

W = 22201.34 plf = 19.30551 ft x 10 ft x 115.00 pcf

Calculate Weight of RSF

$$W_{RSF} = B_{RSF}D_{RSF}\gamma_r$$

B = 13.5 ft
D = 2.5 ft
γ_r= 125.00 pcf

W_{RSF} = 3881.25 plf = 13.5 ft x 2.5 ft x 115.00 pcf

Calculate Weight of Facing

N_{block} = 28.96 = 19.305513 ft/ 0.67 ft
N_{block} = 29

$$W_{face} = N_{block}\frac{W_{block}}{L_{block}}$$

W_{face} = 1643.33 plf = 29 x (85 lbs/ (18.00 in/ 12)

FINAL DESIGN - GRS-IBS WINGWALL -ABUTMENT 3 - NWWW - STEP 2 VTRANS D37 IM 091-1(68)

Calculate Weight of Surplus Retained Fill

Fill is retained at a 2H:1V slope

B =	10 ft					
H_{max} =	5.00 ft =	10 ft/ 2				
W_{SF} =	3125.00 plf =	0.5 x	10 ft x	5.00 ft x	125.00 pcf	

Summary of Applied Loads

q_t =	0.00 psf
W =	22201.34 plf
W_{RSF} =	3881.25 plf
W_{face} =	1643.33 plf
W_{SF} =	3125.00 plf

Calculte Direct Sliding Effects at RSF/GRS Interface

Calculate Sliding per the (1) and (3) Strength I Load Combination

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2$$ (1) Eq. 9

$$F_t = q_t K_{ab} H$$ (1) Eq. 11

$$F_R = \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t$$ (1) Eq. 12

F_b = 9463.11 lbs = 0.5 x 125.00 pcf x 0.41 x 19.30551 ft^ 2

F_t = 0.00 lbs = 0.00 psf 0.41 x 19.305513 ft

F_R = 14.19 klf = (1.5 x 9463.11 lbs) + (1.75 x 0.00 lbs))/ 1000

$$W_{T,R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EV\,MIN}(q_{rb} b_{rb,t})$$ + $\gamma_{EV\,MIN} W_{SF}$ (1) Eq. 14

$W_{T,R}$ = 26.81 klf = ((1.0 x 22201.34 plf) + ((0.9 x 1643.33 plf))/ 1000) + (1.00 x 3.13 klf)

Assume that μ = 2/3*tanϕ, per (1) C.1.6.1.1

μ = 0.67 = 0.67 x tan(45)

$$R_R = \Phi_\tau (W_{T,R}\mu)$$ (1) Eq. 13

R_R = 17.87 klf = 26.805 klf x 0.667

OK

Calculate Direct Sliding at Base of RSF

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2$$ (3) Eq. 17

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF})$$ (3) Eq. 19

$F_{b, RSF}$ =	12.07 klf =	(0.5 x	125.00 pcf x	0.41 x	(19.31 ft +	2.5)^ 2)/	1000
$F_{t, RSF}$ =	0.00 klf =	0.00 psf x	0.41 x	(19.31 ft +	2.5 ft))/	1000	

$$F_{R,RSF} = \gamma_{EH\,MAX}\left(F_{b,RSF} + F_{rb,RSF}\right) + \gamma_{LS}F_{t,RSF}$$ (1) Eq. 20

$F_{R,RSF}$ = 18.11 klf = (1.5 x x 12.07 klf) + (1.75 x 0.00 klf)

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\,MIN}W_{RSF}$$ (1) Eq. 22

$W_{T,R,RSF}$ = 30.69 klf = 26.81 klf + (1.0 x 3.88 klf)

$$R_{R,RSF} = \Phi_\tau(W_{T,R,RSF}\mu_{RSF})$$ (1) Eq. 21

Note that ϕ = 1.0, per (1) 4.3.6.1.2
Per (2), and the previous calculation, the angle of internal friction beneath the RSF is equal to 31 degrees

$R_{R, RSF}$ = 18.44 klf = 1.0 x 30.69 klf x tan(31)

OK

Calculate External Bearing Resistance

Calculate V_R

Per (1) Eq. 26,

$$\sum V_R = \gamma_{EV\,MAX}(W) + \gamma_{EV\,MAX}(W_{RSF}) + \gamma_{DC\,MAX}(W_{face}) + \gamma_{LS}(q_t b_{rb,t}) + \gamma_{EH\,MAX}(q_{rb} b_{rb,t}) + \gamma_{DC\,MAX}(q_{DL}b) + \gamma_{LS}(q_{LL}b)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\,MIN}$

W =	29.97 klf =	1.35 x	22201.34 plf/	1000	
W_{RSF} =	5.24 klf =	1.35 x	3881.25 plf/	1000	
W_{FACE} =	2.05 klf =	1.25 x	1643.33 plf/	1000	
W_{SF} =	4.22 klf =	1.35 x	3125.00 plf/	1000	
Q_t =	0.00 klf =	1.75 x	(0.00 psf x	0.00 ft)/	1000
Total =	41.48 klf				

Calculate $M_{D,R}$

Per (1) Eq. 28

$$\sum M_{D,R} = \gamma_{EH\,MAX}F_{b,RSF}\left(\frac{H + D_{RSF}}{3}\right) + \gamma_{LS}F_{t,RSF}\left(\frac{H + D_{RSF}}{2}\right) + \gamma_{EH\,MAX}F_{rb,RSF}\left(\frac{H + D_{RSF}}{2}\right)$$

$F_{b, RSF}$ =	131.63 k-ft/ ft =	1.5 x	12.07 klf x	(0.33 x	(19.3055 ft +	2.5 ft))
$F_{t, RSF}$ =	0.00 k-ft/ ft =	1.75 x	0.00 klf x	(0.50 x	(19.3055 ft +	2.5 ft))
Total =	131.63 k-ft/ ft					

Note that $M_{D,R}$ is taken about the bottom center of the width of the RSF.

Calculate $M_{R,R}$

Per (1) Eq. 29

$$\sum M_{R,R} = (\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b)\left[\left(\frac{b}{2} + a_b\right) - \left(\frac{B_{RSF}}{2} - x_{RSF} - b_{block}\right)\right] + \left(\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t}\right)\left(\frac{B_{RSF}}{2} - \frac{b_{rb}}{2}\right) + \gamma_{EV\,MAX} W\left(\frac{B_{RSF}}{2} - \frac{B}{2}\right) + \gamma_{DC\,MAX} W_{face}\left(B + \frac{b_{block}}{2} - \frac{B_{RSF}}{2}\right)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\,MAX}$. Resisting component = W_{SF}*((X_{RSF}+D_{block}+(2/3)*B)-BRSF/2)

W =	52.45 k-ft =	1.35 x	22.20 klf x	(6.75 ft -	5 ft)				
W_{Face} =	-8.32 k-ft =	1.35 x	1.64 klf x	((2.5 ft +	0.5 ft) -	6.75 ft)			
W_{SF} =	14.41 k-ft =	1.35 x	3.13 klf x	((2.5 ft +	1 ft +	(0.67 x	10 ft)) -	(13.5 ft/ 2))	
Total =	58.55 k-ft/ ft								

Note that $M_{R,R}$ is taken about the bottom center of the width of the RSF.

Calculate Eccentricity, $e_{B,R}$

Per (1) Eq. 27

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R}$$

$e_{B,R}$ = 1.76 ft = $\frac{131.63 \text{ k-ft/ ft} - 58.55 \text{ k-ft/ ft}}{41.48 \text{ klf}}$

Calculate $\sigma_{v,base,R}$

Since $e_{B,R}$ = negative, set equal to zero in calcaulation of base bearing pressure.

Per (1) Eq. 25,

$$\sigma_{v,base,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}}$$

$\sigma_{v,base,R}$ = 4.16 ksf = 41.48 klf/ (13.5 ft - (2 x 1.76 ft)

Per abutment bearing capacity calculation, allowable bearing capacity = 9.15 ksf

OK

Calculate Reinforcement Strength

q_t = 0.00 psf
W = 22201.34 psf
W_{RSF} = 3881.25 psf
W_{face} = 1643.33 psf

The evaulation of the abutment for the strength limit state is conducted according to (1) 4.3.7.3.1. and using the following equations:

Required Reinforcment:

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 40

Factored Total Lateral Pressure:

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f}$$ (3) Eq. 41

Where:

$$\sigma_{h,W,f} = \gamma_{EH\ MAX}(\gamma_r z K_{ar})$$ (3) Eq. 42

Where:
$\gamma_{EH\ MAX}$ = maximum horizontal earth pressure load factor.
γ_r = unit weight of reinforced backfill.
z = depth from the top of the wall.
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar}$$ (3) Eq. 45

Note that $\sigma_{hbridge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Factored Reinforcement Strength:

$$T_{f,f} = \Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right) = 0.4T_f$$ (3) Eq. 48

Strength Limit State

z	$\sigma_{h,W,f}$	$\sigma_{h,t,f}$	$\sigma_{h,f}$	$T_{req,f}$	$T_{f,f}$	
0.67 ft	0.02 ksf	0.00 ksf	0.02 ksf	0.03 ksf	2.36 klf	OK
1.33 ft	0.05 ksf	0.00 ksf	0.05 ksf	0.06 ksf	2.36 klf	OK
2.00 ft	0.07 ksf	0.00 ksf	0.07 ksf	0.09 ksf	2.36 klf	OK
2.67 ft	0.10 ksf	0.00 ksf	0.10 ksf	0.12 ksf	2.36 klf	OK
3.33 ft	0.12 ksf	0.00 ksf	0.12 ksf	0.16 ksf	2.36 klf	OK
4.00 ft	0.15 ksf	0.00 ksf	0.15 ksf	0.19 ksf	2.36 klf	OK
4.67 ft	0.17 ksf	0.00 ksf	0.17 ksf	0.22 ksf	2.36 klf	OK
5.33 ft	0.20 ksf	0.00 ksf	0.20 ksf	0.25 ksf	2.36 klf	OK
6.00 ft	0.22 ksf	0.00 ksf	0.22 ksf	0.28 ksf	2.36 klf	OK
6.67 ft	0.25 ksf	0.00 ksf	0.25 ksf	0.31 ksf	2.36 klf	OK
7.33 ft	0.27 ksf	0.00 ksf	0.27 ksf	0.34 ksf	2.36 klf	OK
8.00 ft	0.30 ksf	0.00 ksf	0.30 ksf	0.37 ksf	2.36 klf	OK
8.67 ft	0.32 ksf	0.00 ksf	0.32 ksf	0.40 ksf	2.36 klf	OK
9.33 ft	0.35 ksf	0.00 ksf	0.35 ksf	0.44 ksf	2.36 klf	OK
10.00 ft	0.37 ksf	0.00 ksf	0.37 ksf	0.47 ksf	2.36 klf	OK
10.67 ft	0.40 ksf	0.00 ksf	0.40 ksf	0.50 ksf	2.36 klf	OK
11.33 ft	0.42 ksf	0.00 ksf	0.42 ksf	0.53 ksf	2.36 klf	OK
12.00 ft	0.45 ksf	0.00 ksf	0.45 ksf	0.56 ksf	2.36 klf	OK
12.67 ft	0.47 ksf	0.00 ksf	0.47 ksf	0.59 ksf	2.36 klf	OK
13.33 ft	0.49 ksf	0.00 ksf	0.49 ksf	0.62 ksf	2.36 klf	OK
14.00 ft	0.52 ksf	0.00 ksf	0.52 ksf	0.65 ksf	2.36 klf	OK
14.67 ft	0.54 ksf	0.00 ksf	0.54 ksf	0.68 ksf	2.36 klf	OK
15.33 ft	0.57 ksf	0.00 ksf	0.57 ksf	0.71 ksf	2.36 klf	OK
16.00 ft	0.59 ksf	0.00 ksf	0.59 ksf	0.75 ksf	2.36 klf	OK
19.31 ft	0.72 ksf	0.00 ksf	0.72 ksf	0.90 ksf	2.36 klf	OK

The evaulation of the abutment for the service limit state is conducted according to (1) 4.3.7.3.2. and using the following equations:

The nominal required reinforcement strength:

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 50

The lateral pressure within the GRS Abutment at a given dpeth and location:

$$\sigma_h = \sigma_{h,W} + \sigma_{h,bridge,eq} + \sigma_{h,rb} + \sigma_{h,t}$$ (3) Eq. 51

Where:

$$\sigma_{h,W} = \gamma_r z K_{ar}$$ (3) Eq. 2

$$\sigma_{h,t} = q_t K_{ar}$$ (3) Eq. 54

Note that $\sigma_{hbrdge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

FINAL DESIGN - GRS-IBS WINGWALL -ABUTMENT 3 - NWWW - STEP 2 VTRANS D37 IM 091-1(68)

Service Limit State

z	$\sigma_{h,W}$	$\sigma_{h,t}$	σ_h	T_{req}	$T_{@\varepsilon = 2\%}$	
0.67 ft	0.02 ksf	0.00 ksf	0.016 ksf	0.02 klf	1.25 klf	OK
1.33 ft	0.03 ksf	0.00 ksf	0.033 ksf	0.04 klf	1.25 klf	OK
2.00 ft	0.05 ksf	0.00 ksf	0.049 ksf	0.06 klf	1.25 klf	OK
2.67 ft	0.07 ksf	0.00 ksf	0.066 ksf	0.08 klf	1.25 klf	OK
3.33 ft	0.08 ksf	0.00 ksf	0.082 ksf	0.10 klf	1.25 klf	OK
4.00 ft	0.10 ksf	0.00 ksf	0.099 ksf	0.12 klf	1.25 klf	OK
4.67 ft	0.12 ksf	0.00 ksf	0.115 ksf	0.15 klf	1.25 klf	OK
5.33 ft	0.13 ksf	0.00 ksf	0.132 ksf	0.17 klf	1.25 klf	OK
6.00 ft	0.15 ksf	0.00 ksf	0.148 ksf	0.19 klf	1.25 klf	OK
6.67 ft	0.16 ksf	0.00 ksf	0.165 ksf	0.21 klf	1.25 klf	OK
7.33 ft	0.18 ksf	0.00 ksf	0.181 ksf	0.23 klf	1.25 klf	OK
8.00 ft	0.20 ksf	0.00 ksf	0.198 ksf	0.25 klf	1.25 klf	OK
8.67 ft	0.21 ksf	0.00 ksf	0.214 ksf	0.27 klf	1.25 klf	OK
9.33 ft	0.23 ksf	0.00 ksf	0.231 ksf	0.29 klf	1.25 klf	OK
10.00 ft	0.25 ksf	0.00 ksf	0.247 ksf	0.31 klf	1.25 klf	OK
10.67 ft	0.26 ksf	0.00 ksf	0.264 ksf	0.33 klf	1.25 klf	OK
11.33 ft	0.28 ksf	0.00 ksf	0.280 ksf	0.35 klf	1.25 klf	OK
12.00 ft	0.30 ksf	0.00 ksf	0.297 ksf	0.37 klf	1.25 klf	OK
12.67 ft	0.31 ksf	0.00 ksf	0.313 ksf	0.39 klf	1.25 klf	OK
13.33 ft	0.33 ksf	0.00 ksf	0.330 ksf	0.41 klf	1.25 klf	OK
14.00 ft	0.35 ksf	0.00 ksf	0.346 ksf	0.44 klf	1.25 klf	OK
14.67 ft	0.36 ksf	0.00 ksf	0.363 ksf	0.46 klf	1.25 klf	OK
15.33 ft	0.38 ksf	0.00 ksf	0.379 ksf	0.48 klf	1.25 klf	OK
16.00 ft	0.40 ksf	0.00 ksf	0.396 ksf	0.50 klf	1.25 klf	OK
19.31 ft	0.48 ksf	0.00 ksf	0.477 ksf	0.60 klf	1.25 klf	OK

References:

1) GRS-IBS Interim Implementation Guide, FHWA, June 2018
(2) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018
(3) Geotechnical Data Report, Teracon Consultants Inc., March 27. 2019

Material Properties

γ_f =	115.00 pcf	(Reinforced Fill)
ϕ (Reinforced Soil)=	45	degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β =	26.57	degrees

K_a (Reinforced Soil) = 0.22 = 0.894 x (0.894 - (0.800 - 0.5)^ 0.5) / (0.894 + (0.800 - 0.5)^ 0.5)

Fill retained is in-situ soil.

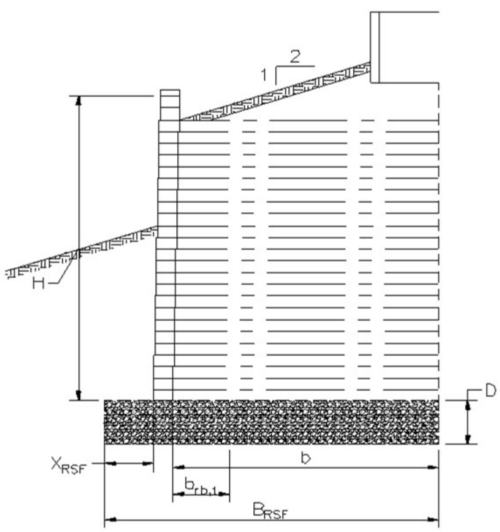
γ_b =	125.00 pcf	(Retained Fill)
ϕ (Existing Filll)=	34.00	degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β =	26.57	degrees

K_a (Retained Fill) = 0.41 = 0.894 x (0.894 - (0.800 - 0.6873)^ 0.5) / (0.894 + (0.800 - 0.6873)^ 0.5)

Geometry



Depth of RSF, D =	2.5 ft		
Top of RSF EL. =	448.46		
Bottom of RSF EL. =	445.96 =	448.46 -	2.5 ft
Top of Wall EL. =	474.25		
Wall Height =	25.79 ft	474.25 -	448.46
H =	25.79 ft		
B =	13.25 ft		
Toe Length, X_{RSF} =	2.5 ft		
Total Width, B_{RSF} =	16.75 ft		
$b_{rb,t}$ =	0.00 ft		
Stepped Footing Length =	14.00 ft		
h_{block} =	8.00 in		
D_{block} =	12.00 in		
L_{block} =	18.00 in		
Weight =	85 lbs	per block	

Reinforcement

S_v =	0.67 ft
d_{max} =	0.75 in
T_f =	5.90 klf

Loading

Note that the superstructure dead loads, roadway bed loads, and traffic live loads are not applicable for this structure. No live load surcharge is applied.

Load Factors

Per (3) Table 3.4.1-1 and Table 3.4.1-2.

γ_p (DC, Min.) =	0.90	3) Table 3.4.1-2
γ_p (DC, Max.) =	1.25	3) Table 3.4.1-2
γ_p (EH, Max) =	1.50	3) Table 3.4.1-2
γ_p (EV, Min.) =	1.00	3) Table 3.4.1-2
γ_p (EV, Max.) =	1.35	3) Table 3.4.1-2
Factor (LS) =	1.75	3) Table 3.4.1-1

Calculate Vertical Loads and Applied Pressures

Calculate Traffic Surcharge, q_T

Due to wingwall layout geometry, assume that there is no live load surcharge acting on the wingwall.

h_{eq} = 0.00 ft

γ_r= 125.00 pcf

$$q_t = h_{eq}\gamma_b$$

q_T = 0.00 psf = 0.00 ft x 125.00 pcf

Calculate Weight of GRS Abutment

$$W = BH\gamma_r$$

H = 25.79 ft

B = 13.25 ft

γ_r= 115.00 pcf

W = 39301.56 plf = 25.79266 ft x 13.25 ft x 115.00 pcf

Calculate Weight of RSF

$$W_{RSF} = B_{RSF}D_{RSF}\gamma_r$$

B = 16.75 ft

D = 2.5 ft

γ_r= 125.00 pcf

W_{RSF} = 4815.63 plf = 16.75 ft x 2.5 ft x 115.00 pcf

Calculate Weight of Facing

N_{block} = 38.69 = 25.792658 ft/ 0.67 ft

N_{block} = 39

$$W_{face} = N_{block}\frac{W_{block}}{L_{block}}$$

W_{face} = 2210.00 plf = 39 x (85 lbs/ (18.00 in/ 12)

Calculate Weight of Surplus Retained Fill

Fill is retained at a 2H:1V slope

B =	13.25 ft					
H_{max} =	6.63 ft =	13.25 ft/ 2				
W_{SF} =	5486.33 plf =		0.5 x	13.25 ft x	6.63 ft x	125.00 pcf

Summary of Applied Loads

q_t =	0.00 psf
W =	39301.56 plf
W_{RSF} =	4815.63 plf
W_{face} =	2210.00 plf
W_{SF} =	5486.33 plf

Calculte Direct Sliding Effects at RSF/GRS Interface

Calculate Sliding per the (1) and (3) Strength I Load Combination

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2$$ (1) Eq. 9

$$F_t = q_t K_{ab} H$$ (1) Eq. 11

$$F_R = \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t$$ (1) Eq. 12

F_b =	16891.31 lbs =	0.5 x	125.00 pcf x	0.41 x	25.79266 ft ^ 2	
F_t =	0.00 lbs =	0.00 psf		0.41 x	25.7926576 ft	
F_R =	25.34 klf =	(1.5 x	16891.31 lbs) +	(1.75 x	0.00 lbs))/	1000

$$W_{T,R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EV\,MIN}(q_{rb} b_{rb,t})$$ + $\gamma_{EV\,MIN} W_{SF}$ (1) Eq. 14

$W_{T,R}$ = 46.78 klf = ((1.0 x 39301.56 plf) + ((0.9 x 2210.00 plf))/ 1000) + (1.00 x 5.49 klf)

Assume that μ = 2/3*tanϕ, per (1) C.1.6.1.1

μ = 0.67 = 0.67 x tan(45)

$$R_R = \Phi_\tau (W_{T,R}\mu)$$ (1) Eq. 13

R_R = 31.18 klf = 46.777 klf x 0.667

OK

Calculate Direct Sliding at Base of RSF

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2$$ (3) Eq. 17

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF})$$ (3) Eq. 19

$F_{b,\ RSF}$ =	20.32 klf =	(0.5 x	125.00 pcf x	0.41 x	(25.79 ft +	2.5)^ 2)/	1000
$F_{t,\ RSF}$ =	0.00 klf =	0.00 psf x	0.41 x	(25.79 ft +	2.5 ft))/	1000	

$$F_{R,RSF} = \gamma_{EH\ MAX}(F_{b,RSF} + F_{rb,RSF}) + \gamma_{LS}F_{t,RSF}$$ (1) Eq. 20

$F_{R,RSF}$ = 30.49 klf = (1.5 x x 20.32 klf) + (1.75 x 0.00 klf)

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\ MIN}W_{RSF}$$ (1) Eq. 22

$W_{T,R,RSF}$ = 51.59 klf = 46.78 klf + (1.0 x 4.82 klf)

$$R_{R,RSF} = \Phi_\tau(W_{T,R,RSF}\mu_{RSF})$$ (1) Eq. 21

Note that ϕ = 1.0, per (1) 4.3.6.1.2
Per (2), and the previous calculation, the angle of internal friction beneath the RSF is equal to 31 degrees

$R_{R,\ RSF}$ = 31.00 klf = 1.0 x 51.59 klf x tan(31)

OK

Calculate External Bearing Resistance

Calculate V_R

Per (1) Eq. 26,

$$\sum V_R = \gamma_{EV\ MAX}(W) + \gamma_{EV\ MAX}(W_{RSF}) + \gamma_{DC\ MAX}(W_{face}) + \gamma_{LS}(q_t b_{rb,t}) + \gamma_{EH\ MAX}(q_{rb} b_{rb,t}) + \gamma_{DC\ MAX}(q_{DL} b) + \gamma_{LS}(q_{LL} b)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\ MIN}$

W =	53.06 klf =	1.35 x	39301.56 plf/	1000	
W_{RSF} =	6.50 klf =	1.35 x	4815.63 plf/	1000	
W_{FACE} =	2.76 klf =	1.25 x	2210.00 plf/	1000	
W_{SF} =	7.41 klf =	1.35 x	5486.33 plf/	1000	
Q_t =	0.00 klf =	1.75 x	(0.00 psf x	0.00 ft)/	1000
Total =	69.73 klf				

Calculate $M_{D,R}$

Per (1) Eq. 28

$$\sum M_{D,R} = \gamma_{EH\ MAX}F_{b,RSF}\left(\frac{H + D_{RSF}}{3}\right) + \gamma_{LS}F_{t,RSF}\left(\frac{H + D_{RSF}}{2}\right) + \gamma_{EH\ MAX}F_{rb,RSF}\left(\frac{H + D_{RSF}}{2}\right)$$

$F_{b,\ RSF}$ =	287.52 k-ft/ ft =	1.5 x	20.32 klf x	(0.33 x	(25.7927 ft +	2.5 ft))
$F_{t,\ RSF}$ =	0.00 k-ft/ ft =	1.75 x	0.00 klf x	(0.50 x	(25.7927 ft +	2.5 ft))
Total =	287.52 k-ft/ ft					

Note that $M_{D,R}$ is taken about the bottom center of the width of the RSF.

Calculate $M_{R,R}$

Per (1) Eq. 29

$$\sum M_{R,R} = (\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b)\left[\left(\frac{b}{2} + a_b\right) - \left(\frac{B_{RSF}}{2} - x_{RSF} - b_{block}\right)\right] + \left(\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t}\right)\left(\frac{B_{RSF}}{2} - \frac{b_{rb}}{2}\right) + \gamma_{EV\,MAX} W\left(\frac{B_{RSF}}{2} - \frac{B}{2}\right) + \gamma_{DC\,MAX} W_{face}\left(B + \frac{b_{block}}{2} - \frac{B_{RSF}}{2}\right)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\,MAX}$. Resisting component = W_{SF}*((X_{RSF}+D_{block}+(2/3)*B)-BRSF/2)

W =	92.85 k-ft =	1.35 x	39.30 klf x	(8.375 ft -	6.625 ft)				
W_{Face} =	-16.04 k-ft =	1.35 x	2.21 klf x	((2.5 ft +	0.5 ft) -	8.375 ft)			
W_{SF} =	29.32 k-ft =	1.35 x	5.49 klf x	((2.5 ft +	1 ft +	(0.67 x	13.25 ft)) -	(16.75 ft/ 2))	
Total =	106.13 k-ft/ ft								

Note that $M_{R,R}$ is taken about the bottom center of the width of the RSF.

Calculate Eccentricity, $e_{B,R}$

Per (1) Eq. 27

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R}$$

$e_{B,R}$ = 2.60 ft = (287.52 k-ft/ ft - 106.13 k-ft/ ft) / 69.73 klf

Calculate $\sigma_{v,base,R}$

Since $e_{B,R}$ = negative, set equal to zero in calcaulation of base bearing pressure.

Per (1) Eq. 25,

$$\sigma_{v,base,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}}$$

$\sigma_{v,base,R}$ = 6.04 ksf = 69.73 klf/ (16.75 ft - (2 x 2.60 ft)

Per abutment bearing capacity calculation, allowable bearing capacity = 9.15 ksf

OK

Calculate Reinforcement Strength

q_t = 0.00 psf
W = 39301.56 psf
W_{RSF} = 4815.63 psf
W_{face} = 2210.00 psf

The evaulation of the abutment for the strength limit state is conducted according to (1) 4.3.7.3.1. and using the following equations:

Required Reinforcment:

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v \quad \text{(3) Eq. 40}$$

Factored Total Lateral Pressure:

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f} \quad \text{(3) Eq. 41}$$

Where:

$$\sigma_{h,W,f} = \gamma_{EH\ MAX}(\gamma_r z K_{ar})$$

(3) Eq. 42

Where:

$\gamma_{EH\ MAX}$ = maximum horizontal earth pressure load factor.
γ_r = unit weight of reinforced backfill.
z = depth from the top of the wall.
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar}$$

(3) Eq. 45

Note that $\sigma_{hbridge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Factored Reinforcement Strength:

$$T_{f,f} = \Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right) = 0.4T_f$$

(3) Eq. 48

Strength Limit State

z	$\sigma_{h,W,f}$	$\sigma_{h,t,f}$	$\sigma_{h,f}$	$T_{req,f}$	$T_{f,f}$	
0.67 ft	0.02 ksf	0.00 ksf	0.02 ksf	0.03 ksf	2.36 klf	OK
1.33 ft	0.05 ksf	0.00 ksf	0.05 ksf	0.06 ksf	2.36 klf	OK
2.00 ft	0.07 ksf	0.00 ksf	0.07 ksf	0.09 ksf	2.36 klf	OK
2.67 ft	0.10 ksf	0.00 ksf	0.10 ksf	0.12 ksf	2.36 klf	OK
3.33 ft	0.12 ksf	0.00 ksf	0.12 ksf	0.16 ksf	2.36 klf	OK
4.00 ft	0.15 ksf	0.00 ksf	0.15 ksf	0.19 ksf	2.36 klf	OK
4.67 ft	0.17 ksf	0.00 ksf	0.17 ksf	0.22 ksf	2.36 klf	OK
5.33 ft	0.20 ksf	0.00 ksf	0.20 ksf	0.25 ksf	2.36 klf	OK
6.00 ft	0.22 ksf	0.00 ksf	0.22 ksf	0.28 ksf	2.36 klf	OK
6.67 ft	0.25 ksf	0.00 ksf	0.25 ksf	0.31 ksf	2.36 klf	OK
7.33 ft	0.27 ksf	0.00 ksf	0.27 ksf	0.34 ksf	2.36 klf	OK
8.00 ft	0.30 ksf	0.00 ksf	0.30 ksf	0.37 ksf	2.36 klf	OK
8.67 ft	0.32 ksf	0.00 ksf	0.32 ksf	0.40 ksf	2.36 klf	OK
9.33 ft	0.35 ksf	0.00 ksf	0.35 ksf	0.44 ksf	2.36 klf	OK
10.00 ft	0.37 ksf	0.00 ksf	0.37 ksf	0.47 ksf	2.36 klf	OK
10.67 ft	0.40 ksf	0.00 ksf	0.40 ksf	0.50 ksf	2.36 klf	OK
11.33 ft	0.42 ksf	0.00 ksf	0.42 ksf	0.53 ksf	2.36 klf	OK
12.00 ft	0.45 ksf	0.00 ksf	0.45 ksf	0.56 ksf	2.36 klf	OK
12.67 ft	0.47 ksf	0.00 ksf	0.47 ksf	0.59 ksf	2.36 klf	OK
13.33 ft	0.49 ksf	0.00 ksf	0.49 ksf	0.62 ksf	2.36 klf	OK
14.00 ft	0.52 ksf	0.00 ksf	0.52 ksf	0.65 ksf	2.36 klf	OK
14.67 ft	0.54 ksf	0.00 ksf	0.54 ksf	0.68 ksf	2.36 klf	OK
15.33 ft	0.57 ksf	0.00 ksf	0.57 ksf	0.71 ksf	2.36 klf	OK
16.00 ft	0.59 ksf	0.00 ksf	0.59 ksf	0.75 ksf	2.36 klf	OK
16.67 ft	0.62 ksf	0.00 ksf	0.62 ksf	0.78 ksf	2.36 klf	OK
17.33 ft	0.64 ksf	0.00 ksf	0.64 ksf	0.81 ksf	2.36 klf	OK
18.00 ft	0.67 ksf	0.00 ksf	0.67 ksf	0.84 ksf	2.36 klf	OK
18.67 ft	0.69 ksf	0.00 ksf	0.69 ksf	0.87 ksf	2.36 klf	OK
19.33 ft	0.72 ksf	0.00 ksf	0.72 ksf	0.90 ksf	2.36 klf	OK
20.00 ft	0.74 ksf	0.00 ksf	0.74 ksf	0.93 ksf	2.36 klf	OK
20.67 ft	0.77 ksf	0.00 ksf	0.77 ksf	0.96 ksf	2.36 klf	OK
21.33 ft	0.79 ksf	0.00 ksf	0.79 ksf	0.99 ksf	2.36 klf	OK
22.00 ft	0.82 ksf	0.00 ksf	0.82 ksf	1.03 ksf	2.36 klf	OK
25.79 ft	0.96 ksf	0.00 ksf	0.96 ksf	1.20 ksf	2.36 klf	OK

The evaluation of the abutment for the service limit state is conducted according to (1) 4.3.7.3.2. and using the following equations:

The nominal required reinforcement strength:

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 50

The lateral pressure within the GRS Abutment at a given dpeth and location:

$$\sigma_h = \sigma_{h,W} + \sigma_{h,bridge,eq} + \sigma_{h,rb} + \sigma_{h,t}$$ (3) Eq. 51

Where:

$$\sigma_{h,W} = \gamma_r z K_{ar}$$ (3) Eq. 2

$$\sigma_{h,t} = q_t K_{ar}$$ (3) Eq. 54

Note that $\sigma_{hbridge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Service Limit State

z	$\sigma_{h,W}$	$\sigma_{h,t}$	σ_h	T_{req}	$T_{@\varepsilon = 2\%}$	
0.67 ft	0.02 ksf	0.00 ksf	0.016 ksf	0.02 klf	1.25 klf	OK
1.33 ft	0.03 ksf	0.00 ksf	0.033 ksf	0.04 klf	1.25 klf	OK
2.00 ft	0.05 ksf	0.00 ksf	0.049 ksf	0.06 klf	1.25 klf	OK
2.67 ft	0.07 ksf	0.00 ksf	0.066 ksf	0.08 klf	1.25 klf	OK
3.33 ft	0.08 ksf	0.00 ksf	0.082 ksf	0.10 klf	1.25 klf	OK
4.00 ft	0.10 ksf	0.00 ksf	0.099 ksf	0.12 klf	1.25 klf	OK
4.67 ft	0.12 ksf	0.00 ksf	0.115 ksf	0.15 klf	1.25 klf	OK
5.33 ft	0.13 ksf	0.00 ksf	0.132 ksf	0.17 klf	1.25 klf	OK
6.00 ft	0.15 ksf	0.00 ksf	0.148 ksf	0.19 klf	1.25 klf	OK
6.67 ft	0.16 ksf	0.00 ksf	0.165 ksf	0.21 klf	1.25 klf	OK
7.33 ft	0.18 ksf	0.00 ksf	0.181 ksf	0.23 klf	1.25 klf	OK
8.00 ft	0.20 ksf	0.00 ksf	0.198 ksf	0.25 klf	1.25 klf	OK
8.67 ft	0.21 ksf	0.00 ksf	0.214 ksf	0.27 klf	1.25 klf	OK
9.33 ft	0.23 ksf	0.00 ksf	0.231 ksf	0.29 klf	1.25 klf	OK
10.00 ft	0.25 ksf	0.00 ksf	0.247 ksf	0.31 klf	1.25 klf	OK
10.67 ft	0.26 ksf	0.00 ksf	0.264 ksf	0.33 klf	1.25 klf	OK
11.33 ft	0.28 ksf	0.00 ksf	0.280 ksf	0.35 klf	1.25 klf	OK
12.00 ft	0.30 ksf	0.00 ksf	0.297 ksf	0.37 klf	1.25 klf	OK
12.67 ft	0.31 ksf	0.00 ksf	0.313 ksf	0.39 klf	1.25 klf	OK
13.33 ft	0.33 ksf	0.00 ksf	0.330 ksf	0.41 klf	1.25 klf	OK
14.00 ft	0.35 ksf	0.00 ksf	0.346 ksf	0.44 klf	1.25 klf	OK
14.67 ft	0.36 ksf	0.00 ksf	0.363 ksf	0.46 klf	1.25 klf	OK
15.33 ft	0.38 ksf	0.00 ksf	0.379 ksf	0.48 klf	1.25 klf	OK
16.00 ft	0.40 ksf	0.00 ksf	0.396 ksf	0.50 klf	1.25 klf	OK
16.67 ft	0.41 ksf	0.00 ksf	0.412 ksf	0.52 klf	1.25 klf	OK
17.33 ft	0.43 ksf	0.00 ksf	0.429 ksf	0.54 klf	1.25 klf	OK
18.00 ft	0.45 ksf	0.00 ksf	0.445 ksf	0.56 klf	1.25 klf	OK
18.67 ft	0.46 ksf	0.00 ksf	0.462 ksf	0.58 klf	1.25 klf	OK
19.33 ft	0.48 ksf	0.00 ksf	0.478 ksf	0.60 klf	1.25 klf	OK
20.00 ft	0.49 ksf	0.00 ksf	0.495 ksf	0.62 klf	1.25 klf	OK
20.67 ft	0.51 ksf	0.00 ksf	0.511 ksf	0.64 klf	1.25 klf	OK
21.33 ft	0.53 ksf	0.00 ksf	0.528 ksf	0.66 klf	1.25 klf	OK
22.00 ft	0.54 ksf	0.00 ksf	0.544 ksf	0.68 klf	1.25 klf	OK
25.79 ft	0.64 ksf	0.00 ksf	0.638 ksf	0.80 klf	1.25 klf	OK

References:

1) GRS-IBS Interim Implementation Guide, FHWA, June 2018
(2) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018
(3) Geotechnical Data Report, Teracon Consultants Inc., March 27. 2019

Material Properties

γ_f = 115.00 pcf (Reinforced Fill)
ϕ (Reinforced Soil) = 45 degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β = 26.57 degrees

K_a (Reinforced Soil) = 0.22 = 0.894 x $\frac{0.894 - (0.800 - 0.5)^\wedge 0.5}{0.894 + (0.800 - 0.5)^\wedge 0.5}$

Fill retained is in-situ soil.

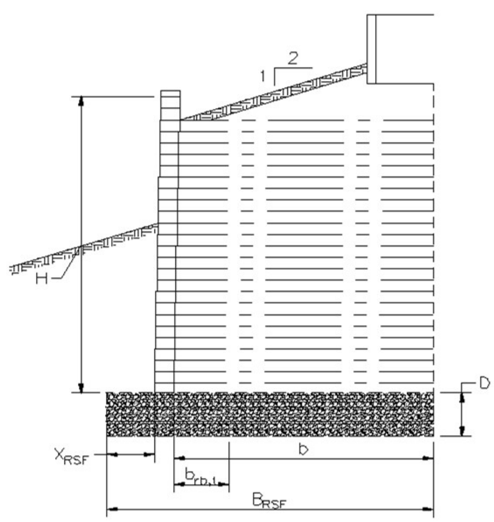
γ_b = 125.00 pcf (Retained Fill)
ϕ (Existing Filll) = 34.00 degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β = 26.57 degrees

K_a (Retained Fill) = 0.41 = 0.894 x $\frac{0.894 - (0.800 - 0.6873)^\wedge 0.5}{0.894 + (0.800 - 0.6873)^\wedge 0.5}$

Geometry



Depth of RSF, D = 2.5 ft
Top of RSF EL. = 450.96
Bottom of RSF EL. = 448.46 = 450.96 - 2.5 ft

Top of Wall EL. = 466.58
Wall Height = 15.62 ft 466.58 - 450.96
H = 15.62 ft

B = 8 ft
Toe Length, X_{RSF} = 2.5 ft
Total Width, B_{RSF} = 11.5 ft
$b_{rb,t}$ = 0.00 ft
Stepped Footing Length = 12.00 ft

h_{block} = 8.00 in
D_{block} = 12.00 in
L_{block} = 18.00 in
Weight = 85 lbs per block

Reinforcement

S_v = 0.67 ft
d_{max} = 0.75 in
T_f = 5.90 klf

Loading
Note that the superstructure dead loads, roadway bed loads, and traffic live loads are not applicable for this structure. No live load surcharge is applied.

Load Factors
Per (3) Table 3.4.1-1 and Table 3.4.1-2.

γ_p (DC, Min.) =	0.90	3) Table 3.4.1-2
γ_p (DC, Max.) =	1.25	3) Table 3.4.1-2
γ_p (EH, Max) =	1.50	3) Table 3.4.1-2
γ_p (EV, Min.) =	1.00	3) Table 3.4.1-2
γ_p (EV, Max.) =	1.35	3) Table 3.4.1-2
Factor (LS) =	1.75	3) Table 3.4.1-1

Calculate Vertical Loads and Applied Pressures
Calculate Traffic Surcharge, q_T

Due to wingwall layout geometry, assume that there is no live load surcharge acting on the wingwall.

h_{eq} = 0.00 ft
γ_r= 125.00 pcf

$$q_t = h_{eq}\gamma_b$$

q_T = 0.00 psf = 0.00 ft x 125.00 pcf

Calculate Weight of GRS Abutment

$$W = BH\gamma_r$$

H = 15.62 ft
B = 8 ft
γ_r= 115.00 pcf

W = 14369.09 plf = 15.61857 ft x 8 ft x 115.00 pcf

Calculate Weight of RSF

$$W_{RSF} = B_{RSF}D_{RSF}\gamma_r$$

B = 11.5 ft
D = 2.5 ft
γ_r= 125.00 pcf

W_{RSF} = 3306.25 plf = 11.5 ft x 2.5 ft x 115.00 pcf

Calculate Weight of Facing

N_{block} = 23.43 = 15.618571 ft/ 0.67 ft
N_{block} = 24

$$W_{face} = N_{block}\frac{W_{block}}{L_{block}}$$

W_{face} = 1360.00 plf = 24 x (85 lbs/ (18.00 in/ 12)

Calculate Weight of Surplus Retained Fill

Fill is retained at a 2H:1V slope

B = 8 ft
H_{max} = 4.00 ft = 8 ft/ 2
W_{SF} = 2000.00 plf = 0.5 x 8 ft x 4.00 ft x 125.00 pcf

Summary of Applied Loads

q_t = 0.00 psf
W = 14369.09 plf
W_{RSF} = 3306.25 plf
W_{face} = 1360.00 plf
W_{SF} = 2000.00 plf

Calculte Direct Sliding Effects at RSF/GRS Interface

Calculate Sliding per the (1) and (3) Strength I Load Combination

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2$$ (1) Eq. 9

$$F_t = q_t K_{ab} H$$ (1) Eq. 11

$$F_R = \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t$$ (1) Eq. 12

F_b = 6193.75 lbs = 0.5 x 125.00 pcf x 0.41 x 15.61857 ft^ 2
F_t = 0.00 lbs = 0.00 psf 0.41 x 15.6185714 ft

F_R = 9.29 klf = (1.5 x 6193.75 lbs) + (1.75 x 0.00 lbs))/ 1000

$$W_{T,R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EV\,MIN}(q_{rb} b_{rb,t}) + \gamma_{EV\,MIN} W_{SF}$$ (1) Eq. 14

$W_{T,R}$ = 17.59 klf = ((1.0 x 14369.09 plf) + ((0.9 x 1360.00 plf))/ 1000) + (1.00 x 2.00 klf)

Assume that μ = 2/3*tanϕ, per (1) C.1.6.1.1

μ = 0.67 = 0.67 x tan(45)

$$R_R = \Phi_\tau (W_{T,R}\mu)$$ (1) Eq. 13

R_R = 11.73 klf = 17.593 klf x 0.667

OK

Calculate Direct Sliding at Base of RSF

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2$$ (3) Eq. 17

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF})$$ (3) Eq. 19

$F_{b,\ RSF}$ =	8.34 klf =	(0.5 x	125.00 pcf x	0.41 x	(15.62 ft +	2.5)^ 2)/	1000
$F_{t,\ RSF}$ =	0.00 klf =	0.00 psf x	0.41 x	(15.62 ft +	2.5 ft))/	1000	

$$F_{R,RSF} = \gamma_{EH\ MAX}(F_{b,RSF} + F_{rb,RSF}) + \gamma_{LS}F_{t,RSF}$$ (1) Eq. 20

$F_{R,RSF}$ = 12.50 klf = (1.5 x x 8.34 klf) + (1.75 x 0.00 klf)

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\ MIN}W_{RSF}$$ (1) Eq. 22

$W_{T,R,RSF}$ = 20.90 klf = 17.59 klf + (1.0 x 3.31 klf)

$$R_{R,RSF} = \Phi_\tau(W_{T,R,RSF}\mu_{RSF})$$ (1) Eq. 21

Note that ϕ = 1.0, per (1) 4.3.6.1.2
Per (2), and the previous calculation, the angle of internal friction beneath the RSF is equal to 31 degrees

$R_{R,\ RSF}$ = 12.56 klf = 1.0 x 20.90 klf x tan(31)

OK

Calculate External Bearing Resistance

Calculate V_R

Per (1) Eq. 26,

$$\sum V_R = \gamma_{EV\ MAX}(W) + \gamma_{EV\ MAX}(W_{RSF}) + \gamma_{DC\ MAX}(W_{face}) + \gamma_{LS}(q_t b_{rb,t}) + \gamma_{EH\ MAX}(q_{rb} b_{rb,t}) + \gamma_{DC\ MAX}(q_{DL} b) + \gamma_{LS}(q_{LL} b)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\ MIN}$

W =	19.40 klf =	1.35 x	14369.09 plf/	1000	
W_{RSF} =	4.46 klf =	1.35 x	3306.25 plf/	1000	
W_{FACE} =	1.70 klf =	1.25 x	1360.00 plf/	1000	
W_{SF} =	2.70 klf =	1.35 x	2000.00 plf/	1000	
Q_t =	0.00 klf =	1.75 x	(0.00 psf x	0.00 ft)/	1000
Total =	28.26 klf				

Calculate $M_{D,R}$

Per (1) Eq. 28

$$\sum M_{D,R} = \gamma_{EH\ MAX}F_{b,RSF}\left(\frac{H + D_{RSF}}{3}\right) + \gamma_{LS}F_{t,RSF}\left(\frac{H + D_{RSF}}{2}\right) + \gamma_{EH\ MAX}F_{rb,RSF}\left(\frac{H + D_{RSF}}{2}\right)$$

$F_{b,\ RSF}$ =	75.51 k-ft/ ft =	1.5 x	8.34 klf x	(0.33 x	(15.6186 ft +	2.5 ft))
$F_{t,\ RSF}$ =	0.00 k-ft/ ft =	1.75 x	0.00 klf x	(0.50 x	(15.6186 ft +	2.5 ft))
Total =	75.51 k-ft/ ft					

Note that $M_{D,R}$ is taken about the bottom center of the width of the RSF.

Calculate $M_{R,R}$

Per (1) Eq. 29

$$\sum M_{R,R} = (\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b)\left[\left(\frac{b}{2} + a_b\right) - \left(\frac{B_{RSF}}{2} - x_{RSF} - b_{block}\right)\right] + \left(\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t}\right)\left(\frac{B_{RSF}}{2} - \frac{b_{rb}}{2}\right) + \gamma_{EV\,MAX} W\left(\frac{B_{RSF}}{2} - \frac{B}{2}\right) + \gamma_{DC\,MAX} W_{face}\left(B + \frac{b_{block}}{2} - \frac{B_{RSF}}{2}\right)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\,MAX}$. Resisting component = W_{SF}*((X_{RSF}+D_{block}+(2/3)*B)-BRSF/2)

W =	33.95 k-ft =	1.35 x	14.37 klf x	(5.75 ft -	4 ft)				
W_{Face} =	-5.05 k-ft =	1.35 x	1.36 klf x	((2.5 ft +	0.5 ft) -	5.75 ft)			
W_{SF} =	8.33 k-ft =	1.35 x	2.00 klf x	((2.5 ft +	1 ft +	(0.67 x	8 ft)) -	(11.5 ft/ 2))	
Total =	37.22 k-ft/ ft								

Note that $M_{R,R}$ is taken about the bottom center of the width of the RSF.

Calculate Eccentricity, $e_{B,R}$

Per (1) Eq. 27

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R}$$

$e_{B,R}$ = 1.35 ft = (75.51 k-ft/ ft - 37.22 k-ft/ ft) / 28.26 klf

Calculate $\sigma_{v,base,R}$

Since $e_{B,R}$ = negative, set equal to zero in calcaulation of base bearing pressure.

Per (1) Eq. 25,

$$\sigma_{v,base,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}}$$

$\sigma_{v,base,R}$ = 3.22 ksf = 28.26 klf/ (11.5 ft - (2 x 1.35 ft)

Per abutment bearing capacity calculation, allowable bearing capacity = 9.15 ksf

OK

Calculate Reinforcement Strength

q_t = 0.00 psf
W = 14369.09 psf
W_{RSF} = 3306.25 psf
W_{face} = 1360.00 psf

The evaulation of the abutment for the strength limit state is conducted according to (1) 4.3.7.3.1. and using the following equations:

Required Reinforcment:

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 40

Factored Total Lateral Pressure:

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f}$$ (3) Eq. 41

Where:

$$\sigma_{h,W,f} = \gamma_{EH\,MAX}\left(\gamma_r z K_{ar}\right)$$ (3) Eq. 42

Where:
$\gamma_{EH\,MAX}$ = maximum horizontal earth pressure load factor.
γ_r = unit weight of reinforced backfill.
z = depth from the top of the wall.
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar}$$ (3) Eq. 45

Note that $\sigma_{hbridge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Factored Reinforcement Strength:

$$T_{f,f} = \Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right) = 0.4T_f$$ (3) Eq. 48

Strength Limit State

z	$\sigma_{h,W,f}$	$\sigma_{h,t,f}$	$\sigma_{h,f}$	$T_{req,f}$	$T_{f,f}$	
0.67 ft	0.02 ksf	0.00 ksf	0.02 ksf	0.03 ksf	2.36 klf	OK
1.33 ft	0.05 ksf	0.00 ksf	0.05 ksf	0.06 ksf	2.36 klf	OK
2.00 ft	0.07 ksf	0.00 ksf	0.07 ksf	0.09 ksf	2.36 klf	OK
2.67 ft	0.10 ksf	0.00 ksf	0.10 ksf	0.12 ksf	2.36 klf	OK
3.33 ft	0.12 ksf	0.00 ksf	0.12 ksf	0.16 ksf	2.36 klf	OK
4.00 ft	0.15 ksf	0.00 ksf	0.15 ksf	0.19 ksf	2.36 klf	OK
4.67 ft	0.17 ksf	0.00 ksf	0.17 ksf	0.22 ksf	2.36 klf	OK
5.33 ft	0.20 ksf	0.00 ksf	0.20 ksf	0.25 ksf	2.36 klf	OK
6.00 ft	0.22 ksf	0.00 ksf	0.22 ksf	0.28 ksf	2.36 klf	OK
6.67 ft	0.25 ksf	0.00 ksf	0.25 ksf	0.31 ksf	2.36 klf	OK
7.33 ft	0.27 ksf	0.00 ksf	0.27 ksf	0.34 ksf	2.36 klf	OK
8.00 ft	0.30 ksf	0.00 ksf	0.30 ksf	0.37 ksf	2.36 klf	OK
8.67 ft	0.32 ksf	0.00 ksf	0.32 ksf	0.40 ksf	2.36 klf	OK
9.33 ft	0.35 ksf	0.00 ksf	0.35 ksf	0.44 ksf	2.36 klf	OK
10.00 ft	0.37 ksf	0.00 ksf	0.37 ksf	0.47 ksf	2.36 klf	OK
10.67 ft	0.40 ksf	0.00 ksf	0.40 ksf	0.50 ksf	2.36 klf	OK
11.33 ft	0.42 ksf	0.00 ksf	0.42 ksf	0.53 ksf	2.36 klf	OK
12.00 ft	0.45 ksf	0.00 ksf	0.45 ksf	0.56 ksf	2.36 klf	OK
12.67 ft	0.47 ksf	0.00 ksf	0.47 ksf	0.59 ksf	2.36 klf	OK
13.33 ft	0.49 ksf	0.00 ksf	0.49 ksf	0.62 ksf	2.36 klf	OK
14.00 ft	0.52 ksf	0.00 ksf	0.52 ksf	0.65 ksf	2.36 klf	OK
14.67 ft	0.54 ksf	0.00 ksf	0.54 ksf	0.68 ksf	2.36 klf	OK
15.33 ft	0.57 ksf	0.00 ksf	0.57 ksf	0.71 ksf	2.36 klf	OK
15.62 ft	0.58 ksf	0.00 ksf	0.58 ksf	0.73 ksf	2.36 klf	OK

The evaulation of the abutment for the service limit state is conducted according to (1) 4.3.7.3.2. and using the following equations:

The nominal required reinforcement strength:

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 50

The lateral pressure within the GRS Abutment at a given dpeth and location:

$$\sigma_h = \sigma_{h,W} + \sigma_{h,bridge,eq} + \sigma_{h,rb} + \sigma_{h,t}$$ (3) Eq. 51

Where:

$$\sigma_{h,W} = \gamma_r z K_{ar}$$ (3) Eq. 2

$$\sigma_{h,t} = q_t K_{ar}$$ (3) Eq. 54

Note that $\sigma_{hbrdge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Service Limit State

z	$\sigma_{h,W}$	$\sigma_{h,t}$	σ_h	T_{req}	$T_{@\varepsilon = 2\%}$	
0.67 ft	0.02 ksf	0.00 ksf	0.016 ksf	0.02 klf	1.25 klf	OK
1.33 ft	0.03 ksf	0.00 ksf	0.033 ksf	0.04 klf	1.25 klf	OK
2.00 ft	0.05 ksf	0.00 ksf	0.049 ksf	0.06 klf	1.25 klf	OK
2.67 ft	0.07 ksf	0.00 ksf	0.066 ksf	0.08 klf	1.25 klf	OK
3.33 ft	0.08 ksf	0.00 ksf	0.082 ksf	0.10 klf	1.25 klf	OK
4.00 ft	0.10 ksf	0.00 ksf	0.099 ksf	0.12 klf	1.25 klf	OK
4.67 ft	0.12 ksf	0.00 ksf	0.115 ksf	0.15 klf	1.25 klf	OK
5.33 ft	0.13 ksf	0.00 ksf	0.132 ksf	0.17 klf	1.25 klf	OK
6.00 ft	0.15 ksf	0.00 ksf	0.148 ksf	0.19 klf	1.25 klf	OK
6.67 ft	0.16 ksf	0.00 ksf	0.165 ksf	0.21 klf	1.25 klf	OK
7.33 ft	0.18 ksf	0.00 ksf	0.181 ksf	0.23 klf	1.25 klf	OK
8.00 ft	0.20 ksf	0.00 ksf	0.198 ksf	0.25 klf	1.25 klf	OK
8.67 ft	0.21 ksf	0.00 ksf	0.214 ksf	0.27 klf	1.25 klf	OK
9.33 ft	0.23 ksf	0.00 ksf	0.231 ksf	0.29 klf	1.25 klf	OK
10.00 ft	0.25 ksf	0.00 ksf	0.247 ksf	0.31 klf	1.25 klf	OK
10.67 ft	0.26 ksf	0.00 ksf	0.264 ksf	0.33 klf	1.25 klf	OK
11.33 ft	0.28 ksf	0.00 ksf	0.280 ksf	0.35 klf	1.25 klf	OK
12.00 ft	0.30 ksf	0.00 ksf	0.297 ksf	0.37 klf	1.25 klf	OK
12.67 ft	0.31 ksf	0.00 ksf	0.313 ksf	0.39 klf	1.25 klf	OK
13.33 ft	0.33 ksf	0.00 ksf	0.330 ksf	0.41 klf	1.25 klf	OK
14.00 ft	0.35 ksf	0.00 ksf	0.346 ksf	0.44 klf	1.25 klf	OK
14.67 ft	0.36 ksf	0.00 ksf	0.363 ksf	0.46 klf	1.25 klf	OK
15.33 ft	0.38 ksf	0.00 ksf	0.379 ksf	0.48 klf	1.25 klf	OK
15.62 ft	0.39 ksf	0.00 ksf	0.386 ksf	0.49 klf	1.25 klf	OK

FINAL DESIGN - GRS-IBS WINGWALL - ABUTMENT 3 - SWWW - STEP 2 VTRANS D37 IM 091-1(68)

References:

1) GRS-IBS Interim Implementation Guide, FHWA, June 2018
(2) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018
(3) Geotechnical Data Report, Teracon Consultants Inc., March 27. 2019

Material Properties

γ_f = 115.00 pcf (Reinforced Fill)
ϕ (Reinforced Soil) = 45 degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β = 26.57 degrees

K_a (Reinforced Soil) = 0.22 = $0.894 \times \dfrac{0.894 - (0.800 - 0.5)^{\wedge}\ 0.5}{0.894 + (0.800 - 0.5)^{\wedge}\ 0.5}$

Fill retained is in-situ soil.

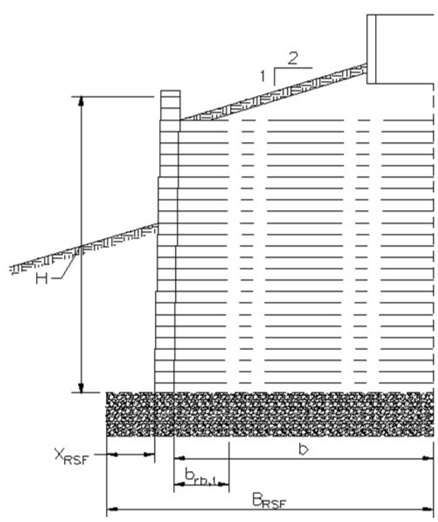
γ_b = 125.00 pcf (Retained Fill)
ϕ (Existing Filll) = 34.00 degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β = 26.57 degrees

K_a (Retained Fill) = 0.41 = $0.894 \times \dfrac{0.894 - (0.800 - 0.6873)^{\wedge}\ 0.5}{0.894 + (0.800 - 0.6873)^{\wedge}\ 0.5}$

Geometry



Depth of RSF, D =	2.5 ft		
Top of RSF EL. =	448.46		
Bottom of RSF EL. =	445.96 =	448.46 -	2.5 ft
Top of Wall EL. =	470.53		
Wall Height =	22.07 ft	470.53 -	448.46
H =	22.07 ft		
B =	11.25 ft		
Toe Length, X_{RSF} =	2.5 ft		
Total Width, B_{RSF} =	14.75 ft		
$b_{rb,t}$ =	0.00 ft		
Stepped Footing Length =	12.00 ft		
h_{block} =	8.00 in		
D_{block} =	12.00 in		
L_{block} =	18.00 in		
Weight =	85 lbs	per block	

Reinforcement

S_v = 0.67 ft
d_{max} = 0.75 in
T_f = 5.90 klf

Loading
Note that the superstructure dead loads, roadway bed loads, and traffic live loads are not applicable for this structure. No live load surcharge is applied.

Load Factors
Per (3) Table 3.4.1-1 and Table 3.4.1-2.

γ_p (DC, Min.) =	0.90	3) Table 3.4.1-2
γ_p (DC, Max.) =	1.25	3) Table 3.4.1-2
γ_p (EH, Max) =	1.50	3) Table 3.4.1-2
γ_p (EV, Min.) =	1.00	3) Table 3.4.1-2
γ_p (EV, Max.) =	1.35	3) Table 3.4.1-2
Factor (LS) =	1.75	3) Table 3.4.1-1

Calculate Vertical Loads and Applied Pressures
Calculate Traffic Surcharge, q_T

Due to wingwall layout geometry, assume that there is no live load surcharge acting on the wingwall.

h_{eq} = 0.00 ft
γ_r= 125.00 pcf

$$q_t = h_{eq}\gamma_b$$

q_T = 0.00 psf = 0.00 ft x 125.00 pcf

Calculate Weight of GRS Abutment

$$W = BH\gamma_r$$

H = 22.07 ft
B = 11.25 ft
γ_r= 115.00 pcf

W = 28557.54 plf = 22.07346 ft x 11.25 ft x 115.00 pcf

Calculate Weight of RSF

$$W_{RSF} = B_{RSF}D_{RSF}\gamma_r$$

B = 14.75 ft
D = 2.5 ft
γ_r= 125.00 pcf

W_{RSF} = 4240.63 plf = 14.75 ft x 2.5 ft x 115.00 pcf

Calculate Weight of Facing

N_{block} = 33.11 = 22.073457 ft/ 0.67 ft
N_{block} = 34

$$W_{face} = N_{block}\frac{W_{block}}{L_{block}}$$

W_{face} = 1926.67 plf = 34 x (85 lbs/ (18.00 in/ 12)

Calculate Weight of Surplus Retained Fill

Fill is retained at a 2H:1V slope

B = 11.25 ft
H_{max} = 5.63 ft = 11.25 ft/ 2
W_{SF} = 3955.08 plf = 0.5 x 11.25 ft x 5.63 ft x 125.00 pcf

Summary of Applied Loads

q_t = 0.00 psf
W = 28557.54 plf
W_{RSF} = 4240.63 plf
W_{face} = 1926.67 plf
W_{SF} = 3955.08 plf

Calculte Direct Sliding Effects at RSF/GRS Interface

Calculate Sliding per the (1) and (3) Strength I Load Combination

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2$$ (1) Eq. 9

$$F_t = q_t K_{ab} H$$ (1) Eq. 11

$$F_R = \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t$$ (1) Eq. 12

F_b = 12371.20 lbs = 0.5 x 125.00 pcf x 0.41 x 22.07346 ft^ 2
F_t = 0.00 lbs = 0.00 psf 0.41 x 22.0734572 ft

F_R = 18.56 klf = (1.5 x 12371.20 lbs) + (1.75 x 0.00 lbs))/ 1000

$$W_{T.R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EV\,MIN}(q_{rb} b_{rb,t}) + \gamma_{EV\,MIN} W_{SF}$$ (1) Eq. 14

$W_{T,R}$ = 34.25 klf = ((1.0 x 28557.54 plf) + ((0.9 x 1926.67 plf))/ 1000) + (1.00 x 3.96 klf)

Assume that μ = 2/3*tanϕ, per (1) C.1.6.1.1

μ = 0.67 = 0.67 x tan(45)

$$R_R = \Phi_\tau (W_{T,R}\mu)$$ (1) Eq. 13

R_R = 22.83 klf = 34.247 klf x 0.667

OK

Calculate Direct Sliding at Base of RSF

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2$$ (3) Eq. 17

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF})$$ (3) Eq. 19

$F_{b,\,RSF}$ =	15.33 klf =	(0.5 x	125.00 pcf x	0.41 x	(22.07 ft +	2.5)^ 2)/	1000
$F_{t,\,RSF}$ =	0.00 klf =	0.00 psf x	0.41 x	(22.07 ft +	2.5 ft))/	1000	

$$F_{R,RSF} = \gamma_{EH\,MAX}\left(F_{b,RSF} + F_{rb,RSF}\right) + \gamma_{LS}F_{t,RSF}$$ (1) Eq. 20

$F_{R,RSF}$ = 23.00 klf = (1.5 x x 15.33 klf) + (1.75 x 0.00 klf)

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\,MIN}W_{RSF}$$ (1) Eq. 22

$W_{T,R,RSF}$ = 38.49 klf = 34.25 klf + (1.0 x 4.24 klf)

$$R_{R,RSF} = \Phi_\tau(W_{T,R,RSF}\mu_{RSF})$$ (1) Eq. 21

Note that ϕ = 1.0, per (1) 4.3.6.1.2
Per (2), and the previous calculation, the angle of internal friction beneath the RSF is equal to 31 degrees

$R_{R,\,RSF}$ = 23.13 klf = 1.0 x 38.49 klf x tan(31)

OK

Calculate External Bearing Resistance

Calculate V_R

Per (1) Eq. 26,

$$\sum V_R = \gamma_{EV\,MAX}(W) + \gamma_{EV\,MAX}(W_{RSF}) + \gamma_{DC\,MAX}(W_{face}) + \gamma_{LS}(q_t b_{rb,t}) + \gamma_{EH\,MAX}(q_{rb} b_{rb,t}) + \gamma_{DC\,MAX}(q_{DL} b) + \gamma_{LS}(q_{LL} b)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\,MIN}$

W =	38.55 klf =	1.35 x	28557.54 plf/	1000	
W_{RSF} =	5.72 klf =	1.35 x	4240.63 plf/	1000	
W_{FACE} =	2.41 klf =	1.25 x	1926.67 plf/	1000	
W_{SF} =	5.34 klf =	1.35 x	3955.08 plf/	1000	
Q_t =	0.00 klf =	1.75 x	(0.00 psf x	0.00 ft)/	1000
Total =	52.03 klf				

Calculate $M_{D,R}$

Per (1) Eq. 28

$$\sum M_{D,R} = \gamma_{EH\,MAX}F_{b,RSF}\left(\frac{H + D_{RSF}}{3}\right) + \gamma_{LS}F_{t,RSF}\left(\frac{H + D_{RSF}}{2}\right) + \gamma_{EH\,MAX}F_{rb,RSF}\left(\frac{H + D_{RSF}}{2}\right)$$

$F_{b,\,RSF}$ =	188.38 k-ft/ ft =	1.5 x	15.33 klf x	(0.33 x	(22.0735 ft +	2.5 ft))
$F_{t,\,RSF}$ =	0.00 k-ft/ ft =	1.75 x	0.00 klf x	(0.50 x	(22.0735 ft +	2.5 ft))
Total =	188.38 k-ft/ ft					

Note that $M_{D,R}$ is taken about the bottom center of the width of the RSF.

Calculate $M_{R,R}$

Per (1) Eq. 29

$$\sum M_{R,R} = (\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b)\left[\left(\frac{b}{2}+a_b\right)-\left(\frac{B_{RSF}}{2}-x_{RSF}-b_{block}\right)\right]+ \left(\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t}\right)\left(\frac{B_{RSF}}{2}-\frac{b_{rb}}{2}\right)+\gamma_{EV\,MAX} W\left(\frac{B_{RSF}}{2}-\frac{B}{2}\right)+ \gamma_{DC\,MAX} W_{face}\left(B+\frac{b_{block}}{2}-\frac{B_{RSF}}{2}\right)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\,MAX}$. Resisting component = W_{SF}*((X_{RSF}+D_{block}+(2/3)*B)-BRSF/2)

W =	67.47 k-ft =	1.35 x	28.56 klf x	(7.375 ft -	5.625 ft)			
W_{Face} =	-11.38 k-ft =	1.35 x	1.93 klf x	((2.5 ft +	0.5 ft) -	7.375 ft)		
W_{SF} =	19.36 k-ft =	1.35 x	3.96 klf x	((2.5 ft +	1 ft +	(0.67 x	11.25 ft)) -	(14.75 ft/ 2))
Total =	75.44 k-ft/ ft							

Note that $M_{R,R}$ is taken about the bottom center of the width of the RSF.

Calculate Eccentricity, $e_{B,R}$

Per (1) Eq. 27

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R}$$

$e_{B,R}$ = 2.17 ft = $\frac{188.38 \text{ k-ft/ ft} - 75.44 \text{ k-ft/ ft}}{52.03 \text{ klf}}$

Calculate $\sigma_{v,base,R}$

Since $e_{B,R}$ = negative, set equal to zero in calcaulation of base bearing pressure.

Per (1) Eq. 25,

$$\sigma_{v,base,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}}$$

$\sigma_{v,base,R}$ = 5.00 ksf = 52.03 klf/ (14.75 ft - (2 x 2.17 ft)

Per abutment bearing capacity calculation, allowable bearing capacity = 9.15 ksf

OK

Calculate Reinforcement Strength

q_t = 0.00 psf
W = 28557.54 psf
W_{RSF} = 4240.63 psf
W_{face} = 1926.67 psf

The evaulation of the abutment for the strength limit state is conducted according to (1) 4.3.7.3.1. and using the following equations:

Required Reinforcment:

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 40

Factored Total Lateral Pressure:

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f}$$ (3) Eq. 41

Where:

$$\sigma_{h,W,f} = \gamma_{EH\ MAX}\left(\gamma_r z K_{ar}\right)$$ (3) Eq. 42

Where:
$\gamma_{EH\ MAX}$ = maximum horizontal earth pressure load factor.
γ_r = unit weight of reinforced backfill.
z = depth from the top of the wall.
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar}$$ (3) Eq. 45

Note that $\sigma_{hbridge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Factored Reinforcement Strength:

$$T_{f,f} = \Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right) = 0.4T_f$$ (3) Eq. 48

Strength Limit State

z	$\sigma_{h,W,f}$	$\sigma_{h,t,f}$	$\sigma_{h,f}$	$T_{req,f}$	$T_{f,f}$	
0.67 ft	0.02 ksf	0.00 ksf	0.02 ksf	0.03 ksf	2.36 klf	OK
1.33 ft	0.05 ksf	0.00 ksf	0.05 ksf	0.06 ksf	2.36 klf	OK
2.00 ft	0.07 ksf	0.00 ksf	0.07 ksf	0.09 ksf	2.36 klf	OK
2.67 ft	0.10 ksf	0.00 ksf	0.10 ksf	0.12 ksf	2.36 klf	OK
3.33 ft	0.12 ksf	0.00 ksf	0.12 ksf	0.16 ksf	2.36 klf	OK
4.00 ft	0.15 ksf	0.00 ksf	0.15 ksf	0.19 ksf	2.36 klf	OK
4.67 ft	0.17 ksf	0.00 ksf	0.17 ksf	0.22 ksf	2.36 klf	OK
5.33 ft	0.20 ksf	0.00 ksf	0.20 ksf	0.25 ksf	2.36 klf	OK
6.00 ft	0.22 ksf	0.00 ksf	0.22 ksf	0.28 ksf	2.36 klf	OK
6.67 ft	0.25 ksf	0.00 ksf	0.25 ksf	0.31 ksf	2.36 klf	OK
7.33 ft	0.27 ksf	0.00 ksf	0.27 ksf	0.34 ksf	2.36 klf	OK
8.00 ft	0.30 ksf	0.00 ksf	0.30 ksf	0.37 ksf	2.36 klf	OK
8.67 ft	0.32 ksf	0.00 ksf	0.32 ksf	0.40 ksf	2.36 klf	OK
9.33 ft	0.35 ksf	0.00 ksf	0.35 ksf	0.44 ksf	2.36 klf	OK
10.00 ft	0.37 ksf	0.00 ksf	0.37 ksf	0.47 ksf	2.36 klf	OK
10.67 ft	0.40 ksf	0.00 ksf	0.40 ksf	0.50 ksf	2.36 klf	OK
11.33 ft	0.42 ksf	0.00 ksf	0.42 ksf	0.53 ksf	2.36 klf	OK
12.00 ft	0.45 ksf	0.00 ksf	0.45 ksf	0.56 ksf	2.36 klf	OK
12.67 ft	0.47 ksf	0.00 ksf	0.47 ksf	0.59 ksf	2.36 klf	OK
13.33 ft	0.49 ksf	0.00 ksf	0.49 ksf	0.62 ksf	2.36 klf	OK
14.00 ft	0.52 ksf	0.00 ksf	0.52 ksf	0.65 ksf	2.36 klf	OK
14.67 ft	0.54 ksf	0.00 ksf	0.54 ksf	0.68 ksf	2.36 klf	OK
15.33 ft	0.57 ksf	0.00 ksf	0.57 ksf	0.71 ksf	2.36 klf	OK
16.00 ft	0.59 ksf	0.00 ksf	0.59 ksf	0.75 ksf	2.36 klf	OK
16.67 ft	0.62 ksf	0.00 ksf	0.62 ksf	0.78 ksf	2.36 klf	OK
17.33 ft	0.64 ksf	0.00 ksf	0.64 ksf	0.81 ksf	2.36 klf	OK
18.00 ft	0.67 ksf	0.00 ksf	0.67 ksf	0.84 ksf	2.36 klf	OK
18.67 ft	0.69 ksf	0.00 ksf	0.69 ksf	0.87 ksf	2.36 klf	OK
22.07 ft	0.82 ksf	0.00 ksf	0.82 ksf	1.03 ksf	2.36 klf	OK

The evaulation of the abutment for the service limit state is conducted according to (1) 4.3.7.3.2. and using the following equations:

The nominal required reinforcement strength:

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 50

The lateral pressure within the GRS Abutment at a given dpeth and location:

$$\sigma_h = \sigma_{h,W} + \sigma_{h,bridge,eq} + \sigma_{h,rb} + \sigma_{h,t}$$ (3) Eq. 51

Where:

$$\sigma_{h,W} = \gamma_r z K_{ar}$$ (3) Eq. 2

$$\sigma_{h,t} = q_t K_{ar}$$ (3) Eq. 54

Note that $\sigma_{hbrdge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

FINAL DESIGN - GRS-IBS WINGWALL - ABUTMENT 3 - SWWW - STEP 2 VTRANS D37 IM 091-1(68)

Service Limit State

z	$\sigma_{h,W}$	$\sigma_{h,t}$	σ_h	T_{req}	$T_{@\varepsilon = 2\%}$	
0.67 ft	0.02 ksf	0.00 ksf	0.016 ksf	0.02 klf	1.25 klf	OK
1.33 ft	0.03 ksf	0.00 ksf	0.033 ksf	0.04 klf	1.25 klf	OK
2.00 ft	0.05 ksf	0.00 ksf	0.049 ksf	0.06 klf	1.25 klf	OK
2.67 ft	0.07 ksf	0.00 ksf	0.066 ksf	0.08 klf	1.25 klf	OK
3.33 ft	0.08 ksf	0.00 ksf	0.082 ksf	0.10 klf	1.25 klf	OK
4.00 ft	0.10 ksf	0.00 ksf	0.099 ksf	0.12 klf	1.25 klf	OK
4.67 ft	0.12 ksf	0.00 ksf	0.115 ksf	0.15 klf	1.25 klf	OK
5.33 ft	0.13 ksf	0.00 ksf	0.132 ksf	0.17 klf	1.25 klf	OK
6.00 ft	0.15 ksf	0.00 ksf	0.148 ksf	0.19 klf	1.25 klf	OK
6.67 ft	0.16 ksf	0.00 ksf	0.165 ksf	0.21 klf	1.25 klf	OK
7.33 ft	0.18 ksf	0.00 ksf	0.181 ksf	0.23 klf	1.25 klf	OK
8.00 ft	0.20 ksf	0.00 ksf	0.198 ksf	0.25 klf	1.25 klf	OK
8.67 ft	0.21 ksf	0.00 ksf	0.214 ksf	0.27 klf	1.25 klf	OK
9.33 ft	0.23 ksf	0.00 ksf	0.231 ksf	0.29 klf	1.25 klf	OK
10.00 ft	0.25 ksf	0.00 ksf	0.247 ksf	0.31 klf	1.25 klf	OK
10.67 ft	0.26 ksf	0.00 ksf	0.264 ksf	0.33 klf	1.25 klf	OK
11.33 ft	0.28 ksf	0.00 ksf	0.280 ksf	0.35 klf	1.25 klf	OK
12.00 ft	0.30 ksf	0.00 ksf	0.297 ksf	0.37 klf	1.25 klf	OK
12.67 ft	0.31 ksf	0.00 ksf	0.313 ksf	0.39 klf	1.25 klf	OK
13.33 ft	0.33 ksf	0.00 ksf	0.330 ksf	0.41 klf	1.25 klf	OK
14.00 ft	0.35 ksf	0.00 ksf	0.346 ksf	0.44 klf	1.25 klf	OK
14.67 ft	0.36 ksf	0.00 ksf	0.363 ksf	0.46 klf	1.25 klf	OK
15.33 ft	0.38 ksf	0.00 ksf	0.379 ksf	0.48 klf	1.25 klf	OK
16.00 ft	0.40 ksf	0.00 ksf	0.396 ksf	0.50 klf	1.25 klf	OK
16.67 ft	0.41 ksf	0.00 ksf	0.412 ksf	0.52 klf	1.25 klf	OK
17.33 ft	0.43 ksf	0.00 ksf	0.429 ksf	0.54 klf	1.25 klf	OK
18.00 ft	0.45 ksf	0.00 ksf	0.445 ksf	0.56 klf	1.25 klf	OK
18.67 ft	0.46 ksf	0.00 ksf	0.462 ksf	0.58 klf	1.25 klf	OK
22.07 ft	0.55 ksf	0.00 ksf	0.546 ksf	0.69 klf	1.25 klf	OK

FINAL DESIGN - GRS-IBS WINGWALL - ABUTMENT 4 - NEWW - STEP 1 VTRANS D37 IM 091-1(68)

References:

1) GRS-IBS Interim Implementation Guide, FHWA, June 2018
(2) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018
(3) Geotechnical Data Report, Teracon Consultants Inc., March 27. 2019

Material Properties

γ_f = 115.00 pcf (Reinforced Fill)
ϕ (Reinforced Soil)= 45 degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β = 26.57 degrees

K_a (Reinforced Soil) = 0.22 = 0.894 x $\dfrac{0.894 - (0.800 - 0.5)^\wedge 0.5}{0.894 + (0.800 - 0.5)^\wedge 0.5}$

Fill retained is in-situ soil.

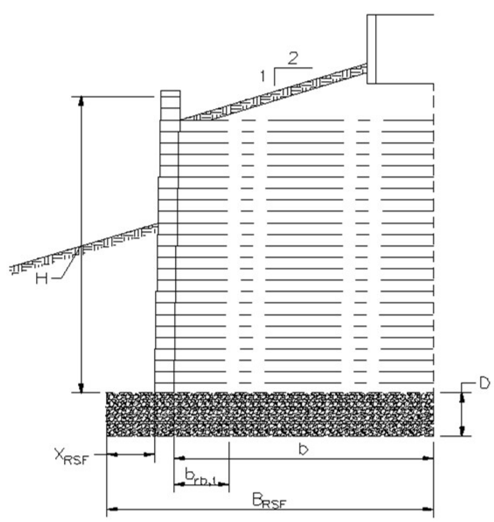
γ_b = 125.00 pcf (Retained Fill)
ϕ (Existing Filll)= 34.00 degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β = 26.57 degrees

K_a (Retained Fill) = 0.41 = 0.894 x $\dfrac{0.894 - (0.800 - 0.6873)^\wedge 0.5}{0.894 + (0.800 - 0.6873)^\wedge 0.5}$

Geometry



Depth of RSF, D = 2.5 ft
Top of RSF EL. = 449.96
Bottom of RSF EL. = 447.46 = 449.96 - 2.5 ft

Top of Wall EL. = 464.79
Wall Height = 14.83 ft 464.79 - 449.96
H = 14.83 ft

B = 7.5 ft
Toe Length, X_{RSF} = 2.5 ft
Total Width, B_{RSF} = 11 ft
$b_{rb,t}$ = 0.00 ft
Stepped Footing Length = 12.00 ft

h_{block} = 8.00 in
D_{block} = 12.00 in
L_{block} = 18.00 in
Weight = 85 lbs per block

Reinforcement

S_v = 0.67 ft
d_{max} = 0.75 in
T_f = 5.90 klf

Loading

Note that the superstructure dead loads, roadway bed loads, and traffic live loads are not applicable for this structure. No live load surcharge is applied.

Load Factors

Per (3) Table 3.4.1-1 and Table 3.4.1-2.

γ_p (DC, Min.) =	0.90	3) Table 3.4.1-2
γ_p (DC, Max.) =	1.25	3) Table 3.4.1-2
γ_p (EH, Max) =	1.50	3) Table 3.4.1-2
γ_p (EV, Min.) =	1.00	3) Table 3.4.1-2
γ_p (EV, Max.) =	1.35	3) Table 3.4.1-2
Factor (LS) =	1.75	3) Table 3.4.1-1

Calculate Vertical Loads and Applied Pressures

Calculate Traffic Surcharge, q_T

Due to wingwall layout geometry, assume that there is no live load surcharge acting on the wingwall.

h_{eq} = 0.00 ft
γ_r= 125.00 pcf

$$q_t = h_{eq}\gamma_b$$

q_T = 0.00 psf = 0.00 ft x 125.00 pcf

Calculate Weight of GRS Abutment

$$W = BH\gamma_r$$

H = 14.83 ft
B = 7.5 ft
γ_r= 115.00 pcf

W = 12787.55 plf = 14.82615 ft x 7.5 ft x 115.00 pcf

Calculate Weight of RSF

$$W_{RSF} = B_{RSF}D_{RSF}\gamma_r$$

B = 11 ft
D = 2.5 ft
γ_r= 125.00 pcf

W_{RSF} = 3162.50 plf = 11 ft x 2.5 ft x 115.00 pcf

Calculate Weight of Facing

N_{block} = 22.24 = 14.826151 ft/ 0.67 ft
N_{block} = 23

$$W_{face} = N_{block}\frac{W_{block}}{L_{block}}$$

W_{face} = 1303.33 plf = 23 x (85 lbs/ (18.00 in/ 12)

Calculate Weight of Surplus Retained Fill

Fill is retained at a 2H:1V slope

B = 7.5 ft
H_{max} = 3.75 ft = 7.5 ft/ 2
W_{SF} = 1757.81 plf = 0.5 x 7.5 ft x 3.75 ft x 125.00 pcf

Summary of Applied Loads

q_t =	0.00	psf
W =	12787.55	plf
W_{RSF} =	3162.50	plf
W_{face} =	1303.33	plf
W_{SF} =	1757.81	plf

Calculte Direct Sliding Effects at RSF/GRS Interface

Calculate Sliding per the (1) and (3) Strength I Load Combination

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2$$ (1) Eq. 9

$$F_t = q_t K_{ab} H$$ (1) Eq. 11

$$F_R = \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t$$ (1) Eq. 12

F_b = 5581.20 lbs = 0.5 x 125.00 pcf x 0.41 x 14.82615 ft^ 2

F_t = 0.00 lbs = 0.00 psf 0.41 x 14.8261507 ft

F_R = 8.37 klf = (1.5 x 5581.20 lbs) + (1.75 x 0.00 lbs))/ 1000

$$W_{T,R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EV\,MIN}(q_{rb} b_{rb,t}) + \gamma_{EV\,MIN} W_{SF}$$ (1) Eq. 14

$W_{T,R}$ = 15.72 klf = ((1.0 x 12787.55 plf) + ((0.9 x 1303.33 plf))/ 1000) + (1.00 x 1.76 klf)

Assume that μ = 2/3*tanϕ, per (1) C.1.6.1.1

μ = 0.67 = 0.67 x tan(45)

$$R_R = \Phi_\tau (W_{T,R}\mu)$$ (1) Eq. 13

R_R = 10.48 klf = 15.718 klf x 0.667

OK

FINAL DESIGN - GRS-IBS WINGWALL - ABUTMENT 4 - NEWW - STEP 1 VTRANS D37 IM 091-1(68)

Calculate Direct Sliding at Base of RSF

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2$$ (3) Eq. 17

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF})$$ (3) Eq. 19

$F_{b,\ RSF}$ =	7.62 klf =	(0.5 x	125.00 pcf x	0.41 x	(14.83 ft +	2.5)^ 2)/	1000
$F_{t,\ RSF}$ =	0.00 klf =	0.00 psf x	0.41 x	(14.83 ft +	2.5 ft))/	1000	

$$F_{R,RSF} = \gamma_{EH\ MAX}(F_{b,RSF} + F_{rb,RSF}) + \gamma_{LS}F_{t,RSF}$$ (1) Eq. 20

$F_{R,RSF}$ = 11.43 klf = (1.5 x x 7.62 klf) + (1.75 x 0.00 klf)

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\ MIN}W_{RSF}$$ (1) Eq. 22

$W_{T,R,RSF}$ = 18.88 klf = 15.72 klf + (1.0 x 3.16 klf)

$$R_{R,RSF} = \Phi_\tau(W_{T,R,RSF}\mu_{RSF})$$ (1) Eq. 21

Note that ϕ = 1.0, per (1) 4.3.6.1.2

Per (2), and the previous calculation, the angle of internal friction beneath the RSF is equal to 32 degrees

$R_{R,\ RSF}$ = 11.80 klf = 1.0 x 18.88 klf x tan(32)

OK

Calculate External Bearing Resistance

Calculate V_R

Per (1) Eq. 26,

$$\sum V_R = \gamma_{EV\ MAX}(W) + \gamma_{EV\ MAX}(W_{RSF}) + \gamma_{DC\ MAX}(W_{face}) + \gamma_{LS}(q_t b_{rb,t}) + \gamma_{EH\ MAX}(q_{rb} b_{rb,t}) + \gamma_{DC\ MAX}(q_{DL} b) + \gamma_{LS}(q_{LL} b)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\ MIN}$

W =	17.26 klf =	1.35 x	12787.55 plf/	1000	
W_{RSF} =	4.27 klf =	1.35 x	3162.50 plf/	1000	
W_{FACE} =	1.63 klf =	1.25 x	1303.33 plf/	1000	
W_{SF} =	2.37 klf =	1.35 x	1757.81 plf/	1000	
Q_t =	0.00 klf =	1.75 x	(0.00 psf x	0.00 ft)/	1000
Total =	25.53 klf				

Calculate $M_{D,R}$

Per (1) Eq. 28

$$\sum M_{D,R} = \gamma_{EH\ MAX}F_{b,RSF}\left(\frac{H + D_{RSF}}{3}\right) + \gamma_{LS}F_{t,RSF}\left(\frac{H + D_{RSF}}{2}\right) + \gamma_{EH\ MAX}F_{rb,RSF}\left(\frac{H + D_{RSF}}{2}\right)$$

$F_{b,\ RSF}$ =	66.03 k-ft/ ft =	1.5 x	7.62 klf x	(0.33 x	(14.8262 ft +	2.5 ft))
$F_{t,\ RSF}$ =	0.00 k-ft/ ft =	1.75 x	0.00 klf x	(0.50 x	(14.8262 ft +	2.5 ft))
Total =	66.03 k-ft/ ft					

Note that $M_{D,R}$ is taken about the bottom center of the width of the RSF.

Calculate $M_{R,R}$

Per (1) Eq. 29

$$\sum M_{R,R} = (\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b)\left[\left(\frac{b}{2} + a_b\right) - \left(\frac{B_{RSF}}{2} - x_{RSF} - b_{block}\right)\right] + \left(\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t}\right)\left(\frac{B_{RSF}}{2} - \frac{b_{rb}}{2}\right) + \gamma_{EV\,MAX} W\left(\frac{B_{RSF}}{2} - \frac{B}{2}\right) + \gamma_{DC\,MAX} W_{face}\left(B + \frac{b_{block}}{2} - \frac{B_{RSF}}{2}\right)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\,MAX}$. Resisting component = W_{SF}*((X_{RSF}+D_{block}+(2/3)*B)-BRSF/2)

W =	30.21 k-ft =	1.35 x	12.79 klf x	(5.5 ft -	3.75 ft)				
W_{Face} =	-4.40 k-ft =	1.35 x	1.30 klf x	((2.5 ft +	0.5 ft) -	5.5 ft)			
W_{SF} =	7.12 k-ft =	1.35 x	1.76 klf x	((2.5 ft +	1 ft +	(0.67 x	7.5 ft)) -	(11 ft/ 2))	
Total =	32.93 k-ft/ ft								

Note that $M_{R,R}$ is taken about the bottom center of the width of the RSF.

Calculate Eccentricity, $e_{B,R}$

Per (1) Eq. 27

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R}$$

$e_{B,R}$ = 1.30 ft = $\frac{66.03 \text{ k-ft/ ft} - 32.93 \text{ k-ft/ ft}}{25.53 \text{ klf}}$

Calculate $\sigma_{v,base,R}$

Since $e_{B,R}$ = negative, set equal to zero in calcaulation of base bearing pressure.

Per (1) Eq. 25,

$$\sigma_{v,base,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}}$$

$\sigma_{v,base,R}$ = 3.04 ksf = 25.53 klf/ (11 ft - (2 x 1.30 ft)

Per abutment bearing capacity calculation, allowable bearing capacity = 11.88 ksf

OK

Calculate Reinforcement Strength

q_t = 0.00 psf
W = 12787.55 psf
W_{RSF} = 3162.50 psf
W_{face} = 1303.33 psf

The evaulation of the abutment for the strength limit state is conducted according to (1) 4.3.7.3.1. and using the following equations:

Required Reinforcment:

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 40

Factored Total Lateral Pressure:

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f}$$ (3) Eq. 41

Where:

$$\sigma_{h,W,f} = \gamma_{EH\,MAX}(\gamma_r z K_{ar})$$ (3) Eq. 42

Where:

$\gamma_{EH\,MAX}$ = maximum horizontal earth pressure load factor.
γ_r = unit weight of reinforced backfill.
z = depth from the top of the wall.
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar}$$ (3) Eq. 45

Note that $\sigma_{hbridge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Factored Reinforcement Strength:

$$T_{f,f} = \Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right) = 0.4T_f$$ (3) Eq. 48

Strength Limit State

z	$\sigma_{h,W,f}$	$\sigma_{h,t,f}$	$\sigma_{h,f}$	$T_{req,f}$	$T_{f,f}$	
0.67 ft	0.02 ksf	0.00 ksf	0.02 ksf	0.03 ksf	2.36 klf	OK
1.33 ft	0.05 ksf	0.00 ksf	0.05 ksf	0.06 ksf	2.36 klf	OK
2.00 ft	0.07 ksf	0.00 ksf	0.07 ksf	0.09 ksf	2.36 klf	OK
2.67 ft	0.10 ksf	0.00 ksf	0.10 ksf	0.12 ksf	2.36 klf	OK
3.33 ft	0.12 ksf	0.00 ksf	0.12 ksf	0.16 ksf	2.36 klf	OK
4.00 ft	0.15 ksf	0.00 ksf	0.15 ksf	0.19 ksf	2.36 klf	OK
4.67 ft	0.17 ksf	0.00 ksf	0.17 ksf	0.22 ksf	2.36 klf	OK
5.33 ft	0.20 ksf	0.00 ksf	0.20 ksf	0.25 ksf	2.36 klf	OK
6.00 ft	0.22 ksf	0.00 ksf	0.22 ksf	0.28 ksf	2.36 klf	OK
6.67 ft	0.25 ksf	0.00 ksf	0.25 ksf	0.31 ksf	2.36 klf	OK
7.33 ft	0.27 ksf	0.00 ksf	0.27 ksf	0.34 ksf	2.36 klf	OK
8.00 ft	0.30 ksf	0.00 ksf	0.30 ksf	0.37 ksf	2.36 klf	OK
8.67 ft	0.32 ksf	0.00 ksf	0.32 ksf	0.40 ksf	2.36 klf	OK
9.33 ft	0.35 ksf	0.00 ksf	0.35 ksf	0.44 ksf	2.36 klf	OK
10.00 ft	0.37 ksf	0.00 ksf	0.37 ksf	0.47 ksf	2.36 klf	OK
10.67 ft	0.40 ksf	0.00 ksf	0.40 ksf	0.50 ksf	2.36 klf	OK
11.33 ft	0.42 ksf	0.00 ksf	0.42 ksf	0.53 ksf	2.36 klf	OK
12.00 ft	0.45 ksf	0.00 ksf	0.45 ksf	0.56 ksf	2.36 klf	OK
12.67 ft	0.47 ksf	0.00 ksf	0.47 ksf	0.59 ksf	2.36 klf	OK
13.33 ft	0.49 ksf	0.00 ksf	0.49 ksf	0.62 ksf	2.36 klf	OK
14.00 ft	0.52 ksf	0.00 ksf	0.52 ksf	0.65 ksf	2.36 klf	OK
14.67 ft	0.54 ksf	0.00 ksf	0.54 ksf	0.68 ksf	2.36 klf	OK
14.83 ft	0.55 ksf	0.00 ksf	0.55 ksf	0.69 ksf	2.36 klf	OK

The evaulation of the abutment for the service limit state is conducted according to (1) 4.3.7.3.2. and using the following equations:

The nominal required reinforcement strength:

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 50

The lateral pressure within the GRS Abutment at a given dpeth and location:

$$\sigma_h = \sigma_{h,W} + \sigma_{h,bridge,eq} + \sigma_{h,rb} + \sigma_{h,t}$$ (3) Eq. 51

Where:

$$\sigma_{h,W} = \gamma_r z K_{ar}$$ (3) Eq. 2

$$\sigma_{h,t} = q_t K_{ar}$$ (3) Eq. 54

Note that $\sigma_{hbrdge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Service Limit State

z	$\sigma_{h,W}$	$\sigma_{h,t}$	σ_h	T_{req}	$T_{@\varepsilon = 2\%}$	
0.67 ft	0.02 ksf	0.00 ksf	0.016 ksf	0.02 klf	1.25 klf	OK
1.33 ft	0.03 ksf	0.00 ksf	0.033 ksf	0.04 klf	1.25 klf	OK
2.00 ft	0.05 ksf	0.00 ksf	0.049 ksf	0.06 klf	1.25 klf	OK
2.67 ft	0.07 ksf	0.00 ksf	0.066 ksf	0.08 klf	1.25 klf	OK
3.33 ft	0.08 ksf	0.00 ksf	0.082 ksf	0.10 klf	1.25 klf	OK
4.00 ft	0.10 ksf	0.00 ksf	0.099 ksf	0.12 klf	1.25 klf	OK
4.67 ft	0.12 ksf	0.00 ksf	0.115 ksf	0.15 klf	1.25 klf	OK
5.33 ft	0.13 ksf	0.00 ksf	0.132 ksf	0.17 klf	1.25 klf	OK
6.00 ft	0.15 ksf	0.00 ksf	0.148 ksf	0.19 klf	1.25 klf	OK
6.67 ft	0.16 ksf	0.00 ksf	0.165 ksf	0.21 klf	1.25 klf	OK
7.33 ft	0.18 ksf	0.00 ksf	0.181 ksf	0.23 klf	1.25 klf	OK
8.00 ft	0.20 ksf	0.00 ksf	0.198 ksf	0.25 klf	1.25 klf	OK
8.67 ft	0.21 ksf	0.00 ksf	0.214 ksf	0.27 klf	1.25 klf	OK
9.33 ft	0.23 ksf	0.00 ksf	0.231 ksf	0.29 klf	1.25 klf	OK
10.00 ft	0.25 ksf	0.00 ksf	0.247 ksf	0.31 klf	1.25 klf	OK
10.67 ft	0.26 ksf	0.00 ksf	0.264 ksf	0.33 klf	1.25 klf	OK
11.33 ft	0.28 ksf	0.00 ksf	0.280 ksf	0.35 klf	1.25 klf	OK
12.00 ft	0.30 ksf	0.00 ksf	0.297 ksf	0.37 klf	1.25 klf	OK
12.67 ft	0.31 ksf	0.00 ksf	0.313 ksf	0.39 klf	1.25 klf	OK
13.33 ft	0.33 ksf	0.00 ksf	0.330 ksf	0.41 klf	1.25 klf	OK
14.00 ft	0.35 ksf	0.00 ksf	0.346 ksf	0.44 klf	1.25 klf	OK
14.67 ft	0.36 ksf	0.00 ksf	0.363 ksf	0.46 klf	1.25 klf	OK
14.83 ft	0.37 ksf	0.00 ksf	0.367 ksf	0.46 klf	1.25 klf	OK

FINAL DESIGN - GRS-IBS WINGWALL - ABUTMENT 4 - NEWW - STEP 2 VTRANS D37 IM 091-1(68)

References:

1) GRS-IBS Interim Implementation Guide, FHWA, June 2018
(2) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018
(3) Geotechnical Data Report, Teracon Consultants Inc., March 27. 2019

Material Properties

γ_f = 115.00 pcf (Reinforced Fill)
ϕ (Reinforced Soil) = 45 degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β = 26.57 degrees

K_a (Reinforced Soil) = 0.22 = $0.894 \times \dfrac{0.894 - (0.800 - 0.5)^\wedge 0.5}{0.894 + (0.800 - 0.5)^\wedge 0.5}$

Fill retained is in-situ soil.

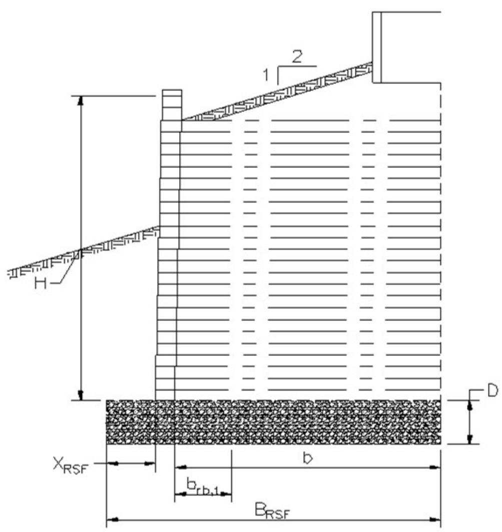
γ_b = 125.00 pcf (Retained Fill)
ϕ (Existing Filll) = 34.00 degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β = 26.57 degrees

K_a (Retained Fill) = 0.41 = $0.894 \times \dfrac{0.894 - (0.800 - 0.6873)^\wedge 0.5}{0.894 + (0.800 - 0.6873)^\wedge 0.5}$

Geometry



Depth of RSF, D = 2.5 ft
Top of RSF EL. = 447.46
Bottom of RSF EL. = 444.96 = 447.46 - 2.5 ft

Top of Wall EL. = 469.23
Wall Height = 21.77 ft 469.23 - 447.46
H = 21.77 ft

B = 10.75 ft
Toe Length, X_{RSF} = 2.5 ft
Total Width, B_{RSF} = 14.25 ft
$b_{rb,t}$ = 0.00 ft
Stepped Footing Length = 12.00 ft

h_{block} = 8.00 in
D_{block} = 12.00 in
L_{block} = 18.00 in
Weight = 85 lbs per block

Reinforcement

S_v = 0.67 ft
d_{max} = 0.75 in
T_f = 5.90 klf

Loading
Note that the superstructure dead loads, roadway bed loads, and traffic live loads are not applicable for this structure. No live load surcharge is applied.

Load Factors
Per (3) Table 3.4.1-1 and Table 3.4.1-2.

γ_p (DC, Min.) =	0.90	3) Table 3.4.1-2
γ_p (DC, Max.) =	1.25	3) Table 3.4.1-2
γ_p (EH, Max) =	1.50	3) Table 3.4.1-2
γ_p (EV, Min.) =	1.00	3) Table 3.4.1-2
γ_p (EV, Max.) =	1.35	3) Table 3.4.1-2
Factor (LS) =	1.75	3) Table 3.4.1-1

Calculate Vertical Loads and Applied Pressures
Calculate Traffic Surcharge, q_T

Due to wingwall layout geometry, assume that there is no live load surcharge acting on the wingwall.

h_{eq} = 0.00 ft
γ_r= 125.00 pcf

$$q_t = h_{eq}\gamma_b$$

q_T = 0.00 psf = 0.00 ft x 125.00 pcf

Calculate Weight of GRS Abutment

$$W = BH\gamma_r$$

H = 21.77 ft
B = 10.75 ft
γ_r= 115.00 pcf

W = 26918.37 plf = 21.77422 ft x 10.75 ft x 115.00 pcf

Calculate Weight of RSF

$$W_{RSF} = B_{RSF}D_{RSF}\gamma_r$$

B = 14.25 ft
D = 2.5 ft
γ_r= 125.00 pcf

W_{RSF} = 4096.88 plf = 14.25 ft x 2.5 ft x 115.00 pcf

Calculate Weight of Facing

N_{block} = 32.66 = 21.774216 ft/ 0.67 ft
N_{block} = 33

$$W_{face} = N_{block}\frac{W_{block}}{L_{block}}$$

W_{face} = 1870.00 plf = 33 x (85 lbs/ (18.00 in/ 12)

Calculate Weight of Surplus Retained Fill

Fill is retained at a 2H:1V slope

B = 10.75 ft
H_{max} = 5.38 ft = 10.75 ft/ 2
W_{SF} = 3611.33 plf = 0.5 x 10.75 ft x 5.38 ft x 125.00 pcf

Summary of Applied Loads

q_t = 0.00 psf
W = 26918.37 plf
W_{RSF} = 4096.88 plf
W_{face} = 1870.00 plf
W_{SF} = 3611.33 plf

Calculte Direct Sliding Effects at RSF/GRS Interface

Calculate Sliding per the (1) and (3) Strength I Load Combination

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2$$ (1) Eq. 9

$$F_t = q_t K_{ab} H$$ (1) Eq. 11

$$F_R = \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t$$ (1) Eq. 12

F_b = 12038.05 lbs = 0.5 x 125.00 pcf x 0.41 x 21.77422 ft^ 2
F_t = 0.00 lbs = 0.00 psf 0.41 x 21.7742159 ft

F_R = 18.06 klf = (1.5 x 12038.05 lbs) + (1.75 x 0.00 lbs))/ 1000

$$W_{T,R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EV\,MIN}(q_{rb} b_{rb,t}) + \gamma_{EV\,MIN} W_{SF}$$ (1) Eq. 14

$W_{T,R}$ = 32.21 klf = ((1.0 x 26918.37 plf) + ((0.9 x 1870.00 plf))/ 1000) + (1.00 x 3.61 klf)

Assume that μ = 2/3*tanϕ, per (1) C.1.6.1.1

μ = 0.67 = 0.67 x tan(45)

$$R_R = \Phi_\tau (W_{T,R}\mu)$$ (1) Eq. 13

R_R = 21.48 klf = 32.213 klf x 0.667

OK

Calculate Direct Sliding at Base of RSF

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2$$ (3) Eq. 17

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF})$$ (3) Eq. 19

$F_{b,\ RSF}$ =	14.96 klf =	(0.5 x	125.00 pcf x	0.41 x	(21.77 ft +	2.5)^ 2)/	1000
$F_{t,\ RSF}$ =	0.00 klf =	0.00 psf x	0.41 x	(21.77 ft +	2.5 ft))/	1000	

$$F_{R,RSF} = \gamma_{EH\ MAX}\left(F_{b,RSF} + F_{rb,RSF}\right) + \gamma_{LS}F_{t,RSF}$$ (1) Eq. 20

$F_{R,RSF}$ = 22.44 klf = (1.5 x x 14.96 klf) + (1.75 x 0.00 klf)

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\ MIN}W_{RSF}$$ (1) Eq. 22

$W_{T,R,RSF}$ = 36.31 klf = 32.21 klf + (1.0 x 4.10 klf)

$$R_{R,RSF} = \Phi_{\tau}(W_{T,R,RSF}\mu_{RSF})$$ (1) Eq. 21

Note that ϕ = 1.0, per (1) 4.3.6.1.2
Per (2), and the previous calculation, the angle of internal friction beneath the RSF is equal to 32 degrees

$R_{R,\ RSF}$ = 22.69 klf = 1.0 x 36.31 klf x tan(32)

OK

Calculate External Bearing Resistance

Calculate V_R

Per (1) Eq. 26,

$$\sum V_R = \gamma_{EV\ MAX}(W) + \gamma_{EV\ MAX}(W_{RSF}) + \gamma_{DC\ MAX}(W_{face}) + \gamma_{LS}(q_t b_{rb,t}) + \gamma_{EH\ MAX}(q_{rb} b_{rb,t}) + \gamma_{DC\ MAX}(q_{DL} b) + \gamma_{LS}(q_{LL} b)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\ MIN}$

W =	36.34 klf =	1.35 x	26918.37 plf/	1000	
W_{RSF} =	5.53 klf =	1.35 x	4096.88 plf/	1000	
W_{FACE} =	2.34 klf =	1.25 x	1870.00 plf/	1000	
W_{SF} =	4.88 klf =	1.35 x	3611.33 plf/	1000	
Q_t =	0.00 klf =	1.75 x	(0.00 psf x	0.00 ft)/	1000
Total =	49.08 klf				

Calculate $M_{D,R}$

Per (1) Eq. 28

$$\sum M_{D,R} = \gamma_{EH\ MAX}F_{b,RSF}\left(\frac{H + D_{RSF}}{3}\right) + \gamma_{LS}F_{t,RSF}\left(\frac{H + D_{RSF}}{2}\right) + \gamma_{EH\ MAX}F_{rb,RSF}\left(\frac{H + D_{RSF}}{2}\right)$$

$F_{b,\ RSF}$ =	181.58 k-ft/ ft =	1.5 x	14.96 klf x	(0.33 x	(21.7742 ft +	2.5 ft))
$F_{t,\ RSF}$ =	0.00 k-ft/ ft =	1.75 x	0.00 klf x	(0.50 x	(21.7742 ft +	2.5 ft))
Total =	181.58 k-ft/ ft					

Note that $M_{D,R}$ is taken about the bottom center of the width of the RSF.

Calculate $M_{R,R}$

Per (1) Eq. 29

$$\sum M_{R,R} = (\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b)\left[\left(\frac{b}{2} + a_b\right) - \left(\frac{B_{RSF}}{2} - x_{RSF} - b_{block}\right)\right] + \left(\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t}\right)\left(\frac{B_{RSF}}{2} - \frac{b_{rb}}{2}\right) + \gamma_{EV\,MAX} W\left(\frac{B_{RSF}}{2} - \frac{B}{2}\right) + \gamma_{DC\,MAX} W_{face}\left(B + \frac{b_{block}}{2} - \frac{B_{RSF}}{2}\right)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\,MAX}$. Resisting component = W_{SF}*((X_{RSF}+D_{block}+(2/3)*B)-BRSF/2)

W =	63.59 k-ft =	1.35 x	26.92 klf x	(7.125 ft -	5.375 ft)			
W_{Face} =	-10.41 k-ft =	1.35 x	1.87 klf x	((2.5 ft +	0.5 ft) -	7.125 ft)		
W_{SF} =	17.27 k-ft =	1.35 x	3.61 klf x	((2.5 ft +	1 ft +	(0.67 x	10.75 ft)) -	(14.25 ft/ 2))
Total =	70.45 k-ft/ ft							

Note that $M_{R,R}$ is taken about the bottom center of the width of the RSF.

Calculate Eccentricity, $e_{B,R}$

Per (1) Eq. 27

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R}$$

$e_{B,R}$ = 2.26 ft = (181.58 k-ft/ ft - 70.45 k-ft/ ft) / 49.08 klf

Calculate $\sigma_{v,base,R}$

Since $e_{B,R}$ = negative, set equal to zero in calcaulation of base bearing pressure.

Per (1) Eq. 25,

$$\sigma_{v,base,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}}$$

$\sigma_{v,base,R}$ = 5.05 ksf = 49.08 klf/ (14.25 ft - (2 x 2.26 ft)

Per abutment bearing capacity calculation, allowable bearing capacity = 11.88 ksf

OK

Calculate Reinforcement Strength

q_t = 0.00 psf
W = 26918.37 psf
W_{RSF} = 4096.88 psf
W_{face} = 1870.00 psf

The evaulation of the abutment for the strength limit state is conducted according to (1) 4.3.7.3.1. and using the following equations:

Required Reinforcment:

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 40

Factored Total Lateral Pressure:

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f}$$ (3) Eq. 41

Where:

$$\sigma_{h,W,f} = \gamma_{EH\ MAX}(\gamma_r z K_{ar})$$

Where:
$\gamma_{EH\ MAX}$ = maximum horizontal earth pressure load factor.
γ_r = unit weight of reinforced backfill.
z = depth from the top of the wall.
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

(3) Eq. 42

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar}$$ (3) Eq. 45

Note that $\sigma_{h,bridge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Factored Reinforcement Strength:

$$T_{f,f} = \Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right) = 0.4T_f$$ (3) Eq. 48

Strength Limit State

z	$\sigma_{h,W,f}$	$\sigma_{h,t,f}$	$\sigma_{h,f}$	$T_{req,f}$	$T_{f,f}$	
0.67 ft	0.02 ksf	0.00 ksf	0.02 ksf	0.03 ksf	2.36 klf	OK
1.33 ft	0.05 ksf	0.00 ksf	0.05 ksf	0.06 ksf	2.36 klf	OK
2.00 ft	0.07 ksf	0.00 ksf	0.07 ksf	0.09 ksf	2.36 klf	OK
2.67 ft	0.10 ksf	0.00 ksf	0.10 ksf	0.12 ksf	2.36 klf	OK
3.33 ft	0.12 ksf	0.00 ksf	0.12 ksf	0.16 ksf	2.36 klf	OK
4.00 ft	0.15 ksf	0.00 ksf	0.15 ksf	0.19 ksf	2.36 klf	OK
4.67 ft	0.17 ksf	0.00 ksf	0.17 ksf	0.22 ksf	2.36 klf	OK
5.33 ft	0.20 ksf	0.00 ksf	0.20 ksf	0.25 ksf	2.36 klf	OK
6.00 ft	0.22 ksf	0.00 ksf	0.22 ksf	0.28 ksf	2.36 klf	OK
6.67 ft	0.25 ksf	0.00 ksf	0.25 ksf	0.31 ksf	2.36 klf	OK
7.33 ft	0.27 ksf	0.00 ksf	0.27 ksf	0.34 ksf	2.36 klf	OK
8.00 ft	0.30 ksf	0.00 ksf	0.30 ksf	0.37 ksf	2.36 klf	OK
8.67 ft	0.32 ksf	0.00 ksf	0.32 ksf	0.40 ksf	2.36 klf	OK
9.33 ft	0.35 ksf	0.00 ksf	0.35 ksf	0.44 ksf	2.36 klf	OK
10.00 ft	0.37 ksf	0.00 ksf	0.37 ksf	0.47 ksf	2.36 klf	OK
10.67 ft	0.40 ksf	0.00 ksf	0.40 ksf	0.50 ksf	2.36 klf	OK
11.33 ft	0.42 ksf	0.00 ksf	0.42 ksf	0.53 ksf	2.36 klf	OK
12.00 ft	0.45 ksf	0.00 ksf	0.45 ksf	0.56 ksf	2.36 klf	OK
12.67 ft	0.47 ksf	0.00 ksf	0.47 ksf	0.59 ksf	2.36 klf	OK
13.33 ft	0.49 ksf	0.00 ksf	0.49 ksf	0.62 ksf	2.36 klf	OK
14.00 ft	0.52 ksf	0.00 ksf	0.52 ksf	0.65 ksf	2.36 klf	OK
14.67 ft	0.54 ksf	0.00 ksf	0.54 ksf	0.68 ksf	2.36 klf	OK
15.33 ft	0.57 ksf	0.00 ksf	0.57 ksf	0.71 ksf	2.36 klf	OK
16.00 ft	0.59 ksf	0.00 ksf	0.59 ksf	0.75 ksf	2.36 klf	OK
16.67 ft	0.62 ksf	0.00 ksf	0.62 ksf	0.78 ksf	2.36 klf	OK
17.33 ft	0.64 ksf	0.00 ksf	0.64 ksf	0.81 ksf	2.36 klf	OK
18.00 ft	0.67 ksf	0.00 ksf	0.67 ksf	0.84 ksf	2.36 klf	OK
18.67 ft	0.69 ksf	0.00 ksf	0.69 ksf	0.87 ksf	2.36 klf	OK
19.33 ft	0.72 ksf	0.00 ksf	0.72 ksf	0.90 ksf	2.36 klf	OK
20.00 ft	0.74 ksf	0.00 ksf	0.74 ksf	0.93 ksf	2.36 klf	OK
20.67 ft	0.77 ksf	0.00 ksf	0.77 ksf	0.96 ksf	2.36 klf	OK
21.33 ft	0.79 ksf	0.00 ksf	0.79 ksf	0.99 ksf	2.36 klf	OK
21.77 ft	0.81 ksf	0.00 ksf	0.81 ksf	1.02 ksf	2.36 klf	OK

The evaulation of the abutment for the service limit state is conducted according to (1) 4.3.7.3.2. and using the following equations:

The nominal required reinforcement strength:

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 50

The lateral pressure within the GRS Abutment at a given dpeth and location:

$$\sigma_h = \sigma_{h,W} + \sigma_{h,bridge,eq} + \sigma_{h,rb} + \sigma_{h,t}$$ (3) Eq. 51

Where:

$$\sigma_{h,W} = \gamma_r z K_{ar}$$ (3) Eq. 2

$$\sigma_{h,t} = q_t K_{ar}$$ (3) Eq. 54

Note that $\sigma_{hbridge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Service Limit State

z	$\sigma_{h,W}$	$\sigma_{h,t}$	σ_h	T_{req}	$T_{@\varepsilon = 2\%}$	
0.67 ft	0.02 ksf	0.00 ksf	0.016 ksf	0.02 klf	1.25 klf	OK
1.33 ft	0.03 ksf	0.00 ksf	0.033 ksf	0.04 klf	1.25 klf	OK
2.00 ft	0.05 ksf	0.00 ksf	0.049 ksf	0.06 klf	1.25 klf	OK
2.67 ft	0.07 ksf	0.00 ksf	0.066 ksf	0.08 klf	1.25 klf	OK
3.33 ft	0.08 ksf	0.00 ksf	0.082 ksf	0.10 klf	1.25 klf	OK
4.00 ft	0.10 ksf	0.00 ksf	0.099 ksf	0.12 klf	1.25 klf	OK
4.67 ft	0.12 ksf	0.00 ksf	0.115 ksf	0.15 klf	1.25 klf	OK
5.33 ft	0.13 ksf	0.00 ksf	0.132 ksf	0.17 klf	1.25 klf	OK
6.00 ft	0.15 ksf	0.00 ksf	0.148 ksf	0.19 klf	1.25 klf	OK
6.67 ft	0.16 ksf	0.00 ksf	0.165 ksf	0.21 klf	1.25 klf	OK
7.33 ft	0.18 ksf	0.00 ksf	0.181 ksf	0.23 klf	1.25 klf	OK
8.00 ft	0.20 ksf	0.00 ksf	0.198 ksf	0.25 klf	1.25 klf	OK
8.67 ft	0.21 ksf	0.00 ksf	0.214 ksf	0.27 klf	1.25 klf	OK
9.33 ft	0.23 ksf	0.00 ksf	0.231 ksf	0.29 klf	1.25 klf	OK
10.00 ft	0.25 ksf	0.00 ksf	0.247 ksf	0.31 klf	1.25 klf	OK
10.67 ft	0.26 ksf	0.00 ksf	0.264 ksf	0.33 klf	1.25 klf	OK
11.33 ft	0.28 ksf	0.00 ksf	0.280 ksf	0.35 klf	1.25 klf	OK
12.00 ft	0.30 ksf	0.00 ksf	0.297 ksf	0.37 klf	1.25 klf	OK
12.67 ft	0.31 ksf	0.00 ksf	0.313 ksf	0.39 klf	1.25 klf	OK
13.33 ft	0.33 ksf	0.00 ksf	0.330 ksf	0.41 klf	1.25 klf	OK
14.00 ft	0.35 ksf	0.00 ksf	0.346 ksf	0.44 klf	1.25 klf	OK
14.67 ft	0.36 ksf	0.00 ksf	0.363 ksf	0.46 klf	1.25 klf	OK
15.33 ft	0.38 ksf	0.00 ksf	0.379 ksf	0.48 klf	1.25 klf	OK
16.00 ft	0.40 ksf	0.00 ksf	0.396 ksf	0.50 klf	1.25 klf	OK
16.67 ft	0.41 ksf	0.00 ksf	0.412 ksf	0.52 klf	1.25 klf	OK
17.33 ft	0.43 ksf	0.00 ksf	0.429 ksf	0.54 klf	1.25 klf	OK
18.00 ft	0.45 ksf	0.00 ksf	0.445 ksf	0.56 klf	1.25 klf	OK
18.67 ft	0.46 ksf	0.00 ksf	0.462 ksf	0.58 klf	1.25 klf	OK
19.33 ft	0.48 ksf	0.00 ksf	0.478 ksf	0.60 klf	1.25 klf	OK
20.00 ft	0.49 ksf	0.00 ksf	0.495 ksf	0.62 klf	1.25 klf	OK
20.67 ft	0.51 ksf	0.00 ksf	0.511 ksf	0.64 klf	1.25 klf	OK
21.33 ft	0.53 ksf	0.00 ksf	0.528 ksf	0.66 klf	1.25 klf	OK
21.77 ft	0.54 ksf	0.00 ksf	0.538 ksf	0.68 klf	1.25 klf	OK

References:

1) GRS-IBS Interim Implementation Guide, FHWA, June 2018
(2) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018
(3) Geotechnical Data Report, Teracon Consultants Inc., March 27. 2019

Material Properties

γ_f =	115.00 pcf	(Reinforced Fill)
ϕ (Reinforced Soil)=	45	degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β =	26.57	degrees

K_a (Reinforced Soil) = 0.22 = 0.894 x (0.894 - (0.800 - 0.5)^ 0.5) / (0.894 + (0.800 - 0.5)^ 0.5)

Fill retained is in-situ soil.

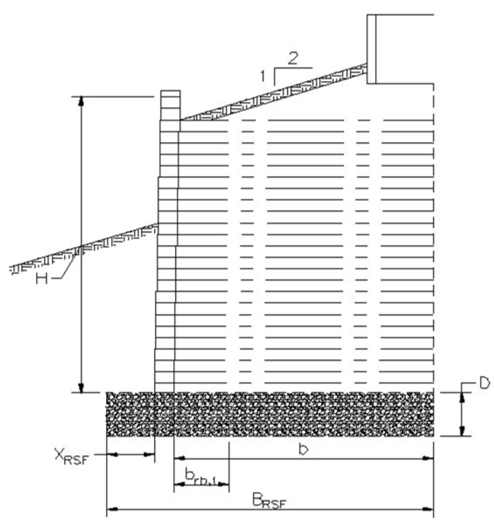
γ_b =	125.00 pcf	(Retained Fill)
ϕ (Existing Filll)=	34.00	degrees

Backfill has a 2H:1V slope, therefore β = 26 degrees

β =	26.57	degrees

K_a (Retained Fill) = 0.41 = 0.894 x (0.894 - (0.800 - 0.6873)^ 0.5) / (0.894 + (0.800 - 0.6873)^ 0.5)

Geometry



Depth of RSF, D =	2.5 ft		
Top of RSF EL. =	447.46		
Bottom of RSF EL. =	444.96 =	447.46 -	2.5 ft
Top of Wall EL. =	466.51		
Wall Height =	19.05 ft	466.51 -	447.46
H =	19.05 ft		
B =	9.5 ft		
Toe Length, X_{RSF} =	2.5 ft		
Total Width, B_{RSF} =	13 ft		
$b_{rb,t}$ =	0.00 ft		
Stepped Footing Length =	18.50 ft		
h_{block} =	8.00 in		
D_{block} =	12.00 in		
L_{block} =	18.00 in		
Weight =	85 lbs	per block	

Reinforcement

S_v = 0.67 ft
d_{max} = 0.75 in
T_f = 5.90 klf

Loading
Note that the superstructure dead loads, roadway bed loads, and traffic live loads are not applicable for this structure. No live load surcharge is applied.

Load Factors
Per (3) Table 3.4.1-1 and Table 3.4.1-2.

γ_p (DC, Min.) =	0.90	3) Table 3.4.1-2
γ_p (DC, Max.) =	1.25	3) Table 3.4.1-2
γ_p (EH, Max) =	1.50	3) Table 3.4.1-2
γ_p (EV, Min.) =	1.00	3) Table 3.4.1-2
γ_p (EV, Max.) =	1.35	3) Table 3.4.1-2
Factor (LS) =	1.75	3) Table 3.4.1-1

Calculate Vertical Loads and Applied Pressures
Calculate Traffic Surcharge, q_T

Due to wingwall layout geometry, assume that there is no live load surcharge acting on the wingwall.

h_{eq} = 0.00 ft
γ_r= 125.00 pcf

$$q_t = h_{eq}\gamma_b$$

q_T = 0.00 psf = 0.00 ft x 125.00 pcf

Calculate Weight of GRS Abutment

$$W = BH\gamma_r$$

H = 19.05 ft
B = 9.5 ft
γ_r= 115.00 pcf

W = 20810.46 plf = 19.04847 ft x 9.5 ft x 115.00 pcf

Calculate Weight of RSF

$$W_{RSF} = B_{RSF}D_{RSF}\gamma_r$$

B = 13 ft
D = 2.5 ft
γ_r= 125.00 pcf

W_{RSF} = 3737.50 plf = 13 ft x 2.5 ft x 115.00 pcf

Calculate Weight of Facing

N_{block} = 28.57 = 19.048475 ft/ 0.67 ft
N_{block} = 29

$$W_{face} = N_{block}\frac{W_{block}}{L_{block}}$$

W_{face} = 1643.33 plf = 29 x (85 lbs/ (18.00 in/ 12)

Calculate Weight of Surplus Retained Fill

Fill is retained at a 2H:1V slope

B = 9.5 ft
H_{max} = 4.75 ft = 9.5 ft/ 2
W_{SF} = 2820.31 plf = 0.5 x 9.5 ft x 4.75 ft x 125.00 pcf

FINAL DESIGN - GRS-IBS WINGWALL - ABUTMENT 4 - SEWW - STEP 1 VTRANS D37 IM 091-1(68)

Summary of Applied Loads

q_t = 0.00 psf
W = 20810.46 plf
W_{RSF} = 3737.50 plf
W_{face} = 1643.33 plf
W_{SF} = 2820.31 plf

Calculte Direct Sliding Effects at RSF/GRS Interface

Calculate Sliding per the (1) and (3) Strength I Load Combination

$$F_b = \frac{1}{2}\gamma_b K_{ab} H^2$$ (1) Eq. 9

$$F_t = q_t K_{ab} H$$ (1) Eq. 11

$$F_R = \gamma_{EH\,MAX}(F_b + F_{rb}) + \gamma_{LS} F_t$$ (1) Eq. 12

F_b = 9212.80 lbs = 0.5 x 125.00 pcf x 0.41 x 19.04847 ft^ 2
F_t = 0.00 lbs = 0.00 psf 0.41 x 19.0484746 ft

F_R = 13.82 klf = (1.5 x 9212.80 lbs) + (1.75 x 0.00 lbs))/ 1000

$$W_{T,R} = \gamma_{EV\,MIN} W + \gamma_{DC\,MIN}(q_{DL} b) + \gamma_{DC\,MIN}(W_{face}) + \gamma_{EV\,MIN}(q_{rb} b_{rb,t}) + \gamma_{EV\,MIN} W_{SF}$$ (1) Eq. 14

$W_{T,R}$ = 25.11 klf = ((1.0 x 20810.46 plf) + ((0.9 x 1643.33 plf))/ 1000) + (1.00 x 2.82 klf)

Assume that μ = 2/3*tanϕ, per (1) C.1.6.1.1

μ = 0.67 = 0.67 x tan(45)

$$R_R = \Phi_\tau (W_{T,R}\mu)$$ (1) Eq. 13

R_R = 16.74 klf = 25.110 klf x 0.667

OK

Calculate Direct Sliding at Base of RSF

$$F_{b,RSF} = \frac{1}{2}\gamma_b K_{ab}(H + D_{RSF})^2$$ (3) Eq. 17

$$F_{t,RSF} = q_t K_{ab}(H + D_{RSF})$$ (3) Eq. 19

$F_{b,\ RSF}$ =	11.79 klf =	(0.5 x	125.00 pcf x	0.41 x	(19.05 ft +	2.5)^ 2)/	1000
$F_{t,\ RSF}$ =	0.00 klf =	0.00 psf x	0.41 x	(19.05 ft +	2.5 ft))/	1000	

$$F_{R,RSF} = \gamma_{EH\,MAX}\left(F_{b,RSF} + F_{rb,RSF}\right) + \gamma_{LS} F_{t,RSF}$$ (1) Eq. 20

$F_{R,RSF}$ = 17.68 klf = (1.5 x x 11.79 klf) + (1.75 x 0.00 klf)

$$W_{T,R,RSF} = W_{T,R} + \gamma_{EV\,MIN} W_{RSF}$$ (1) Eq. 22

$W_{T,R,RSF}$ = 28.85 klf = 25.11 klf + (1.0 x 3.74 klf)

$$R_{R,RSF} = \Phi_\tau(W_{T,R,RSF}\,\mu_{RSF})$$ (1) Eq. 21

Note that φ = 1.0, per (1) 4.3.6.1.2
Per (2), and the previous calculation, the angle of internal friction beneath the RSF is equal to 32 degrees

$R_{R,\ RSF}$ = 18.03 klf = 1.0 x 28.85 klf x tan(32)

OK

Calculate External Bearing Resistance

Calculate V_R

Per (1) Eq. 26,

$$\sum V_R = \gamma_{EV\,MAX}(W) + \gamma_{EV\,MAX}(W_{RSF}) + \gamma_{DC\,MAX}(W_{face}) + \gamma_{LS}(q_t b_{rb,t}) + \gamma_{EH\,MAX}(q_{rb} b_{rb,t}) + \gamma_{DC\,MAX}(q_{DL} b) + \gamma_{LS}(q_{LL} b)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\,MIN}$

W =	28.09 klf =	1.35 x	20810.46 plf/	1000	
W_{RSF} =	5.05 klf =	1.35 x	3737.50 plf/	1000	
W_{FACE} =	2.05 klf =	1.25 x	1643.33 plf/	1000	
W_{SF} =	3.81 klf =	1.35 x	2820.31 plf/	1000	
Q_t =	0.00 klf =	1.75 x	(0.00 psf x	0.00 ft)/	1000
Total =	39.00 klf				

Calculate $M_{D,R}$

Per (1) Eq. 28

$$\sum M_{D,R} = \gamma_{EH\,MAX} F_{b,RSF}\left(\frac{H + D_{RSF}}{3}\right) + \gamma_{LS} F_{t,RSF}\left(\frac{H + D_{RSF}}{2}\right) + \gamma_{EH\,MAX} F_{rb,RSF}\left(\frac{H + D_{RSF}}{2}\right)$$

$F_{b,\,RSF}$ =	127.03 k-ft/ ft =	1.5 x	11.79 klf x	(0.33 x	(19.0485 ft +	2.5 ft))
$F_{t,\,RSF}$ =	0.00 k-ft/ ft =	1.75 x	0.00 klf x	(0.50 x	(19.0485 ft +	2.5 ft))
Total =	127.03 k-ft/ ft					

Note that $M_{D,R}$ is taken about the bottom center of the width of the RSF.

Calculate $M_{R,R}$

Per (1) Eq. 29

$$\sum M_{R,R} = (\gamma_{DC\,MAX} q_{DL} b + \gamma_{LS} q_{LL} b)\left[\left(\frac{b}{2} + a_b\right) - \left(\frac{B_{RSF}}{2} - x_{RSF} - b_{block}\right)\right] + (\gamma_{LS} q_t b_{rb,t} + \gamma_{EV\,MAX} q_{rb} b_{rb,t})\left(\frac{B_{RSF}}{2} - \frac{b_{rb}}{2}\right) + \gamma_{EV\,MAX} W\left(\frac{B_{RSF}}{2} - \frac{B}{2}\right) + \gamma_{DC\,MAX} W_{face}\left(B + \frac{b_{block}}{2} - \frac{B_{RSF}}{2}\right)$$

Note that the additional weight of the 2H:1V retained fill to be included, factored by $\gamma_{EV\,MAX}$. Resisting component = W_{SF}*((X_{RSF}+D_{block}+(2/3)*B)-BRSF/2)

W =	49.16 k-ft =	1.35 x	20.81 klf x	(6.5 ft -	4.75 ft)				
W_{Face} =	-7.76 k-ft =	1.35 x	1.64 klf x	((2.5 ft +	0.5 ft) -	6.5 ft)			
W_{SF} =	12.69 k-ft =	1.35 x	2.82 klf x	((2.5 ft +	1 ft +	(0.67 x	9.5 ft)) -	(13 ft/ 2))	
Total =	54.09 k-ft/ ft								

Note that $M_{R,R}$ is taken about the bottom center of the width of the RSF.

Calculate Eccentricity, $e_{B,R}$

Per (1) Eq. 27

$$e_{B,R} = \frac{\sum M_{D,R} - \sum M_{R,R}}{\sum V_R}$$

$e_{B,R}$ = 1.87 ft = (127.03 k-ft/ ft - 54.09 k-ft/ ft) / 39.00 klf

Calculate $\sigma_{v,base,R}$

Since $e_{B,R}$ = negative, set equal to zero in calcaulation of base bearing pressure.

Per (1) Eq. 25,

$$\sigma_{v,base,R} = \frac{\sum V_R}{B_{RSF} - 2e_{B,R}}$$

$\sigma_{v,base,R}$ = 4.21 ksf = 39.00 klf/ (13 ft - (2 x 1.87 ft)

Per abutment bearing capacity calculation, allowable bearing capacity = 11.88 ksf

OK

Calculate Reinforcement Strength

q_t = 0.00 psf
W = 20810.46 psf
W_{RSF} = 3737.50 psf
W_{face} = 1643.33 psf

The evaulation of the abutment for the strength limit state is conducted according to (1) 4.3.7.3.1. and using the following equations:

Required Reinforcment:

$$T_{req,f} = \left[\frac{\sigma_{h,f}}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 40

Factored Total Lateral Pressure:

$$\sigma_{h,f} = \sigma_{h,W,f} + \sigma_{h,bridge,f} + \sigma_{h,rb,f} + \sigma_{h,t,f}$$ (3) Eq. 41

Where:

$$\sigma_{h,W,f} = \gamma_{EH\,MAX}(\gamma_r z K_{ar})$$ (3) Eq. 42

Where:
$\gamma_{EH\,MAX}$ = maximum horizontal earth pressure load factor.
γ_r = unit weight of reinforced backfill.
z = depth from the top of the wall.
K_{ar} = coefficient of active earth pressure for the reinforced backfill.

$$\sigma_{h,t,f} = \gamma_{LS} q_t K_{ar}$$ (3) Eq. 45

Note that $\sigma_{h,bridge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

Factored Reinforcement Strength:

$$T_{f,f} = \Phi_{reinf}\left(\frac{T_f}{RF_{global}}\right) = 0.4T_f$$ (3) Eq. 48

Strength Limit State

z	$\sigma_{h,W,f}$	$\sigma_{h,t,f}$	$\sigma_{h,f}$	$T_{req,f}$	$T_{f,f}$	
0.67 ft	0.02 ksf	0.00 ksf	0.02 ksf	0.03 ksf	2.36 klf	OK
1.33 ft	0.05 ksf	0.00 ksf	0.05 ksf	0.06 ksf	2.36 klf	OK
2.00 ft	0.07 ksf	0.00 ksf	0.07 ksf	0.09 ksf	2.36 klf	OK
2.67 ft	0.10 ksf	0.00 ksf	0.10 ksf	0.12 ksf	2.36 klf	OK
3.33 ft	0.12 ksf	0.00 ksf	0.12 ksf	0.16 ksf	2.36 klf	OK
4.00 ft	0.15 ksf	0.00 ksf	0.15 ksf	0.19 ksf	2.36 klf	OK
4.67 ft	0.17 ksf	0.00 ksf	0.17 ksf	0.22 ksf	2.36 klf	OK
5.33 ft	0.20 ksf	0.00 ksf	0.20 ksf	0.25 ksf	2.36 klf	OK
6.00 ft	0.22 ksf	0.00 ksf	0.22 ksf	0.28 ksf	2.36 klf	OK
6.67 ft	0.25 ksf	0.00 ksf	0.25 ksf	0.31 ksf	2.36 klf	OK
7.33 ft	0.27 ksf	0.00 ksf	0.27 ksf	0.34 ksf	2.36 klf	OK
8.00 ft	0.30 ksf	0.00 ksf	0.30 ksf	0.37 ksf	2.36 klf	OK
8.67 ft	0.32 ksf	0.00 ksf	0.32 ksf	0.40 ksf	2.36 klf	OK
9.33 ft	0.35 ksf	0.00 ksf	0.35 ksf	0.44 ksf	2.36 klf	OK
10.00 ft	0.37 ksf	0.00 ksf	0.37 ksf	0.47 ksf	2.36 klf	OK
10.67 ft	0.40 ksf	0.00 ksf	0.40 ksf	0.50 ksf	2.36 klf	OK
11.33 ft	0.42 ksf	0.00 ksf	0.42 ksf	0.53 ksf	2.36 klf	OK
12.00 ft	0.45 ksf	0.00 ksf	0.45 ksf	0.56 ksf	2.36 klf	OK
12.67 ft	0.47 ksf	0.00 ksf	0.47 ksf	0.59 ksf	2.36 klf	OK
13.33 ft	0.49 ksf	0.00 ksf	0.49 ksf	0.62 ksf	2.36 klf	OK
14.00 ft	0.52 ksf	0.00 ksf	0.52 ksf	0.65 ksf	2.36 klf	OK
14.67 ft	0.54 ksf	0.00 ksf	0.54 ksf	0.68 ksf	2.36 klf	OK
15.33 ft	0.57 ksf	0.00 ksf	0.57 ksf	0.71 ksf	2.36 klf	OK
16.00 ft	0.59 ksf	0.00 ksf	0.59 ksf	0.75 ksf	2.36 klf	OK
16.67 ft	0.62 ksf	0.00 ksf	0.62 ksf	0.78 ksf	2.36 klf	OK
17.33 ft	0.64 ksf	0.00 ksf	0.64 ksf	0.81 ksf	2.36 klf	OK
18.00 ft	0.67 ksf	0.00 ksf	0.67 ksf	0.84 ksf	2.36 klf	OK
18.67 ft	0.69 ksf	0.00 ksf	0.69 ksf	0.87 ksf	2.36 klf	OK
19.05 ft	0.71 ksf	0.00 ksf	0.71 ksf	0.89 ksf	2.36 klf	OK

The evaulation of the abutment for the service limit state is conducted according to (1) 4.3.7.3.2. and using the following equations:

The nominal required reinforcement strength:

$$T_{req} = \left[\frac{\sigma_h}{0.7^{\left(\frac{S_v}{6d_{max}}\right)}}\right] S_v$$ (3) Eq. 50

The lateral pressure within the GRS Abutment at a given dpeth and location:

$$\sigma_h = \sigma_{h,W} + \sigma_{h,bridge,eq} + \sigma_{h,rb} + \sigma_{h,t}$$ (3) Eq. 51

Where:

$$\sigma_{h,W} = \gamma_r z K_{ar}$$ (3) Eq. 2

$$\sigma_{h,t} = q_t K_{ar}$$ (3) Eq. 54

Note that $\sigma_{hbrdge,f}$ and $\sigma_{h,rb,f}$ are not applicable for the wingwall check.

FINAL DESIGN - GRS-IBS WINGWALL - ABUTMENT 4 - SEWW - STEP 1 VTRANS D37 IM 091-1(68)

Service Limit State

z	$\sigma_{h,W}$	$\sigma_{h,t}$	σ_h	T_{req}	$T_{@\varepsilon = 2\%}$	
0.67 ft	0.02 ksf	0.00 ksf	0.016 ksf	0.02 klf	1.25 klf	OK
1.33 ft	0.03 ksf	0.00 ksf	0.033 ksf	0.04 klf	1.25 klf	OK
2.00 ft	0.05 ksf	0.00 ksf	0.049 ksf	0.06 klf	1.25 klf	OK
2.67 ft	0.07 ksf	0.00 ksf	0.066 ksf	0.08 klf	1.25 klf	OK
3.33 ft	0.08 ksf	0.00 ksf	0.082 ksf	0.10 klf	1.25 klf	OK
4.00 ft	0.10 ksf	0.00 ksf	0.099 ksf	0.12 klf	1.25 klf	OK
4.67 ft	0.12 ksf	0.00 ksf	0.115 ksf	0.15 klf	1.25 klf	OK
5.33 ft	0.13 ksf	0.00 ksf	0.132 ksf	0.17 klf	1.25 klf	OK
6.00 ft	0.15 ksf	0.00 ksf	0.148 ksf	0.19 klf	1.25 klf	OK
6.67 ft	0.16 ksf	0.00 ksf	0.165 ksf	0.21 klf	1.25 klf	OK
7.33 ft	0.18 ksf	0.00 ksf	0.181 ksf	0.23 klf	1.25 klf	OK
8.00 ft	0.20 ksf	0.00 ksf	0.198 ksf	0.25 klf	1.25 klf	OK
8.67 ft	0.21 ksf	0.00 ksf	0.214 ksf	0.27 klf	1.25 klf	OK
9.33 ft	0.23 ksf	0.00 ksf	0.231 ksf	0.29 klf	1.25 klf	OK
10.00 ft	0.25 ksf	0.00 ksf	0.247 ksf	0.31 klf	1.25 klf	OK
10.67 ft	0.26 ksf	0.00 ksf	0.264 ksf	0.33 klf	1.25 klf	OK
11.33 ft	0.28 ksf	0.00 ksf	0.280 ksf	0.35 klf	1.25 klf	OK
12.00 ft	0.30 ksf	0.00 ksf	0.297 ksf	0.37 klf	1.25 klf	OK
12.67 ft	0.31 ksf	0.00 ksf	0.313 ksf	0.39 klf	1.25 klf	OK
13.33 ft	0.33 ksf	0.00 ksf	0.330 ksf	0.41 klf	1.25 klf	OK
14.00 ft	0.35 ksf	0.00 ksf	0.346 ksf	0.44 klf	1.25 klf	OK
14.67 ft	0.36 ksf	0.00 ksf	0.363 ksf	0.46 klf	1.25 klf	OK
15.33 ft	0.38 ksf	0.00 ksf	0.379 ksf	0.48 klf	1.25 klf	OK
16.00 ft	0.40 ksf	0.00 ksf	0.396 ksf	0.50 klf	1.25 klf	OK
16.67 ft	0.41 ksf	0.00 ksf	0.412 ksf	0.52 klf	1.25 klf	OK
17.33 ft	0.43 ksf	0.00 ksf	0.429 ksf	0.54 klf	1.25 klf	OK
18.00 ft	0.45 ksf	0.00 ksf	0.445 ksf	0.56 klf	1.25 klf	OK
18.67 ft	0.46 ksf	0.00 ksf	0.462 ksf	0.58 klf	1.25 klf	OK
19.05 ft	0.47 ksf	0.00 ksf	0.471 ksf	0.59 klf	1.25 klf	OK

References:

1) AASHTO LRFD Bridge Design, 8th Edition with Errata, 2018

2) VTrans Structures Design Manual, 2010

Determine Design Lateral Load

Per (1) 3.10.9.2, for Seismic Zone 1 where A_s < 0.05, the horizontal design connection force shall not be less than 0.15 times the beam dead load tributary reaction. For all other locations within Zone 1, the design force shall not be 0.25 times the beam dead load tributary reaction. Conservatively assume that A_s > 0.05.

By inspection, controlling DL reaction comes from the North Exterior Beam from Span 2

DL =	56.44 k			
Horizontal Force =	14.11 k =	0.25 x	56.44 k	

Determine Factored Shear Resistance per Anchor Bolt

Assume threads are excluded from the shear plane.

Bolt Type =	ASTM F1554
Number of Bolts =	1
Bolt Diam. =	1.5 in
Bolt Area =	1.77 in²
F_u =	58 ksi

Per (1) 6.13.2.7-1,

$$R_n = 0.56 A_b F_{ub} N_s \qquad (6.13.2.7\text{-}1)$$

ϕ_s =	0.75			(1) 6.5.4.2, A307 bolts in shear	
R_n =	57.40 k =	0.56 x	1.77 in² x	58.00 ksi x	1
R_r =	43.05 k =	0.75 x	57.40 k		
Design Force =	14.11 k				

OK

Determine Anchor Bolt Embedment Length, L_{emb}

Assume that bolt bearing stress varies linearly from zero at the end of the embedded length, to a maximum value at the top surface of the concrete.

Per (1) 5.6.5,

In the absence of confinement reinforcement in the concrete supporting the bearing device, the factored bearing resistance shall be taken as:

$$P_r = \phi P_n \quad (5.6.5\text{-}1)$$

in which:

$$P_n = 0.85 f'_c A_1 m \quad (5.6.5\text{-}2)$$

where:

P_n = nominal bearing resistance (kip)
A_1 = area under bearing device (in.^{2})
m = confinement modification factor
A_2 = notional area defined as shown in Figure 5.6.5-1 (in.^{2})

Conservatively assume m = 0.75

f'_c = 4 ksi   1) 5.5.4.2
ϕ_b = 0.70
m = 0.75

Bearing Stress = 1.79 ksi = 0.70 x 0.85 x 4 ksi x 0.75

Design Load = 14.11 k
P_{bolt} = 14.11 k = 14.11 k/ 1

A_1 = 15.81 in² = 14.11 k/ (1.79 ksi/ 2)

L_{emb} = 10.54 in = 15.81 in²/ 1.5 in

Per (2) embedment must be at least 15".

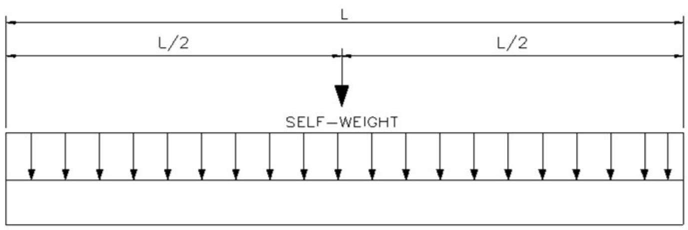
L_{emb} = 15 in
A_1 = 22.50 in² = 15 in x 1.5 in

P_r = 40.16 k = 0.70 x 0.85 x 4 ksi x 22.50 in² x 0.75
Design Force = 14.11 k
OK

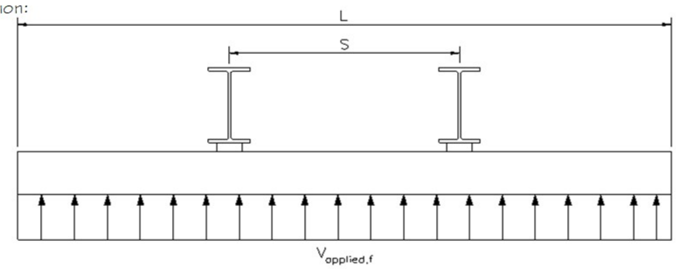
References:
(1) AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017 with interims thru 2018

Narrative:
The precast beam seat is to be designed as to act as a separate, indpentent member. The largest beam seat was used as the design case. The beam seat was subjected to an upward bearing pressure equal to the factored bearing pressure applied by dead and live loading, as calculated in the design of the GRS-IBS abutment. The bridge seat is designed for the controlling condition between the erection condition (assuming a single pick point in the center of the bridge seat, with only the self weight applied) and the final condition (bridge seat is subjected to the full applied bearing pressure). See sketches shown below.

Erection Condition:



Final Condition:



Bridge Seat Geometry

Note that the bridge seat thickness varies. Use the maximum thickness for the calculation of loads, and the minimum thickness for the design.

Bridge Seat Width =	4.75 ft
Bridge Seat Depth (Design) =	1.50 ft
Bridge Seat Depth (Loading) =	3.34 ft
Bridge Seat L (2 Beams) =	12.08 ft
Bridge Seat L (3 Beams) =	20.17 ft

Calculate Moment, Shear for Erection

Beam Seat Area =	15.87 ft² =	3.34 ft x	4.75 ft
Weight of Cap =	2.38 klf =	15.87 ft² x	0.150 kcf

Per (1) 3.4.2.1, apply a factor of 1.25 to construction loading to the weight of structure and appurtenances.
Assume a single pick, therefore beam acts as a cantilever.

Picking Moment =	54.29 k-ft =	1.25 x	0.5 x	2.380 klf x	6.04 ft ^	2
Picking Shear =	17.97 kips =	1.25 x	2.38 klf x (12.08 ft x			0.5)

Calculate Moment, Shear for Final Condition

Factored Upward Pressure =	4.00 ksf		
Beam Seat Width =	4.75 ft		
Uniform Distributed Load, w =	19.00 klf =	4.00 ksf x	4.75 ft

Final Condition - 2 Beams:

Bridge Skew =	10.49	degrees			
Beam Spacing =	6.67 ft				
L (Simple Span) =	6.78 ft =	6.67 ft/	cos(	10.49)	
L (Cantilever) =	2.65 ft =	0.5 x	(12.08 ft -	6.78 ft)	

Assume a simply supported position for bridge seat between beams, and a cantilever condition beyond beams.

Positive Moment =	109.18 k-ft =	0.125 x	19.00 klf x	6.78 ft ^	2
Negative Moment =	66.80 k-ft =	0.5 x	19.00 klf x	2.65 ft ^	2
Shear =	64.41 kips =	19 klf x	(6.78 ft x	0.50)	

As the max shear occurs right at a support, take the design shear at a distance d_v away from support.

d_v =	1.17 ft =	14.06 ft/ 12		
Max. shear @ distance d_v =	42.14 kips =	64.41 kips -	(19.00 klf x	1.17 ft)

Final Condition - 3 Beams:

Bridge Skew =	10.49	degrees				
Beam Spacing =	6.67 ft					
L (Continuous Span) =	6.78 ft =	6.67 ft/	cos(	10.49)		
L (Cantilever) =	3.30 ft =	0.5 x	(20.17 ft -		(2 x	6.78 ft))
Positive Moment (Cont.) =	61.14 k-ft =	0.07 x	19.00 klf x	6.78 ft ^		2)
Negative Moment (Cont.) =	109.18 k-ft =	0.125 x	19.00 klf x	6.78 ft ^		2)
Shear (Cont.) =	80.51 kips =	0.625 x	(19.00 klf x	6.78 ft)		
Negative Moment (Cant.) =	103.66 k-ft =	0.5 x	19.00 klf x	3.30 ft ^ 2		
Shear (Cant.) =	62.76 kips =	19 klf x	3.30 ft			

Determine Controlling Forces

Positive Moment =	109.18 k-ft
Negative Moment =	109.18 k-ft
Shear =	80.51 kips

As the max shear occurs right at a support, take the design shear at a distance d_v away from support.

d_v =	1.17 ft =	14.06 ft/ 12		
Max. shear @ distance d_v =	58.25 kips =	80.51 kips -	(19.00 klf x	1.17 ft)

Calculate Design Loads

By inspection, the moments and shears applied to the bridge seat are governed by the final condition. Since the upward loading pressure of the final condition has been previously factored, the factor for the design moments shalle equal 1.0.

Design Shear =	58.25 kips =	1 x	42.14 kips
Design Positive Moment =	109.18 k-ft =	1 x	109.18 k-ft
Design Negative Moment =	109.18 k-ft =	1 x	66.80 k-ft

V_u = 58.25 kips
M_{u+} = 109.18 k-ft
M_{u-} = 109.18 k-ft

Design Negative Moment Reinforcement

For the purpose of the design, M_{u-} = 109.18 k-ft

f_y =	60 ksi			
f'_c =	4 ksi			
E_c =	3644.15 ksi			
E_s =	29000			
n =	7.96			
γ_e =	1	(1) 5.7.3.4		
b =	57.00 in			
Thickness, H =	18.00 in			
Cover =	2.00 in			
Negative Reinforcement =	8 -	No. 6		
D_{bar} =	0.75 in			
d =	15.625 in =	18.00 in -	2.00 in -	0.375 in
A_{bar} =	0.44 in²			
A_s =	3.53 in² =	8 x	0.44 in²	

Per (1),

$$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_s\left(d_s - \frac{a}{2}\right) - A'_s f'_s\left(d'_s - \frac{a}{2}\right) + \alpha_1 f'_c (b - b_w) h_f\left(\frac{a}{2} - \frac{h_f}{2}\right)$$

Check effective depth:

d_e = 15.63 in

$0.9{*}d_e$ = 14.0625 in
0.72*h = 12.96 in
d_v = max of $0.9{*}d_e$ and 0.72*h = 14.0625 in
a = $c\beta_1$
α_1 = 0.85
β_1 = 0.85

$$c = \frac{A_{ps} f_{pu} + A_s f_s - A'_s f'_s}{\alpha_1 f'_c \beta_1 b + k A_{ps}\dfrac{f_{pu}}{d_p}}$$

c = 1.29 in = (3.53 in² x 60 ksi) / (0.85 x 4 ksi x 0.85 x 57 in)

a = 1.09 in = 0.85 x 1.29

<u>Verify that $f_s = f_y$:</u>

c/d = 0.09 = 1.29 in/ 14.0625 in

c/d < 0.6, therefore $f_s = f_y$

<u>Calculate Moment Strength M_n:</u>

M_n = 238.84 k-ft = (3.53 in² x 60 ksi x (14.0625 in - (1.09/ 2))/ 12

Per (1) 5.5.4.2.1, Φ_b = 0.90

M_r = 214.95 k-ft = 0.90 x 238.84 k-ft
M_u = 109.18 k-ft

OK

<u>Check that Section is Tension Controlled</u>

∈'t > ∈'y = 0.60

∈'t = (d-c/c)*0.003 = 0.039 = (14.06 in - (1.29 in/ 1.29 in)) x 0.003
∈'y = 0.005

OK

Calculate M_{cr}:

Per (1) 5.4.2.6, $f_r = 0.37(f'_c)^{.5}$ =	0.74 ksi =	0.37 x	4.00 ksi ^	0.5		
S_c =	3078.00 in³ =	0.17 x	57 in x	18 in ^	2	
Per (1) 5.7.3.3.2, $M_{cr} = f_r{}^*S_c$ =	189.81 k-ft =	(0.74 ksi x	3078.00 in³)/ 12			

Calculate Required Moment Capacity

$1.2{}^*M_{cr}$ =	227.77 k-ft =	1.2 x	189.81 k-ft
$1.33{}^*M_u$ =	145.20 k-ft =	1.33 x	109.18 k-ft
Min. Moment Strength =	145.20 k-ft		
M_r =	214.95 k-ft		

OK

Design Posative Moment Reinforcement

M_{u+} =	109.18 k-ft				
f_y =	60 ksi				
f'_c =	4 ksi				
E_c =	3644.15 ksi				
E_s =	29000				
n =	7.96				
γ_e =	1	(1) 5.7.3.4			
b =	57.00 in				
Thickness, H =	18.00 in				
Cover =	2.00 in				
Positive Reinforcement =	8 -	No. 6			
D_{bar} =	0.75 in				
d =	15.125 in =	18.00 in -	2.00 in -	0.375 in	0.50 in
A_{bar} =	0.44 in²				
A_s =	3.53 in² =	8 x	0.44 in²		

Per (1),

$$M_n = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_s\left(d_s - \frac{a}{2}\right) - A'_s f'_s\left(d'_s - \frac{a}{2}\right) + \alpha_1 f'_c(b - b_w)h_f\left(\frac{a}{2} - \frac{h_f}{2}\right)$$

d_e =	15.13 in
$0.9{}^*d_e$ =	13.61 in
$0.72{}^*h$ =	12.96 in
d_v = max of $0.9{}^*d_e$ and $0.72{}^*h$ =	13.61 in

$a = c\beta_1$

α_1 =	0.85
β_1 =	0.85

$$c = \frac{A_{ps}f_{pu} + A_s f_s - A'_s f'_s}{\alpha_1 f'_c \beta_1 b + kA_{ps}\dfrac{f_{pu}}{d_p}}$$

$$c = 1.29 \text{ in} = \frac{3.53 \text{ in}^2 \times 60 \text{ ksi}}{0.85 \times 4 \text{ ksi} \times 0.85 \times 57 \text{ in}}$$

a = 1.09 in = 0.85 x 1.29

Verify that $f_s = f_y$:

$c/d =$ 0.09 = 1.29 in/ 13.6125 in

$c/d < 0.6$, therefore $f_s = f_y$

Calculate Moment Strength M_n:

$M_n =$ 230.88 k-ft = (3.53 in² x 60 ksi x (13.6125 in - (1.09/ 2))/ 12

Per (1) 5.5.4.2.1, $\Phi_b =$ 0.90

$M_r =$ 207.80 k-ft = 0.90 x 230.88 k-ft

$M_u =$ 109.18 k-ft

OK

Check that Section is Tension Controlled

$\epsilon't >$ $\epsilon'y = 0.60$

$\epsilon't = (d-c/c)*0.003 =$ 0.038 = (13.61 in - (1.29 in/ 1.29 in)) x 0.003

$\epsilon'y = 0.005$

OK

Calculate M_{cr}:

Per (1) 5.4.2.6, $f_r = 0.37(f'_c)^{.5} =$ 0.74 ksi = 0.37 x 4.00 ksi ^ 0.5

$S_c =$ 3078.00 in³ = 0.17 x 57 in x 18 in ^ 2

Per (1) 5.7.3.3.2, $M_{cr} = f_r{}^*S_c =$ 189.81 k-ft = (0.74 ksi x 3078.00 in³)/ 12

Calculate Required Moment Capacity

$1.2{}^*M_{cr} =$ 227.77 k-ft = 1.2 x 189.81 k-ft

$1.33{}^*M_u =$ 145.20 k-ft = 1.33 x 109.18 k-ft

Min. Moment Strength = 145.20 k-ft

$M_r =$ 207.80 k-ft

OK

Design for Shear

Per (1) 5.8.3.4-1,

$\beta =$ 2.0

$\Theta =$ 45 = 0.785398163 rad

$d_e =$ 15.125 in = 18.00 in - 2.00 in - 0.50 in - 0.375 in

$0.9{}^*d_e =$ 13.61 in = 0.9 x 15.125 in

$0.72{}^*h =$ 12.96 in = 0.72 x 18.00 in

Per (1) 5.8.2.9,

d_v = max(0.90*d_e, 0.72*h) = 13.61 in

Per (1) Eq. 5.8.3.3-3, calcualte V_c:

$V_c = 0.0316*\beta*f'_c{}^{.5}*b*d_v$ = 98.08 kips = 0.0316 x 2 4.00 ksi ^ 0.5 x 57 in x 13.61 in

Per (1) 5.5.4.2.1, ϕ_v = 0.9

ϕ_v*V_c = 88.27 kips = 0.9 x 98.08 kips

Per (1) 5.8.2.4, provide shear reinforcement if $V_u>0.5*\phi V_c$

$0.5*\phi V_c$ = 44.13 kips = 0.5 x 88.27 kips

V_u = 58.25 kips

SHEAR REINFORCEMENT REQUIRED

	Try
stirrup size =	#4
stirrup d_b =	0.50 in
stirrup A_b =	0.20 in²
# legs =	2
s =	8.00 in

A_v = 0.39 in² = 0.20 in² x 2

Per (1) Eq. 5.8.2.5-1:

$A_{v\text{-req'd}} = 0.0316*f'c^{.5}b_v s/f_y$ = 0.15 in² = 0.0316 x (4.00 ksi ^ 0.5) x 18 in x (8 in x 60.00 ksi)

OK

Shear strength of Steel per (1) Eq. C5.8.3.3-1:

$V_s = A_v f_y d\cot\Theta/s$ = 40.09 kips = 0.39 in² x 60.00 ksi x 13.61 in x (1.00/ 8.00 in)

$\phi_v V_c = \phi_v*V_c+V_s$ = 124.35 kips = 0.9 x (98.08 kips + 40.09 kips)

V_u = 58.25 kips

OK

Check maximum spacing of transverse reinforcement:

Calculate shear stress:

Per Ref 1 - Eq 5.8.2.9-1:

vu = (Vu-ΦVp)/Φbd = 0.08 ksi = (58.25 kips - 0))/ (0.90 x 57 in x 13.61 in)

If vu < 0.125f'c then s_{max} = 0.8dv ≤ 24.0in (1) Eq 5.8.2.7-1

If vu ≥ 0.125f'c then s_{max} = 0.4dv ≤ 12.0in (1) Eq 5.8.2.7-2

v_u	<	0.125f'c
0.08 ksi	<	0.50 ksi

$0.8d_v$ = 10.89 in

$0.4d_v$ = 5.45 in

s_{max} = 10.89 in

s	<	smax
8.00 in	<	10.89 in

OK

Check Cracking Control for Negative Flexure:

Per (1) Eq. 5.7.3.4-1

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$$

Per (1) Eq. 5.7.3.4-1

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

d_c =	2.88 in =	2.00 in +	0.375 in+	0.50 in		
β_s =	1.27 =	1 +	(2.875 in/	(0.7 x	(18 in -	2.875 in)))
ρ = A_s/(b*d) =	0.0034 =	3.53 in²/	(57.00 in x	18.00 in)		
ρ*n =	0.027 =	0.0034 x	7.96			
k = $((\rho*n)^2+(2*\rho*n))^{.5}-\rho*n$ =	0.208 =	(0.027413^	2) +	((2 x	0.02741)^	
		0.5) -	0.0274			
j = 1-k/3 =	0.931 =	1 -	0.21/	3		

Use service moment for check of cracking

q_{seat} =	4.00 ksf					
w =	19.00 klf =	4.00 ksf x	4.75 ft			
Service Moment - Negative =	66.80 k-ft =	0.5 x	19.00 klf x	2.65 ft ^ 2		
f_{ss} = M_u/A_s*j*d =	17.90 ksi =	(66.80 k-ft x	12)/	(3.53 in² x	0.931 x	
		13.61 in)				
s ≤	25.00 in =	(700 x	1.00)/	(1.27 x	17.90 ksi) -	
		(2 x	2.88 in)			
Provided S =	7.32 in =	(57 in -	(2 x	2.00 in) -	(2 x	
		0.50 in) -	0.8 in)/	(8 -	1)	

OK

Check Cracking Control for Posative Flexure:

Per (1) Eq. 5.7.3.4-1

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$$

Per (1) Eq. 5.7.3.4-1

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

d_c =	2.88 in =	2.00 in +	0.38 in +	0.50 in		
β_s =	1.27 =	1 +	(2.88 in/	(0.7 x	(18 in -	2.875 in)))
ρ = A_s/(b*d) =	0.0046 =	3.53 in²/	(57.00 in x	13.61 in)		
ρ*n =	0.979 =	0.0046 x	214.95			
k = $((\rho*n)^2+(2*\rho*n))^{.5}-\rho*n$ =	0.729 =	(0.979114^	2) +	((2 x	0.97911)^	
		0.5) -	0.9791			
j = 1-k/3 =	0.757 =	1 -	0.73/	3		

Use service moment for check of cracking

q_{seat} =	4.00 ksf					
w =	19.00 klf =	4.00 ksf x	4.75 ft			
Service Moment - Positive =	109.18 k-ft =	0.125 x	19.00 klf x	6.78 ft ^ 2		
$f_{ss} = M_s/A_s{}^*j^*d$ =	35.97 ksi =	(109.18 k-ft x 13.61 in)	12)/	(3.53 in² x	0.757 x	
s ≤	9.56 in =	(700 x (2 x	1.00)/ 2.88 in)	(1.27 x	35.97 ksi) -	
Provided S =	7.32 in =	(57 in - 0.50 in) -	(2 x 0.8 in3)/	2.00 in) - (8 -	(2 x 1)	

OK

Design Temperature and Shrinkage Reinforcement

Per (1) 5.10.8, temperature and shrinkage steel shall be

$$A_s \ge \frac{1.30bh}{2(b+h)f_y} \qquad (5.10.8\text{-}1)$$

$$0.11 \le A_s \le 0.60 \qquad (5.10.8\text{-}2)$$

where:

A_s = area of reinforcement in each direction and each face (in.²/ft)
b = least width of component section (in.)
h = least thickness of component section (in.)
f_y = specified yield strength of reinforcing bars ≤75 ksi

b =	57.00 in			
h =	18.00 in			
f_y =	60 ksi			
b x h =	1026 in² =	57.00 in x	18.00 in	
b + h =	75.00 in =	57.00 in +	18.00 in	

$$A_s\ (min) = 0.148\ in^2 = \frac{1.30 \times 1026\ in^2}{2 \times 75.00\ in \times 60\ ksi}$$

Use 6 - No. 8 bars, spaced at 5" approx.

A_s = 3.53 in²

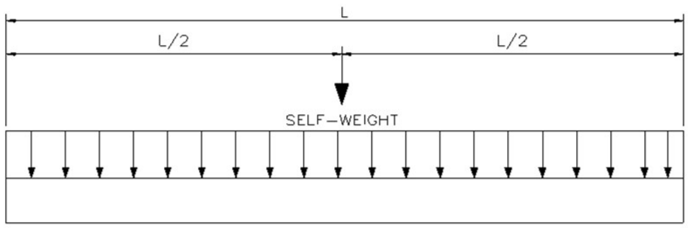
OK

PRECAST BRIDGE SEAT - SPAN 2 VTRANS IM 091-1(68)

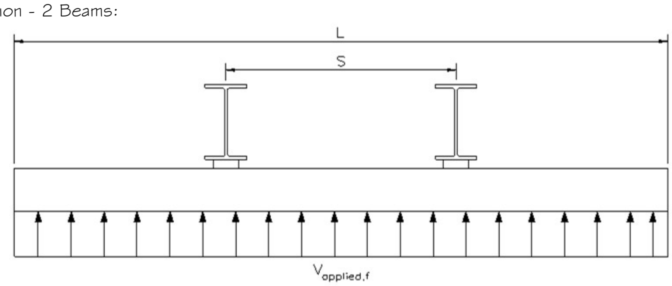
References:
(1) AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017 with interims thru 2018

Narrative:
The precast beam seat is to be designed as to act as a separate, indpentent member. The largest beam seat was used as the design case. The beam seat was subjected to an upward bearing pressure equal to the factored bearing pressure applied by dead and live loading, as calculated in the design of the GRS-IBS abutment. The bridge seat is designed for the controlling condition between the erection condition (assuming a single pick point in the center of the bridge seat, with only the self weight applied) and the final condition (bridge seat is subjected to the full applied bearing pressure). See sketch shown below.

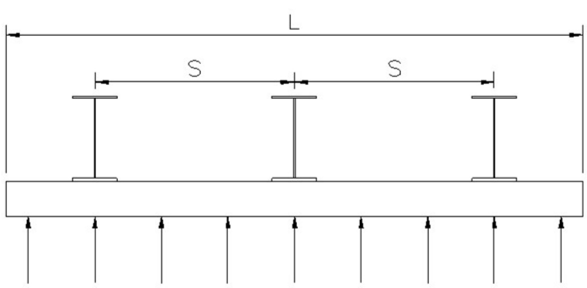
Erection Condition:



Final Condition - 2 Beams:



Final Condition - 3 Beams:



Bridge Seat Geometry

Note that the bridge seat thickness varies. Use the maximum thickness for the calculation of loads, and the minimum thickness for the design.

Bridge Seat Width =	4.75 ft
Bridge Seat Depth (Design) =	1.50 ft
Bridge Seat Depth (Loading) =	3.83 ft
Bridge Seat L (2 Beams) =	14.00 ft
Bridge Seat L (3 Beams) =	23.00 ft

Calculate Moment, Shear for Erection

Beam Seat Area =	18.19 ft² =	3.83 ft x	4.75 ft			
Weight of Cap =	2.73 klf =	18.19 ft² x	0.150 kcf			

Per (1) 3.4.2.1, apply a factor of 1.25 to construction loading to the weight of structure and appurtenances.

Picking Moment =	83.57 k-ft =	1.25 x	0.5 x	2.729 klf x	7.00 ft ^	2
Picking Shear =	23.88 kips =	1.25 x	2.73 klf x	(14.00 ft x		0.5)

Calculate Moment, Shear for Final Condition

Factored Upward Pressure =	4.00 ksf		
Beam Seat Width =	4.75 ft		
Uniform Distributed Load, w =	19.00 klf =	4.00 ksf x	4.75 ft

Final Condition - 2 Beams:

Bridge Skew =	28.66	degrees		
Beam Spacing =	6.67 ft			
L (Simple Span) =	7.60 ft =	6.67 ft/	cos(	28.66)
L (Cantilever) =	3.20 ft =	0.5 x	(14.00 ft -	7.60 ft)

Assume a simply supported position for bridge seat between beams, and a cantilever condition beyond beams.

Positive Moment =	137.10 k-ft =	0.125 x	19.00 klf x	7.60 ft ^ 2
Negative Moment =	97.35 k-ft =	0.5 x	19.00 klf x	3.20 ft ^ 2
Shear =	72.18 kips =	19 klf x	(7.60 ft x	0.50)

Final Condition - 3 Beams:

Bridge Skew =	28.66	degrees				
Beam Spacing =	6.67 ft					
L (Continuous Span) =	7.60 ft =	6.67 ft/	cos(	28.66)		
L (Cantilever) =	3.90 ft =	0.5 x	(23.00 ft -		(2 x	7.60 ft))
Positive Moment (Cont.) =	76.78 k-ft =	0.07 x	19.00 klf x	7.60 ft ^		2)
Negative Moment (Cont.) =	137.10 k-ft =	0.125 x	19.00 klf x	7.60 ft ^		2)
Shear (Cont.) =	90.22 kips =	0.625 x	(19.00 klf x	7.60 ft)		
Negative Moment (Cant.) =	144.66 k-ft =	0.5 x	19.00 klf x	3.90 ft ^ 2		
Shear (Cant.) =	74.14 kips =	19 klf x	3.90 ft			

Determine Controlling Forces

Positive Moment =	137.10 k-ft
Negative Moment =	144.66 k-ft
Shear =	90.22 kips

As the max shear occurs right at a support, take the design shear at a distance d_v away from support.

d_v =	1.17 ft =	14.06 ft/ 12		
Max. shear @ distance d_v =	67.96 kips =	90.22 kips -	(19.00 klf x	1.17 ft)

Calculate Design Loads

By inspection, the moments and shears applied to the bridge seat are governed by the final condition. Since the upward loading pressure of the final condition has been previously factored, the factor for the design moments shalle equal 1.0.

Design Shear =	67.96 kips =	1 x	67.96 kips
Design Positive Moment =	137.10 k-ft =	1 x	137.10 k-ft
Design Negative Moment =	144.66 k-ft =	1 x	144.66 k-ft

V_u = 67.96 kips
M_{u+} = 137.10 k-ft
M_{u-} = 144.66 k-ft

Design Negative Moment Reinforcement

For the purpose of the design, M_{u-} = 144.66 k-ft

f_y =	60 ksi			
f'_c =	4 ksi			
E_c =	3644.15 ksi			
E_s =	29000			
n =	7.96			
γ_e =	1	(1) 5.7.3.4		
b =	57.00 in			
Thickness, H =	18.00 in			
Cover =	2.00 in			
Negative Reinforcement =	8 -	No. 6		
D_{bar} =	0.75 in			
d =	15.625 in =	18.00 in -	2.00 in -	0.375 in
A_{bar} =	0.44 in²			
A_s =	3.53 in² =	8 x	0.44 in²	

Per (1),

$$M_n = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_s\left(d_s - \frac{a}{2}\right) - A'_s f'_s\left(d'_s - \frac{a}{2}\right) + \alpha_1 f'_c(b - b_w)h_f\left(\frac{a}{2} - \frac{h_f}{2}\right)$$

Check effective depth:

d_e = 15.63 in

$0.9{*}d_e$ = 14.0625 in
0.72*h = 12.96 in
d_v = max of $0.9{*}d_e$ and 0.72*h = 14.0625 in
$a = c\beta_1$
α_1 = 0.85
β_1 = 0.85

$$c = \frac{A_{ps}f_{pu} + A_s f_s - A'_s f'_s}{\alpha_1 f'_c \beta_1 b + kA_{ps}\dfrac{f_{pu}}{d_p}}$$

$$c = 1.29\text{ in} = \frac{3.53\text{ in}^2 \times 60\text{ ksi}}{0.85 \times 4\text{ ksi} \times 0.85 \times 57\text{ in}}$$

a = 1.09 in = 0.85 x 1.29

<u>Verify that $f_s = f_y$:</u>

c/d = 0.09 = 1.29 in/ 14.0625 in

c/d < 0.6, therefore $f_s = f_y$

Calculate Moment Strength M_n:

M_n = 238.84 k-ft = (3.53 in² x 60 ksi x (14.0625 in - (1.09/ 2))/ 12

Per (1) 5.5.4.2.1, Φ_b = 0.90

M_r = 214.95 k-ft = 0.90 x 238.84 k-ft
M_u = 144.66 k-ft
OK

Check that Section is Tension Controlled

∈'t > ∈'y = 0.60

∈'t = (d-c/c)*0.003 = 0.039 = (14.06 in - (1.29 in/ 1.29 in)) x 0.003
∈'y = 0.005
OK

Calculate M_{cr}:

Per (1) 5.4.2.6, $f_r = 0.37(f'_c)^{.5}$ = 0.74 ksi = 0.37 x 4.00 ksi ^ 0.5
S_c = 3078.00 in³ = 0.17 x 57 in x 18 in ^ 2

Per (1) 5.7.3.3.2, $M_{cr} = f_r{*}S_c$ = 189.81 k-ft = (0.74 ksi x 3078.00 in³)/ 12

Calculate Required Moment Capacity
$1.2{*}M_{cr}$ = 227.77 k-ft = 1.2 x 189.81 k-ft
$1.33{*}M_u$ = 192.40 k-ft = 1.33 x 144.66 k-ft
Min. Moment Strength = 192.40 k-ft
M_r = 214.95 k-ft
OK

Design Posative Moment Reinforcement

M_{u+} = 137.10 k-ft

f_y = 60 ksi
f'_c = 4 ksi
E_c = 3644.15 ksi
E_s = 29000
n = 7.96
γ_e = 1 (1) 5.7.3.4

b = 57.00 in
Thickness, H = 18.00 in
Cover = 2.00 in
Positive Reinforcement = 8 - No. 6
D_{bar} = 0.75 in
d = 15.125 in = 18.00 in - 2.00 in - 0.375 in 0.50 in
A_{bar} = 0.44 in²
A_s = 3.53 in² = 8 x 0.44 in²

Per (1),

$$M_n = A_{ps}f_{ps}\left(d_p - \frac{a}{2}\right) + A_s f_s\left(d_s - \frac{a}{2}\right) - A'_s f'_s\left(d'_s - \frac{a}{2}\right) + \alpha_1 f'_c(b - b_w)h_f\left(\frac{a}{2} - \frac{h_f}{2}\right)$$

d_e = 15.13 in
$0.9{*}d_e$ = 13.61 in
$0.72{*}h$ = 12.96 in
d_v = max of $0.9{*}d_e$ and $0.72{*}h$ = 13.61 in

$a = c\beta_1$

α_1 = 0.85

β_1 = 0.85

$$c = \frac{A_{ps}f_{pu} + A_s f_s - A'_s f'_s}{\alpha_1 f'_c \beta_1 b + kA_{ps}\dfrac{f_{pu}}{d_p}}$$

c =	1.29 in =	3.53 in² x	60 ksi		
		0.85 x	4 ksi x	0.85 x	57 in
a =	1.09 in =	0.85 x	1.29		

Verify that $f_s = f_y$:

c/d = 0.09 = 1.29 in/ 13.6125 in

c/d < 0.6, therefore $f_s = f_y$

Calculate Moment Strength M_n:

M_n = 230.88 k-ft = (3.53 in² x 60 ksi x (13.6125 in - (1.09/ 2))/ 12

Per (1) 5.5.4.2.1, Φ_b = 0.90

M_r = 207.80 k-ft = 0.90 x 230.88 k-ft

M_u = 137.10 k-ft

OK

Check that Section is Tension Controlled

∈'t > ∈'y = 0.60

∈'t = (d-c/c)*0.003 = 0.038 = (13.61 in - (1.29 in/ 1.29 in)) x 0.003

∈'y = 0.005

OK

Calculate M_{cr}:

Per (1) 5.4.2.6, $f_r = 0.37(f'_c)^{.5}$ = 0.74 ksi = 0.37 x 4.00 ksi ^ 0.5

S_c = 3078.00 in³ = 0.17 x 57 in x 18 in ^ 2

Per (1) 5.7.3.3.2, $M_{cr} = f_r{}^*S_c$ = 189.81 k-ft = (0.74 ksi x 3078.00 in³)/ 12

Calculate Required Moment Capacity

1.2*M_{cr} = 227.77 k-ft = 1.2 x 189.81 k-ft

1.33*M_u = 182.34 k-ft = 1.33 x 137.10 k-ft

Min. Moment Strength = 182.34 k-ft

M_r = 207.80 k-ft

OK

Design for Shear

Per (1) 5.8.3.4-1,

β =	2.0					
Θ =	45 =	0.78539816 rad				
d_e =	15.125 in =	18.00 in -	2.00 in -	0.50 in -	0.375 in	
0.9^*d_e =	13.61 in =	0.9 x	15.125 in			
0.72^*h =	12.96 in =	0.72 x	18.00 in			

Per (1) 5.8.2.9,

$d_v = \max(0.90^*d_e, 0.72^*h) =$ 13.61 in

Per (1) Eq. 5.8.3.3-3, calcualte V_c:

$V_c = 0.0316^*\beta^*f'_c{}^{.5}b^*d_v$ =	98.08 kips =	0.0316 x	2	4.00 ksi ^	0.5 x
		57 in x	13.61 in		
Per (1) 5.5.4.2.1, ϕ_v =	0.9				
$\phi_v{}^*V_c$ =	88.27 kips =	0.9 x	98.08 kips		

Per (1) 5.8.2.4, provide shear reinforcement if $V_u > 0.5^*\phi V_c$

$0.5^*\phi V_c$ =	44.13 kips =	0.5 x	88.27 kips
V_u =	67.96 kips		

SHEAR REINFORCEMENT REQUIRED

	Try
stirrup size =	#4
stirrup d_b =	0.50 in
stirrup A_b =	0.20 in²
# legs =	2
s =	8.00 in

A_v = 0.39 in² = 0.20 in² x 2

Per (1) Eq. 5.8.2.5-1:

$A_{v\text{-req'd}} = 0.0316^*f'c^{.5}b_s/f_y$ =	0.15 in² =	0.0316 x	(4.00 ksi ^	0.5) x	18 in x
		(8 in x	60.00 ksi)		

OK

Shear strength of Steel per (1) Eq. C5.8.3.3-1:

$V_s = A_vf_yd\cot\Theta/s$ =	40.09 kips =	0.39 in² x	60.00 ksi x	13.61 in x
		(1.00/	8.00 in)	
$\phi_vV_c = \phi_v{}^*V_c+V_s$ =	124.35 kips =	0.9 x	(98.08 kips +	40.09 kips)
V_u =	67.96 kips			

OK

Check maximum spacing of transverse reinforcement:

Calculate shear stress:

Per Ref 1 - Eq 5.8.2.9-1:

vu = (Vu-ΦVp)/Φbd =	0.10 ksi =	(67.96 kips -	0.9 x	0)/
		(0.90 x	57 in x	13.61 in)

If vu < 0.125f'c then s_{max} = 0.8dv ≤ 24.0in (1) Eq 5.8.2.7-1

If vu ≥ 0.125f'c then s_{max} = 0.4dv ≤ 12.0in (1) Eq 5.8.2.7-2

v_u	<	0.125f'c
0.10 ksi	<	0.50 ksi

$0.8d_v$ = 10.89 in
$0.4d_v$ = 5.45 in

s_{max} = 10.89 in

s	<	smax
8.00 in	<	10.89 in
	OK	

Check Cracking Control for Negative Flexure:

Per (1) Eq. 5.7.3.4-1

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$$

Per (1) Eq. 5.7.3.4-1

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

d_c =	2.88 in =	2.00 in +	0.38 in +	0.5 in		
β_s =	1.27 =	1 +	(2.875 in/	(0.7 x	(18 in -	2.875 in)))
ρ = A_s/(b*d) =	0.0034 =	3.53 in²/	(57.00 in x	18.00 in)		
ρ*n =	0.027 =	0.0034 x	7.96			
k = ((ρ*n)^2+(2*ρ*n))$^{.5}$-ρ*n =	0.208 =	(0.027413^	2) +	((2 x	0.02741)^	
		0.5) -	0.0274			
j = 1-k/3 =	0.931 =	1 -	0.21/	3		

Use service moment for check of cracking

q_{seat} =	3.66 ksf	See GRS Abutment Calculation			
w =	17.39 klf =	3.66 ksf x	4.75 ft		
Service Moment - Negative =	89.07 k-ft =	0.5 x	17.39 klf x	3.20 ft ^ 2	
f_{ss} = M_u/A_s*j*d =	23.88 ksi =	(89.07 k-ft x	12)/	(3.53 in² x	0.931 x
		13.61 in)			
s ≤	17.31 in =	(700 x	1.00)/	(1.27 x	23.88 ksi) -
		(2 x	2.88 in)		
Provided S =	7.41 in =	(57 in -	(2 x	2.00 in) -	(2 x
		0.20 in) -	0.8 in)/	(8 -	1)

OK

Check Cracking Control for Posative Flexure:

Per (1) Eq. 5.7.3.4-1

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$$

Per (1) Eq. 5.7.3.4-1

$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$

d_c =	2.88 in =	2.00 in +	0.38 in +	0.5 in		
β_s =	1.27 =	1 +	(2.88 in/	(0.7 x	(18 in -	2.875 in)))
ρ = A_s/(b*d) =	0.0046 =	3.53 in²/	(57.00 in x	13.61 in)		
ρ*n =	0.979 =	0.0046 x	214.95			
k = ((ρ*n)^2+(2*ρ*n))$^{.5}$-ρ*n =	0.729 =	(0.979114^ 0.5) -	2) + 0.9791	((2 x	0.97911)^	
j = 1-k/3 =	0.757 =	1 -	0.73/	3		

Use service moment for check of cracking

q_{seat} =	3.66 ksf	See GRS Abutment Calculation			
w =	17.39 klf =	3.66 ksf x	4.75 ft		
Service Moment - Positive =	125.45 k-ft =	0.125 x	17.39 klf x	7.60 ft ^ 2	
f_{ss} = M_u/A_s*j*d =	41.33 ksi =	(125.45 k-ft x 13.61 in)	12)/	(3.53 in² x	0.757 x
s ≤	7.57 in =	(700 x (2 x	1.00)/ 2.88 in)	(1.27 x	41.33 ksi) -
Provided S =	7.32 in =	(57 in - 0.50 in) -	(2 x 0.8 in3)/	2.00 in) - (8 -	(2 x 1)

OK

Design Temperature and Shrinkage Reinforcement

Per (1) 5.10.8, temperature and shrinkage steel shall be

$$A_s \ge \frac{1.30bh}{2(b+h)f_y} \qquad (5.10.8\text{-}1)$$

$$0.11 \le A_s \le 0.60 \qquad (5.10.8\text{-}2)$$

where:

A_s = area of reinforcement in each direction and each face (in.^{2}/ft)
b = least width of component section (in.)
h = least thickness of component section (in.)
f_y = specified yield strength of reinforcing bars ≤75 ksi

b =	57.00 in
h =	18.00 in
f_y =	60 ksi

b x h =	1026 in² =	57.00 in x	18.00 in
b + h =	75.00 in =	57.00 in +	18.00 in

$$A_s\,(min) = 0.148\text{ in}^2 = \frac{1.30 \times 1026\text{ in}^2}{2 \times 75.00\text{ in} \times 60\text{ ksi}}$$

Use 6 - No. 8 bars, spaced at 5" approx.

A_s = 3.53 in²

OK

GILL ENGINEERING

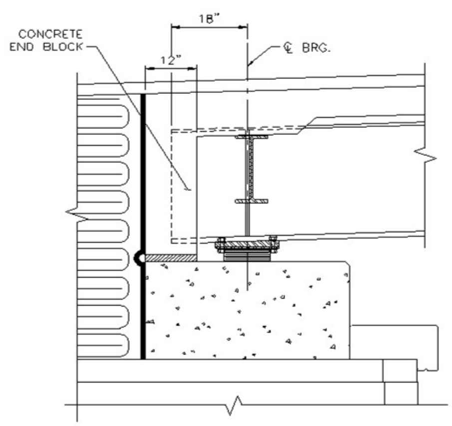
CLIENT VTrans
PROJECT Hartland IM 091-1(68)
BRIDGE NO. Bridge D37
SUBJECT Final Design

CALC BY JAV
CHECK BY 1st Set
DATE June 2019

References:
(1) AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017 with interims thru 2018

Design Narrative:
Concrete end block is monolithic with deck and is designed assuming a cantilever condition, resisting passive earth pressure from the GRS-IBS abutment. The cantilever length is assumed to be equal to the height of bearing assembly (bearing, sole plate, shim plate), plus the girder depth, plus the deck and haunch, less 2" (to account for assumed 2" top flange embedment). Backwall is designed per foot.

End Block Geomety



End Block Width = 12 in
Bearing Assembly = 6 in
Girder Depth = 36.125 in
Deck Thickness = 9 in
Haunch = 1.5 in
Cantilever Length = 4.22 ft = ((6 in + 36.125 in + 9 in + 1.5 in) - 2 in)/ 12

GRS Fill

Fill Height = 4.22 ft
Fill Unit Weight = 0.115 kcf
ϕ = 45 degrees

Calculate Passive Earth Pressure Load

K_p = 5.828 = $\tan^2($ 45 + (45/ 2))
Passive Earth Pressure = 5.965 kips/ ft = 0.5 x 0.115 kcf x (4.219 ft^ 2) x 5.828
Moment Arm = Cantilever Length - (Deck + Haunch + Top Flange + (1/3*Cantilever))
Moment Arm = 1.86 ft = 4.22 ft - ((9 in + 1.5 in + 0.875 in + 16.88 in)/ 12)
Earth Pressure Moment = 11.122 k-ft/ ft = 5.965 kips/ ft x 1.86 ft

Apply Load Factors
Per (2) Table 3.4.1-1 and Table 3.4.1-2, a load factor for passive earth pressure is not explicitly stated, conservatively use a factor of 1.50.

γ_P = 1.50
Design Moment = 16.68 k-ft/ ft = 1.50 x 11.122 kip-ft/ft
Design Shear = 8.95 kips/ft = 1.50 x 5.965 kips/ft

Design Endblock

F_y =	60 ksi					
f'_c =	4 ksi					
Bar Size =	No. 6					
d_b =	0.75 in					
Bar Spacing =	8 in					
A_b =	0.44 in²					
A_s =	0.66 in² =	0.44 in² x	(12 in/	8 in)		
Endblock Thickness =	12 in					
Cover =	2 in					
α_1 =	0.85					
β_1 =	0.85					
b =	4.22 ft					

$$c = 1.14\text{ in} = \frac{0.66\text{ in}^2/\text{ft} \times 60\text{ ksi}}{0.85 \times 4\text{ ksi} \times 0.85 \times 12\text{ in}}$$

Per (2) 5.7.3.2.3, $a = c^*\beta_1$

a =	0.97 in =	1.14 in x	0.85				
d_s =	9.63 in =	12 in -	2 in -	0.375 in			
M_n =	30.16 k-ft =	(0.66 in² x	60 ksi) x	(9.625 in -	(0.97 in/	2))	0.0833
ϕM_n =	27.14 k-ft =	0.9 x	30.16 k-ft				
M_u =	16.68 k-ft						

OK

Verify that $f_s = f_y$

Per (2) 5.7.2.1, for $f_s = f_y$, $c/d_s<0.6$

c/d_s = 0.119 = 1.14 in/ 9.63 in

OK

Verify Section is Tension Controlled

Per (2) Eq. 5.7.2.1-1,

$\varepsilon_t = ((d-c)/c)^*0.003$

ε_t = 0.022 = ((9.625 in - 1.14 in)/ 1.14 in) x 0.003

TENSION CONTROLS

FINAL DESIGN - CONCRETE END BLOCK VTRANS IM 091-1(68)

Check Min. Reinforcement

Per (2)

5.7.3.3.2 Minimum Reinforcement

Unless otherwise specified, at any section of a noncompression-controlled flexural component, the amount of prestressed and nonprestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, M_r, at least equal to the lesser of:

- 1.33 times the factored moment required by the applicable strength load combination specified in Table 3.4.1-1; and
- $$M_{cr} = \gamma_3\left[\left(\gamma_1 f_r + \gamma_2 f_{cpe}\right)S_c - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right)\right] \quad (5.7.3.3.2\text{-}1)$$

where:

f_r = modulus of rupture of concrete specified in Article 5.4.2.6

f_{cpe} = compressive stress in concrete due to effective prestress forces only (after allowance for all prestress losses) at extreme fiber of section where tensile stress is caused by externally applied loads (ksi)

M_{dnc} = total unfactored dead load moment acting on the monolithic or noncomposite section (kip-in.)

S_c = section modulus for the extreme fiber of the composite section where tensile stress is caused by externally applied loads (in.^{3})

S_{nc} = section modulus for the extreme fiber of the monolithic or noncomposite section where tensile stress is caused by externally applied loads (in.^{3})

1.33^*M_u = 22.19 k-ft = 16.68 k-ft x 1.33

Per (2) 5.4.2.6, $f_r = 0.37^*\text{sqrt}(f'_c)$

f_r =	0.48 ksi =	0.37 x	(4 ksi ^ 0.5)			
γ_1 =	1.60					
γ_2 =	0.00					
γ_3 =	0.75	(A706, Gr. 60 Reinf.)				
S_c =	288.00 in³ =	0.17 x	12 in x	12 in ^ 2		
M_{cr} =	23.96 k-ft =	1.3 x	((1.60 x	0.48 ksi) x	288 in³) x	0.083333
M_r =	22.19 k-ft					
$\phi_f M_n$ =	27.14 k-ft					

OK

Check Temperature and Shrinkage Steel

Per (1) 5.10.6, temperature and shrinkage steel shall be

$$A_s \geq \frac{1.30bh}{2(b+h)f_y} \quad (5.10.6\text{-}1)$$

except that:

$$0.11 \leq A_s \leq 0.60 \quad (5.10.6\text{-}2)$$

where:

A_s = area of reinforcement in each direction and each face (in.²/ft)
b = least width of component section (in.)
h = least thickness of component section (in.)
f_y = specified minimum yield strength of reinforcement ≤75.0 ksi

b = 50.63 in
h = 12.00 in
f_y = 60 ksi

b x h = 607.5 in² = 50.63 in x 12.00 in
b + h = 62.63 in = 50.63 in + 12.00 in

$$A_s\,(min) = 0.105\ in^2 = \frac{1.30 \times 607.5\ in^2}{2 \times 62.63\ in \times 60\ ksi}$$

Use No. 5 @ 12"

A_s = 0.31 in²

OK

Design for Shear:

b =	2.0			(2) 5.8.3.4-1
Q =	45 =	0.785398163	rad	(2) 5.8.3.4-1

Calculate Effective Depth:

d_s =	9.63 in =	12.00 in -	2.00 in -	0.38 in		
$d_e = A_s{*}f_y{*}d_s/A_sf_y = d_s$ =	9.63 in					
d_v =	9.14 in =	9.63 in -	(0.97 in /		2)	
$0.90{*}d_e$ =	8.66 in					
$0.72{*}h$ =	8.64 in					
d_v = max($0.90{*}d_e$, $0.72{*}h$) =	9.14 in					(1) 5.8.2.9

Using (1) Eq 5.7.3.3-3, Calculate V_c:

$V_c = 0.0316{*}b{*}f'_c{}^{.5}b{*}d_v$ =	58.48 kips =	0.0316 x	2.0	4.00 ksi ^	0.5 x
	50.625 in x	9.14 in			

Check against (1) Eq. 5.7.3.3-2:

$V_n = 0.25f'_cb_vd_v + V_p$

V_n =	462.7 kips =	0.25 x	4 ksi x	50.63 in x	9.14 in x
Controlling V_n =	58.48 kips				

Vu	>	0.5 Φ Vc
8.95 kips	<	26.32 kips
Transverse Reinforcement Not Required		

Check Crack Control:

$$s \le \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$$ (2) Eq 5.6.7-1

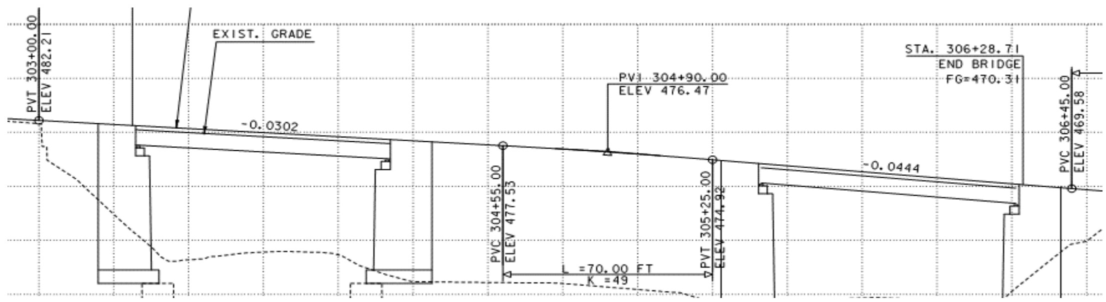
$$\beta_s = 1 + \frac{d_c}{0.7(h - d_c)}$$ (2) Eq 5.6.7-2

E_s =	29000 ksi						
E_c =	3644.15 ksi						
n =	7.96 ksi	29000 ksi/	3644.15 ksi				
d_c =	2.75 in =	2 in +	0.75 in				
b_s =	1.42 =	1 +	(2.75 in/	(0.7 x	(12 in -	2.75 in)))	
$r = A_s/(b{*}d)$ =	0.0011 =	0.66 in2/	(50.625 in x	12 in)			
$r{*}n$ =	0.0086 =	0.0011 x	7.96				
$k = ((r{*}n)^2+(2{*}r{*}n))^{.5}-r{*}n$ =	0.046 =	(0.00108642 ^	2) +	((2 x	0.001086) ^	0.5) -	0.0011
$j = 1-k/3$ =	0.985 =	1 -	0.05/	3			
$f_{ss} = M_s/A_s{*}j{*}d$ =	22.47 ksi =	(11.12 k-ft x	12)/	(0.66 in² x	0.985 x	9.14 in)	
spacing ≤	16.37 in =	(700 x	1.00)/	(1.42 x	22.47 ksi) -		
		(2 x	2.75 in)				
Given Spacing =	8 in						

OK

BRIDGE LAYOUT AND GEOMETRICS CALCULATIONS

Hartland Beamseat elevation calc - Abutment 1



Base point for span 1

STA.	30300	
ELEV	482.21	
Slope	-0.0302	

Distance from base point to point along alignment which is perpendicular to CL BRG.

G1	34.57 ft	*From Sketch*
G2	33.74 ft	*From Sketch*
G3	32.93 ft	*From Sketch*
G4	32.13 ft	*From Sketch*
G5	31.35 ft	*From Sketch*

Elevation of the points described above (using slope along alignment)

G1	481.17
G2	481.19
G3	481.22
G4	481.24
G5	481.26

perpendicular length from CL BRG. to the alignment (- is sloped down, + is sloped up)

G1	-12.87	*From Sketch*
G2	-6.13	*From Sketch*
G3	0.60	*From Sketch*
G4	7.34	*From Sketch*
G5	14.07	*From Sketch*

Cross slope 0.0625

Elevations at CL of BRGS at top of road

G1	480.36
G2	480.81
G3	481.25
G4	481.70
G5	482.14

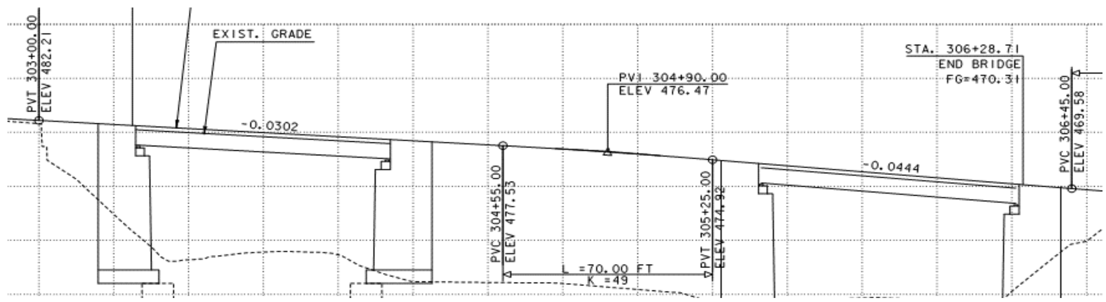
Next subtract structure depth to get the beam seat elevation

	Depth	
Deck	9.00	inches
Haunch	1.50	inches
Plate Girder	36.13	inches
Sole Plate	1.50	inches
Bearing Pad	3.00	inches
Camber factor	1.05	inches
Structure	52.18	inches
Structure	**4.35**	ft

Beamseat Elevations

G1	**476.01**
G2	**476.46**
G3	**476.91**
G4	**477.35**
G5	**477.79**

Hartland Beamseat elevation calc - Abutment 2



Base point for span 1

STA.	30300
ELEV	482.21
Slope	-0.0302

Distance from base point to point along alignment which is perpendicular to CL BRG.

G1	120.22	ft	*From Sketch*
G2	118.51	ft	*From Sketch*
G3	116.84	ft	*From Sketch*
G4	115.21	ft	*From Sketch*
G5	113.60	ft	*From Sketch*

Elevation of the points described above (using slope along alignment)

G1	478.58
G2	478.63
G3	478.68
G4	478.73
G5	478.78

perpendicular length from CL BRG. to the alignment (- is sloped down, + is sloped up)

G1	-12.52	*From Sketch*
G2	-5.95	*From Sketch*
G3	0.62	*From Sketch*
G4	7.20	*From Sketch*
G5	13.78	*From Sketch*

Cross slope	0.0625

Elevations at CL of BRGS at top of road

G1	477.80
G2	478.26
G3	478.72
G4	479.18
G5	479.64

Next subtract structure depth to get the beam seat elevation

	Depth	
Deck	9.00	inches
Haunch	1.50	inches
Plate Girder	36.13	inches
Sole Plate	1.50	inches
Bearing Pad	3.00	inches
Camber factor	1.05	inches
Structure	52.18	inches
Structure	**4.35**	ft

Beamseat Elevations

G1	**473.45**
G2	**473.91**
G3	**474.37**
G4	**474.83**
G5	**475.29**

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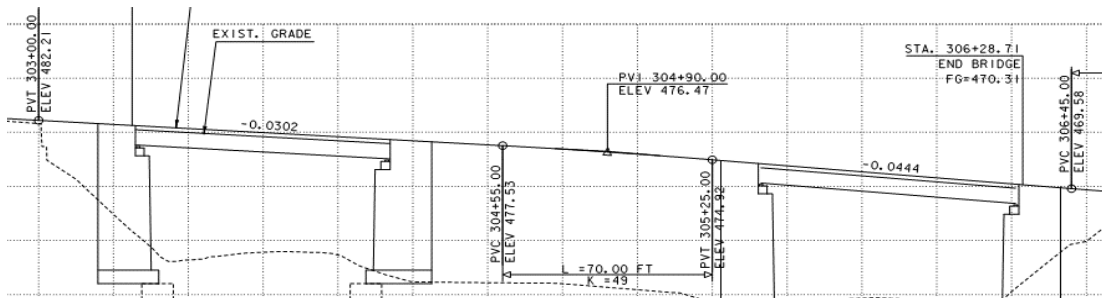
CLIENT VTrans
PROJECT Hartland IM 091-1(68)
BRIDGE NO. Bridge D37
SUBJECT Final Design

CALC BY YS
CHECK BY 1st Set
DATE June 2019

Hartland Beamseat elevation calc - Abutment 3



Base point for span 1

STA.	30525
ELEV	474.92
Slope	-0.0444

Distance from base point to point along alignment which is perpendicular to CL BRG.

G1	22.32	ft	*From Sketch*
G2	19.02	ft	*From Sketch*
G3	15.79	ft	*From Sketch*
G4	12.63	ft	*From Sketch*
G5	9.53	ft	*From Sketch*

Elevation of the points described above (using slope along alignment)

G1	473.93
G2	474.08
G3	474.22
G4	474.36
G5	474.50

perpendicular length from CL BRG. to the alignment (- is sloped down, + is sloped up)

G1	-13.30	*From Sketch*
G2	-6.43	*From Sketch*
G3	0.45	*From Sketch*
G4	7.35	*From Sketch*
G5	14.26	*From Sketch*

Cross slope 0.0625

Elevations at CL of BRGS at top of road

G1	473.10
G2	473.67
G3	474.25
G4	474.82
G5	475.39

Next subtract structure depth to get the beam seat elevation

	Depth	
Deck	9.00	inches
Haunch	1.50	inches
Plate Girder	36.13	inches
Sole Plate	1.50	inches
Bearing Pad	3.00	inches
Camber factor	1.08	inches
Structure	52.20	inches
Structure	**4.35**	ft

Beamseat Elevations

G1	**468.75**
G2	**469.32**
G3	**469.90**
G4	**470.47**
G5	**471.04**

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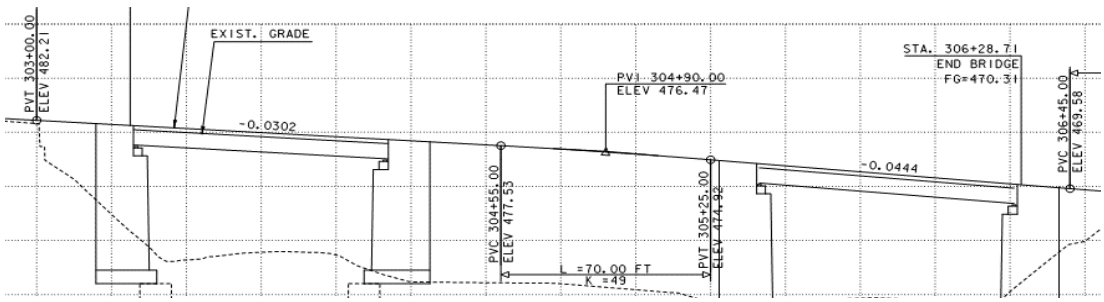
Hartland Beamseat elevation calc - Abutment 4



Base point for span 1

STA.	30525
ELEV	474.92
Slope	-0.0444

Distance from base point to point along alignment which is perpendicular to CL BRG.

G1	110.01	ft	*From Sketch*
G2	105.82	ft	*From Sketch*
G3	101.72	ft	*From Sketch*
G4	97.69	ft	*From Sketch*
G5	93.74	ft	*From Sketch*

Elevation of the points described above (using slope along alignment)

G1	470.04
G2	470.22
G3	470.40
G4	470.58
G5	470.76

perpendicular length from CL BRG. to the alignment (- is sloped down, + is sloped up)

G1	-12.34	*From Sketch*
G2	-5.69	*From Sketch*
G3	0.44	*From Sketch*
G4	6.87	*From Sketch*
G5	13.32	*From Sketch*

Cross slope	0.0625

Elevations at CL of BRGS at top of road

G1	469.26
G2	469.87
G3	470.43
G4	471.01
G5	471.59

Next subtract structure depth to get the beam seat elevation

	Depth	
Deck	9.00	inches
Haunch	1.50	inches
Plate Girder	36.13	inches
Sole Plate	1.50	inches
Bearing Pad	3.00	inches
Camber factor	1.08	inches
Structure	52.20	inches
Structure	**4.35**	ft

Beamseat Elevations

G1	**464.91**
G2	**465.52**
G3	**466.08**
G4	**466.66**
G5	**467.24**

Vertical Under Clearance | I-91 Southbound | Hartland

Calculation Narrative:
Minimum allowable vertical clearance for I-91 is 16'-6". The following calculations determine the proposed vertical under clearances.

Superstructure Depth:

Wearing Surface =	0.00 in
Deck =	9.00 in
Haunch =	1.50 in
Girder =	36.13 in
	46.63 in

TH 41 Profile Information:

PVT Station =	30300
PVT Elevation =	482.21 ft
Grade =	-3.02%
Cross Slope =	-6.25%

I-91West Curbline:

TH 41 Elevation:	Station =	30362.06438				
	PGL Elevation =	480.34 ft				
	PGL Offset =	13.94 ft				
	Top of Roaway Elevation =	479.46 ft				
	Bottom of Beam Elevation =	475.58 ft				
I-91 Elevation:	Northing =	379283.07				
	Easting =	1669774.52				
	Existing I-91 Elevation =	457.15 ft	(Taken from Survey Existing Surface File)			
	Available Clearance =	18.43 ft		>	16.50 ft	OK

I-91 West Laneline:

TH 41 Elevation:	Station =	30370.69				
	PGL Elevation =	480.08 ft				
	PGL Offset =	14.05 ft				
	Top of Roaway Elevation =	479.20 ft				
	Bottom of Beam Elevation =	475.31 ft				
I-91 Elevation:	Northing =	379277.99				
	Easting =	1669781.27				
	Existing I-91 Elevation =	457.75 ft	(Taken from Survey Existing Surface File)			
	Available Clearance =	17.56 ft		>	16.50 ft	OK

Vertical Under Clearance — I-91 Southbound — Hartland

I-91 Crownline:

	Station =	30383.53			
	PGL Elevation =	479.69 ft			
	PGL Offset =	14.02 ft			
	Top of Roaway Elevation =	478.81 ft			
	Bottom of Beam Elevation =	474.93 ft			
	Northing =	379270.43			
	Easting =	1669791.30			
	Existing I-91 Elevation =	458.08 ft	(Taken from Survey Existing Surface File)		
	Available Clearance =	16.85 ft	>	16.50 ft	OK

I-91 East Laneline:

TH 41 Elevation:	Station =	30396.52			
	PGL Elevation =	479.30 ft			
	PGL Offset =	13.72 ft			
	Top of Roaway Elevation =	478.44 ft			
	Bottom of Beam Elevation =	474.55 ft			
I-91 Elevation:	Northing =	379262.78			
	Easting =	1669801.45			
	Existing I-91 Elevation =	457.91 ft	(Taken from Survey Existing Surface File)		
	Available Clearance =	16.64 ft	>	16.50 ft	OK

I-91 East Curbline

TH 41 Elevation:	Station =	30400.78			
	PGL Elevation =	479.17 ft			
	PGL Offset =	13.57 ft			
	Top of Roaway Elevation =	478.32 ft			
	Bottom of Beam Elevation =	474.43 ft			
I-91 Elevation:	Northing =	379260.27			
	Easting =	1669804.78			
	Existing I-91 Elevation =	457.74 ft	(Taken from Survey Existing Surface File)		
	Available Clearance =	16.69 ft	>	16.50 ft	OK

Controlling Clearance = 16.64 ft

Vertical Under Clearance | I-91 Northbound | Hartland

Calculation Narrative:
Minimum allowable vertical clearance for I-91 is 16'-6". The following calculations determine the proposed vertical under clearances.

Superstructure Depth:

Wearing Surface =	0.00 in
Deck =	9.00 in
Haunch =	1.50 in
Girder =	36.13 in
	46.63 in

TH 41 Profile Information:

PVT Station =	30525
PVT Elevation =	474.92 ft
Grade =	-4.44%
Cross Slope =	-6.25%

I-91 West Curbline:

TH 41 Elevation:	Station =	30567.75			
	PGL Elevation =	473.02 ft			
	PGL Offset =	14.10 ft			
	Top of Roaway Elevation =	472.14 ft			
	Bottom of Beam Elevation =	468.26 ft			
I-91 Elevation:	Northing =	379141.693			
	Easting =	1669916.63			
	Existing I-91 Elevation =	449.14 ft	(Taken from Survey Existing Surface File)		
	Available Clearance =	19.12 ft	>	16.50 ft	OK

I-91 West Laneline:

TH 41 Elevation:	Station =	30572.82			
	PGL Elevation =	472.80 ft			
	PGL Offset =	14.20 ft			
	Top of Roaway Elevation =	471.91 ft			
	Bottom of Beam Elevation =	468.02 ft			
I-91 Elevation:	Northing =	379137.62			
	Easting =	1669919.46			
	Existing I-91 Elevation =	449.34 ft	(Taken from Survey Existing Surface File)		
	Available Clearance =	18.68 ft	>	16.50 ft	OK

Vertical Under Clearance — I-91 Northbound — Hartland

I-91 Crownline:

	Station =	30587.27			
	PGL Elevation =	472.16 ft			
	PGL Offset =	14.29 ft			
	Top of Roaway Elevation =	471.26 ft			
	Bottom of Beam Elevation =	467.38 ft			
	Northing =	379126.016			
	Easting =	1669927.53			
	Existing I-91 Elevation =	449.61 ft	(Taken from Survey Existing Surface File)		
	Available Clearance =	17.77 ft	>	16.50 ft	OK

I-91 East Laneline

TH 41 Elevation:	Station =	30601.67			
	PGL Elevation =	471.52 ft			
	PGL Offset =	14.06 ft			
	Top of Roaway Elevation =	470.64 ft			
	Bottom of Beam Elevation =	466.75 ft			
I-91 Elevation:	Northing =	379114.42			
	Easting =	1669935.60			
	Existing I-91 Elevation =	449.37 ft	(Taken from Survey Existing Surface File)		
	Available Clearance =	17.38 ft	>	16.50 ft	OK

I-91 East Curbline

TH 41 Elevation:	Station =	30611.64			
	PGL Elevation =	471.07 ft			
	PGL Offset =	13.72 ft			
	Top of Roaway Elevation =	470.22 ft			
	Bottom of Beam Elevation =	466.33 ft			
I-91 Elevation:	Northing =	379106.432			
	Easting =	1669941.16			
	Existing I-91 Elevation =	449.02 ft	(Taken from Survey Existing Surface File)		
	Available Clearance =	17.31 ft	>	16.50 ft	OK

Controlling Clearance = 17.31 ft