CALCULATION INDEX

Gannett Fleming
Engineers and Architects, P.C.
Excellence Delivered As Promised

ISO 9001:2008 CERTIFIED

PROJECT: **HAM-75-7.85**
BRIDGE NO: **HAM-75-PROS**
Norfolk Southern RR over Prosser Rd.
DATE: **5/9/16 1:37 PM**

ODOT PID NO: *77889*
GF JOB NO: **54682**
UPDATED BY: VDT
UPDATED DATE: May 09, 2016

CALCULATION NUMBER	CALCULATION SUBJECT (Root Folder #)	Current REV #	DATE	Calc	Check	Update	Verify	Scanned Check	NOTES	Overall Pg. No.
HAM075_PROS_01	(10) Geometry: Top of Rail EL, Bottom of Beam EL, Vertical Clearances	02	04/25/16	CTM	VDK	VDT	CTM	Digital	*Alignments from PAVR*	3
HAM075_PROS_02	(35) Girder: Section Properties	03	04/25/16	EFD	CTM	VDT	CTM	Digital	*Includes Proportion Check*	12
HAM075_PROS_03	(35) Girders: Design Loads, Stresses, and Deflections	03	04/25/16	EFD	VDT	VDT	CTM	Digital	*Spreadsheet Linked to PROS_02*	19
HAM075_PROS_04	(15) Loads: Abutment Design Loads	01	06/16/14	EFD	VDK	EFD	VDK	Digital	*Spreadsheet Linked to PROS_03 & PROS_01*	39
HAM075_PROS_05	(25) Abutments: Abutment Analysis (Abut5)	01	06/16/14	EFD	VDK	EFD	VDK	Digital	*Spreadsheet Linked to PROS_04 & PROS_01*	45
HAM075_PROS_06	(15) Loads: Retaining Wall Design Loads	01	06/16/14	VDT	CTM	CTM	VDT	Digital	*Lateral Earth Pressures from Train Surcharge - SE (Controlling) Wingwall and top of Caisson Wall*	55
HAM075_PROS_06C	(15) Loads: Retaining Wall Design Loads	01	06/16/14	VDT	CTM	VDT	CTM	Digital	*Lateral Earth Pressures from Train Surcharge - SW Caisson Wall*	62
HAM075_PROS_07	(25) Abutments: Retaining Wall Analysis SE	01	06/23/14	VDK	CTM	VDK	CTM	Digital	*Three wingwall sections are designed.*	69
HAM075_PROS_08P	(25) Abutments: Retaining Wall Analysis: SW	01	06/16/14	VDT	CTM	VDT	CTM	Digital	*Caisson Wall Design, Permanent Condition*	91
HAM075_PROS_08T	(23) Shoring: SW Caisson Wall	00	06/16/14	VDT	CTM	VDT	CTM	Digital	*Caisson Wall Design, Temporary Condition*	99
HAM075_PROS_09B	(23) Shoring: Sheeet Pile Walls	00	11/19/14	SNH	CTM	CTM	VDT	Digital	*Braced Sheeting Design*	107
HAM075_PROS_09U	(23) Shoring: Sheeet Pile Walls	00	04/25/16	CTM	VDT	VDT	CTM	Digital	*Unbraced Sheeting Design*	132

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CALCULATION NUMBER	CALCULATION SUBJECT (Root Folder #)	Current REV #	DATE	Calc	Check	Update	Verify	Scanned Check	NOTES	Overall Pg. No.
HAM075_PROS_10	(45) Connection Details: Girder Connections	00	04/25/16	VDK	VDT	VDT	CTM	Digital	*Misc.Connections*	176
HAM075_PROS_11	(40) Bearings: Bearing Design	00	08/12/14	VDK	CTM	VDK	EFD	Digital	*Sliding Bronze and Bolster*	205
HAM075_PROS_12	(50) Deck: Concrete Deck Design	01	04/25/16	VDK	CTM	VDT	CTM	Digital	*Composite*	211
HAM075_PROS_13	(45) Connection Details: Shear Studs	01	04/25/16	VDK	CTM	VDT	CTM	Digital		224
HAM075_PROS_14	(60) Quantities: Project Estimated Quantities	01	04/25/16	SNH	CTM	VDT	CTM	Digital		232
HAM075_PROS_15	(25) Abutments: Retaining wall analysis: Caisson wingwall	00	11/19/14	VDK	VDT	SNH	VDT	Digital	*Designed for wingwall stem alone.*	240
HAM075_PROS_PP	(21) Foundations: Pipe Pile Calculation	00	07/17/14	EFD	VDT	N/A	N/A	Digital		248

DESIGN CALCULATION COVER SHEET

Gannett Fleming
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ISO 9001:2008 CERTIFIED

PROJECT: **HAM-75-7.85**	*ODOT PID NO: 77889*
BRIDGE NO: **HAM-75-Prosser**	*EMH&T JOB NO:*
SUBJECT: ***Elevations***	**GF JOB NO: 54682**
PGL, Bottom of Beam	CALC NO.: **HAM075_PROS_01**
and Under-Clearances	REV NO.: **00**
FILE NAME(s): *HAM075_PROS_01.xlsx*	**SHEET: 1 OF 9**

SUPERSEDES CALC NO: N/A (Rev. 00)

CALCULATION DESCRIPTION:

This will calculate points along the vertical profile of the alignment based upon standard geometric calculations.

This will calculate bottom of girder and bottom of bearing elevations.

This will calculate vertical clearances under the structure.

DESIGN BASIS OR REFERENCES:

EMH&T Suppllied Survey Basemaps
GF Rail Alignment Plan and Profile
GF Proposed Stage 1 Bridge Layout

MISCELLANEOUS NOTES:

PGL is assumed to be the top of low rail

Prosser top of pavement elevations are as per survey interpolation.

EFD 12/18/13 Update: Girder bottom flanges are equal in elevation across the skew; resulting in level substructure caps.

VDK 5/22/14 Update: Profile accepted by NS. Revised stationing & elevations for new layout

EFD 6/3/14 Update: Added screed and final deck elevation tables

CTM 7/9/14 Check: Screed and final deck elevation tables

JEA Verify: Final verification of stations/offsets of key components

RECORD OF ISSUES:

REV	DESCRIPTION	BY	DATE	VERIF'D	DATE
00	Stage 1 Calculation	CTM	10/24/13	EFD	12/18/13
01	Final Geometry (Stage 2)	VDK	05/22/14		
02	Stage 3 Update	VDT	01/20/16	CTM	04/25/16

RECORD OF CHECK OF CURRENT CALCULATION VERSION

BY	DATE	CHKD	DATE	UPD.	DATE	VERIF'D	DATE
CTM	10/23/13	VDK	10/29/13	VDK	05/22/14	JEA	07/11/14
				VDT	01/20/16	CTM	04/25/16

7/10/2014 GEOPACK OUTPUT OF HORIZONTAL AND VERTICAL ALIGNMENTS ACROSS THE PROSSER BRIDGE

```
<*       1   DESCRIBE CHAIN RRMLP42                    (HORIZONTAL ALIGNMENT)

Chain RRMLP42 contains:
 35 36 SPI S9 CUR C53 SPI S10 SPI S11 CUR C54 SPI S12 PIMLP11

Beginning chain RRMLP42 description
===============================================================================

Point 35              N     435,177.9979 E   1,406,028.5403 Sta     143+95.95

Course from 35 to 36 N 12° 39' 36.37" E Dist 326.0400

Point 36              N     435,496.1110 E   1,406,099.9975 Sta     147+21.99

Course from 36 to TS S9 N 12° 39' 36.38" E Dist 1.9754

Spiral S9                 Type 1    Spiral Element

Angle       1° 48' 02.93" (LT) P             0.1100   BK N  12° 39' 36.37" E
LS              42.0000        K            20.9993   AH N  10° 51' 33.44" E
R              668.1483        LT           28.0014   CB N  12° 03' 35.41" E
YS               0.4400        ST           14.0013   Defl        0° 36' 00.96"
XS              41.9959        LC           41.9982   Deg         8° 34' 31.11"


                        Spiral Coordinates
                       *------------------*
Point           North                   East                  Station
-----    -------------------    -------------------     -------------------
TS              435,498.0383         1,406,100.4304               147+23.96
PI              435,525.3590         1,406,106.5674               147+51.97
SC              435,539.1096         1,406,109.2052               147+65.96
CC              435,664.9871         1,405,453.0216

                                  Curve Data
                                 *----------*
Curve C53                 (Chord Definition)
P.I.  Station           149+31.39  N        435,701.5694  E      1,406,140.3704
Delta       =      27° 48' 41.96" (LT)
Degree      =       8° 35' 00.00"
Tangent     =            165.4221
Length      =            324.0193
Radius      =            668.1483
External    =             20.1733
Long Chord  =            321.1478
Mid. Ord.   =             19.5821
P.C.  Station           147+65.96  N        435,539.1096  E      1,406,109.2052
P.T.  Station           150+89.98  N        435,859.8035  E      1,406,092.1371
C.C.                               N        435,664.9871  E      1,405,453.0216
Back        = N  10° 51' 33.44" E
Ahead       = N  16° 57' 08.52" W
Chord Bear  = N   3° 02' 47.54" W
```

Spiral S10 Type 2 Spiral Element

Angle	1° 48' 02.93" (LT)	P	0.1100	BK N	16° 57' 08.53" W
LS	42.0000	K	20.9993	AH N	18° 45' 11.46" W
R	668.1483	LT	28.0014	CB N	18° 09' 10.50" W
YS	0.4400	ST	14.0013	Defl	0° 36' 00.96"
XS	41.9959	LC	41.9982	Deg	8° 34' 31.11"

Spiral Coordinates

Point	North	East	Station
CS	435,859.8035	1,406,092.1371	150+89.98
PI	435,873.1965	1,406,088.0547	151+03.99
ST	435,899.7114	1,406,079.0524	151+31.98
CC	435,664.9871	1,405,453.0216	

Course from ST S10 to TS S11 N 18° 45' 11.46" W Dist 138.8565

Spiral S11 Type 1 Spiral Element

Angle	2° 24' 02.41" (LT)	P	0.2165	BK N	18° 45' 11.46" W
LS	62.0000	K	30.9982	AH N	21° 09' 13.87" W
R	739.8643	LT	41.3371	CB N	19° 33' 12.22" W
YS	0.8658	ST	20.6701	Defl	0° 48' 00.76"
XS	61.9891	LC	61.9952	Deg	7° 44' 38.74"

Spiral Coordinates

Point	North	East	Station
TS	436,031.1963	1,406,034.4112	152+70.84
PI	436,070.3389	1,406,021.1216	153+12.18
SC	436,089.6162	1,406,013.6623	153+32.84
CC	435,822.6187	1,405,323.6540	

Curve Data

Curve C54 (Chord Definition)

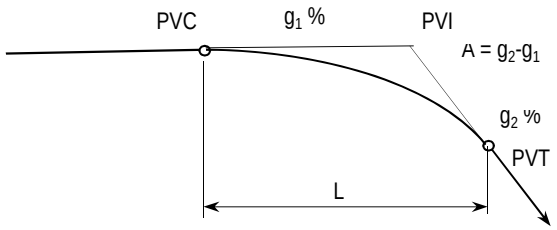
P.I. Station	156+33.68	N	436,370.1831	E	1,405,905.0975
Delta =	44° 15' 16.82" (LT)				
Degree =	7° 45' 00.00"				
Tangent =	300.8390				
Length =	571.0280				
Radius =	739.8643				
External =	58.8242				
Long Chord =	557.3638				
Mid. Ord. =	54.4917				
P.C. Station	153+32.84	N	436,089.6162	E	1,406,013.6623
P.T. Station	159+03.87	N	436,495.3759	E	1,405,631.5452
C.C.		N	435,822.6188	E	1,405,323.6540
Back =	N 21° 09' 13.87" W				
Ahead =	N 65° 24' 30.68" W				
Chord Bear =	N 43° 16' 52.28" W				

```
Spiral S12                Type 2    Spiral Element

Angle       2° 24' 02.41" (LT) P            0.2165   BK N  65° 24' 30.68" W
LS              62.0000        K           30.9982   AH N  67° 48' 33.09" W
R              739.8643        LT          41.3371   CB N  67° 00' 32.33" W
YS               0.8658        ST          20.6701   Defl        0° 48' 00.76"
XS              61.9891        LC          61.9952   Deg         7° 44' 38.74"


                        Spiral Coordinates
                       *------------------*
Point           North                   East                  Station
-----    ------------------    ------------------     ------------------
CS              436,495.3759        1,405,631.5452              159+03.87
PI              436,503.9777        1,405,612.7499              159+24.54
ST              436,519.5904        1,405,574.4745              159+65.87
CC              435,822.6188        1,405,323.6540

Course from ST S12 to PIMLP11 N 67° 48' 33.09" W Dist 766.4915

Point PIMLP11         N     436,809.0883 E   1,404,864.7562 Sta     167+32.36

==============================================================================
Ending chain RRMLP42 description


<*       2   PRINT PROFILE RRMLP_PG                  (HORIZONTAL ALIGNMENT)


Beginning profile RRMLP_PG description:
==============================================================================

              STATION       ELEV      GRADE    TOTAL L    BACK L    AHEAD L

VPI      1    145+61.79   549.9304

VPC           148+03.11   549.7737    -0.0650  K =  159.2
Low Point     148+13.46   549.7703
VPI      2    149+43.11   549.6827             280.0000  140.0000  140.0000
VPT           150+83.11   552.0538     1.6937

VPC           153+70.00   556.9127     1.6937  K =  105.4  SSD = 484.3
High Point    155+48.52   558.4245
VPI      3    155+70.00   560.3000             400.0000  200.0000  200.0000
VPT           157+70.00   556.0976    -2.1012

VPC           164+07.81   542.6959    -2.1012  K =  386.1  SSD = 4215.9
VPI      4    164+57.81   541.6453             100.0000   50.0000   50.0000
VPT           165+07.81   540.4652    -2.3602

VPI      5    166+00.00   538.2893    -2.3602

==============================================================================
Ending profile RRMLP_PG description
```

Gannett Fleming	GF JOB # 54682	CALC NO.: **HAM075_PROS_01**	REV. NO.: **01**
		PROJECT: **HAM-75-7.85**	

ODOT PID NO.: 77889		DESIGNED: *CTM*	DATE: *10/24/13*
BRIDGE NO: *HAM-75-Prosser*		CHECKED: *VDK*	DATE: *10/29/13*
SUBJECT: *Elevations*		UPDATED: *VDK*	DATE: *05/22/14*
PGL Data		VERIFIED: *CTM*	DATE: *05/28/14*
		FINAL VERIFY: *JEA*	DATE: *07/11/14*

PGL Vertical Curve & Profile Data (MLP)



1: SYMMETRICAL PARABOLIC CURVE		2: STRAIGHT GRADE	
L =	**400**	L =	**637.81**
g1 =	**1.6900%**	g =	**-2.1000%**
g2 =	**-2.1000%**		
PVI Station	**155+70.00**		
PVI Elev =	**560.300**		
PVC Station =	153+70.00	PVC Station =	**157+70.00**
PVC Elev =	556.920	PVC Elev =	**556.100**
PVT Station =	157+70.00	PVT Station =	**164+07.81**
PVT Elev =	556.100	PVT Elev =	**542.706**
Location & El of High Point=	155+48.36 558.427		
Chord elev @ PVI =	556.510		
Tangent offset at PVI (e) =	-1.8950		

Miscellaneous Point Entry

1: SYMMETRICAL PARABOLIC CURVE			2: STRAIGHT GRADE		
Station	Location Description	PGL Elevation	Station	Location Description	PGL Elevation
			158+77.95	CL NS = CL Prosser 7+25.16	553.83
			158+56.25	Left Girder @ Rear Prosser Curb	554.29
			158+63.87	Right Girder @ Rear Prosser Curb	554.13
			158+74.12	Left Girder @ Prosser Crown	553.91
			158+81.29	Right Girder @ Prosser Crown	553.76
			158+91.99	Left Girder @ Forward Prosser Curb	553.54
			158+98.71	Right Girder @ Forward Prosser Curb	553.40
			158+45.94	Rear Working Point	554.51
			159+14.20	Forward Working Point	553.07
			159+20.54	FWD ABUT SHLDR	550.69
			159+11.40	FWD ABUT SHLDR	550.88
			158+50.01	REAR ABUT SHLDR	552.17
			158+37.80	REAR ABUT SHLDR	552.43

Skew

Substructure units are skewed in relation to the horizontal curvature of the track alignment. As such; the actual station of C/L bearing for each fascia beam will be entered; as to allow for an exact PGL calculation for each bearing. Girder bottom flanges are equal in elevation across the skew; resulting in level substructure caps.

Gannett Fleming	GF JOB # 54682	CALC NO.: **HAM075_PROS_01**	REV. NO.: **01**
		PROJECT: **HAM-75-7.85**	

ODOT PID NO.: 77889		DESIGNED: *CTM*	DATE: *10/24/13*
BRIDGE NO: *HAM-75-Prosser*		CHECKED: *VDK*	DATE: *10/29/13*
SUBJECT: *Elevations*		UPDATED: *VDK*	DATE: *05/22/14*
PGL Data		VERIFIED: *CTM*	DATE: *05/28/14*
		FINAL VERIFY: *JEA*	DATE: *07/11/14*

PGL Vertical Curve & Profile Data (C/L of MLP)

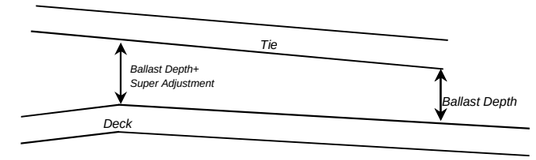
Note: PGL = Top of low rail. Back of Backwall excludes any backwall haunch.

WP = Working Point = Intersection of PGL Alignment

Station	Element (at PGL, U.N.O.)	PGL Elevation	Main Track Data			Shoulder Elevations (MAX)	
			Super Elevation	High Rail Elev.	Low Rail Elev.	"D" To Shldr	Shldr. Elev.
156+91.83	Begin 150' Approach	557.45	1.000 in	557.54	557.45	27.000 in	555.20
157+06.83	0.1	557.24	1.000 in	557.32	557.24	27.000 in	554.99
157+21.83	0.2	557.00	1.000 in	557.08	557.00	27.000 in	554.75
157+36.83	0.3	556.74	1.000 in	556.83	556.74	27.000 in	554.49
157+51.83	0.4	556.47	1.000 in	556.55	556.47	27.000 in	554.22
157+66.83	0.5	556.17	1.000 in	556.25	556.17	27.000 in	553.92
157+81.83	0.6	555.85	1.000 in	555.93	555.85	27.000 in	553.60
157+96.83	0.7	555.54	1.000 in	555.62	555.54	27.000 in	553.29
158+11.83	0.8	555.22	1.000 in	555.30	555.22	27.000 in	552.97
158+26.83	0.9	554.91	1.000 in	554.99	554.91	27.000 in	552.66
158+41.83	Rear Abut @ Rear Face	554.59	1.000 in	554.67	554.59	27.000 in	552.34
158+45.94	Rear Bkwall, Front Face (WP)	554.51	1.000 in	554.59	554.51		
158+44.45	CL Brg @ Left Girder	554.54	1.000 in	554.62	554.54		
158+48.11	Rear Abut C/L Deck @ C/L Brg	554.46	1.000 in	554.54	554.46		
158+51.83	CL Brg @ Right Girder	554.38	1.000 in	554.46	554.38		
158+54.51	0.1 * Span @ CL Deck	554.33	1.000 in	554.41	554.33		
158+60.91	0.2 * Span @ CL Deck	554.19	1.000 in	554.27	554.19		
158+67.30	0.3 * Span @ CL Deck	554.06	1.000 in	554.14	554.06		
158+73.70	0.4 * Span @ CL Deck	553.92	1.000 in	554.01	553.92		
158+80.10	0.5 * Span @ CL Deck	553.79	1.000 in	553.87	553.79		
158+86.50	0.6 * Span @ CL Deck	553.65	1.000 in	553.74	553.65		
158+92.90	0.7 * Span @ CL Deck	553.52	1.000 in	553.60	553.52		
158+99.29	0.8 * Span @ CL Deck	553.38	1.000 in	553.47	553.38		
159+05.69	0.9 * Span @ CL Deck	553.25	1.000 in	553.33	553.25		
159+09.17	CL Brg @ Left Girder	553.18	1.000 in	553.26	553.18		
159+12.09	FWD Abut C/L Deck @ C/L Brg	553.12	1.000 in	553.20	553.12		
159+15.06	CL Brg @ Right Girder	553.05	1.000 in	553.14	553.05		
159+14.20	FWD Bkwall, Front Face (WP)	553.07	1.000 in	553.16	553.07		
159+17.66	FWD Abut @ Rear Face	553.00	1.000 in	553.08	553.00	27.000 in	550.75
159+32.66	0.1	552.68	1.000 in	552.77	552.68	27.000 in	550.43
159+47.66	0.2	552.37	1.000 in	552.45	552.37	27.000 in	550.12
159+62.66	0.3	552.05	1.000 in	552.14	552.05	27.000 in	549.80
159+77.66	0.4	551.74	1.000 in	551.82	551.74	27.000 in	549.49
159+92.66	0.5	551.42	1.000 in	551.51	551.42	27.000 in	549.17
160+07.66	0.6	551.11	1.000 in	551.19	551.11	27.000 in	548.86
160+22.66	0.7	550.79	1.000 in	550.88	550.79	27.000 in	548.54
160+37.66	0.8	550.48	1.000 in	550.56	550.48	27.000 in	548.23
160+52.66	0.9	550.16	1.000 in	550.25	550.16	27.000 in	547.91
160+67.66	End 150' Depart	549.85	1.000 in	549.93	549.85	27.000 in	547.60

Gannett Fleming	GF JOB # 54682	CALC NO.: HAM075_PROS_01	REV. NO.: 01
ODOT PID NO.: 77889		PROJECT: HAM-75-7.85	
BRIDGE NO: *HAM-75-Prosser*		DESIGNED: *CTM*	DATE: *10/24/13*
SUBJECT: *Elevations*		CHECKED: *VDK*	DATE: *10/29/13*
Bottom of Girder and Bearing		UPDATED: *VDT*	DATE: *01/20/16*
		VERIFIED: *CTM*	DATE: *04/25/16*

NOTE 1: 8" Minimum ballast thickness is required below the low end of the tie. The Super Adjustment dimension is the extra thickness required over the crown of the deck to account for the tie cross-slope and deck cross-slope, and includes the horizontal eccentricity of the alignment to the C/L deck. The actual ballast depth at the deck crown is the summation of the two values.



Substructure Elevation Data

Calculation assumes that all girders are at the same elevation along the skew.

Design Span Length (Span Chord Length)	64.000 ft	Deck Cross-Slope	0.010	Super-Elevation	1.000 in.

Live Load Deflection Allowance

L/800 LL deflection will be assumed in a straight line approximation to account for cars parked atop the structure. It is assumed that girders will be cambered for DL deflection. *d=0.960 in*

Element/ Location	Rear Abutment (EXP)			Forward Abutment (FIX)			Vertical Clearance Locations					
	CL Brg @ Left Girder	Rear Abut C/L Deck @ C/L Brg	CL Brg @ Right Girder	CL Brg @ Left Girder	FWD Abut C/L Deck @ C/L Brg	CL Brg @ Right Girder	Left Girder @ Rear Prosser Curb	Right Girder @ Rear Prosser Curb	Left Girder @ Prosser Crown	Right Girder @ Prosser Crown	Left Girder @ Forward Prosser Curb	Right Girder @ Forward Prosser Curb
Dist. Along C/L Chord (along skew)	0.0000 ft	0.0000 ft	0.0000 ft	0.0000 ft	0.0000 ft	0.0000 ft	11.9408 ft	11.9408 ft	29.5883 ft	29.5883 ft	47.2359 ft	47.2359 ft
Station	158+44.45	158+48.11	158+51.83	159+09.17	159+12.09	159+15.06	158+56.25	158+63.87	158+74.12	158+81.29	158+91.99	158+98.71
PGL Elevation	554.54	554.46	554.38	553.18	553.12	553.05	554.29	554.13	553.91	553.76	553.54	553.40
C/L to Chord (+ = Right)	0.1483 ft	0.0000 ft	-0.1388 ft	-0.1346 ft	0.0000 ft	0.1467 ft	-0.3252 ft	-0.5050 ft	-0.6717 ft	-0.6930 ft	-0.5971 ft	-0.4601 ft
141 RE Rail Height	7.438 in	7.438 in	7.438 in	7.438 in	7.438 in	7.438 in	7.438 in	7.438 in	7.438 in	7.438 in	7.438 in	7.438 in
Assumed Tie Plate Thickness	1.000 in	1.000 in	1.000 in	1.000 in	1.000 in	1.000 in	1.000 in	1.000 in	1.000 in	1.000 in	1.000 in	1.000 in
Tie Thickness	7.000 in	7.000 in	7.000 in	7.000 in	7.000 in	7.000 in	7.000 in	7.000 in	7.000 in	7.000 in	7.000 in	7.000 in
Super Adjustment (See Note 1)	0.352 in	0.340 in	0.329 in	0.329 in	0.340 in	0.352 in	0.314 in	0.300 in	0.286 in	0.285 in	0.292 in	0.303 in
Ballast	10.262 in	9.000 in	8.403 in	10.076 in	9.000 in	8.592 in	10.297 in	8.377 in	10.239 in	8.433 in	10.182 in	8.489 in
Waterproofing Thickness	1.125 in	1.125 in	1.125 in	1.125 in	1.125 in	1.125 in	1.125 in	1.125 in	1.125 in	1.125 in	1.125 in	1.125 in
Deck Crown Height	1.000 in	1.000 in	1.000 in	1.000 in	1.000 in	1.000 in	1.000 in	1.000 in	1.000 in	1.000 in	1.000 in	1.000 in
Min Concrete Thickness	9.500 in	9.500 in	9.500 in	9.500 in	9.500 in	9.500 in	9.500 in	9.500 in	9.500 in	9.500 in	9.500 in	9.500 in
Top Flange Thickness	2.750 in	2.750 in	2.750 in	2.750 in	2.750 in	2.750 in	2.750 in	2.750 in	2.750 in	2.750 in	2.750 in	2.750 in
Web Deptth	42.500 in	42.500 in	42.500 in	42.500 in	42.500 in	42.500 in	42.500 in	42.500 in	42.500 in	42.500 in	42.500 in	42.500 in
Bottom Flange Thickness	2.750 in	2.750 in	2.750 in	2.750 in	2.750 in	2.750 in	2.750 in	2.750 in	2.750 in	2.750 in	2.750 in	2.750 in
PGL to Bottom of Steel (Feet)	7.140 ft	7.034 ft	6.983 ft	7.122 ft	7.034 ft	7.000 ft	7.139 ft	6.978 ft	7.132 ft	6.982 ft	7.128 ft	6.988 ft
Min. Bottom of Steel Elevation	547.426	547.426	547.426	546.083	546.083	546.083	547.175	547.175	546.805	546.805	546.435	546.435
Distance Along C/L Girder	0.000 ft	0.000 ft	0.000 ft	64.000 ft	64.000 ft	64.000 ft	11.941 ft	11.941 ft	29.588 ft	29.588 ft	47.236 ft	47.236 ft
Max Allowable LL Deflection	0.000 in	0.000 in	0.000 in	0.000 in	0.000 in	0.000 in	0.358 in	0.358 in	0.888 in	0.888 in	0.503 in	0.503 in
Min. Bottom of Steel & Deflection	547.426	547.426	547.426	546.083	546.083	546.083	547.146	547.146	546.731	546.731	546.393	546.393
Sole Plate Thickness	2.500 in	2.500 in	2.500 in	2.000 in	2.000 in	2.000 in						
Rocker Thickness	0.750 in	0.750 in	0.750 in	0.000 in	0.000 in	0.000 in						
Masonry Plate Thickness	3.250 in	3.250 in	3.250 in	3.000 in	3.000 in	3.000 in						
Total Bearing Thickness	0.54 ft	0.54 ft	0.54 ft	0.42 ft	0.42 ft	0.42 ft						
Max. Top of Cap Elevation	546.88	546.88	546.88	545.67	545.67	545.67						

Left Girder Slope:	-2.0993%	Deck Crown Long. Slope:	-2.0993%	Right Girder Slope:	-2.0993%
		PGL Chord Slope:	-2.0993%	Beam Differential:	- 10/32

Deck extends past bearing CL 2.139 (along CL Girder)

Rear Deck Low Point = 552.26	FWD Abut. Deck Low Point = 550.83
Rear Deck Crown Point: = 552.35	FWD Abut. Deck Crown Point: = 550.91

3.750 feet = Curb Line to Low Point

Rear Top/Curb (24" Standard) = 554.31	FWD Top/Curb (24" Standard) = 552.88
Rear Top/End Dam = 552.44	FWD Top/End Dam = 551.00

	Rear Abutment	Forward Abutment
Top of Cap Elevation	546.88	545.67
Total Differential (Fascia Girders)	0.000 in	0.000 in
No. of Girder Spaces	5	5
Individual Bearing Thickness Step	0.000 in	0.000 in

Gannett Fleming	GF JOB # 54682	CALC NO.: **HAM075_PROS_01**	EV. NO.: **01**
ODOT PID NO.: 77889		DESIGNED: *CTM*	DATE: *10/24/13*
BRIDGE NO: *HAM-75-Prosser*		CHECKED: *VDK*	DATE: *10/29/13*
SUBJECT: *Elevations*		UPDATED: *VDK*	DATE: *05/22/14*
Vertical Clearance Under		VERIFIED: *CTM*	DATE: *05/28/14*
Structure		FINAL VERIFY: *JEA*	DATE: *07/11/14*

PROJECT: **HAM-75-7.85**

Mainline Location	Mainline Station	Mainline Profile Grade Elev.	Mainline Bottom of Beam Elevation (Inc. LL Δ)		Crossroad Station from EX CL Basemap	Crossroad Profile Grade Elev.	Crossroad Offset from Centerline (ft)	Crossroad Cross-slope (Proposed)	Crossroad Pavement Elevation	Vertical Clearance (ft)
Left Girder @ Rear Prosser Curb	158 + 56.250	554.289	547.146		7 + 08.307	525.918	16.500	-0.0224	525.548	21.60
Right Girder @ Rear Prosser Curb	158 + 63.870	554.129	547.146		7 + 28.361	526.011	16.500	-0.0224	525.641	21.50
Left Girder @ Prosser Crown	158 + 74.120	553.913	546.731		7 + 14.567	525.947	0.000	0.0000	525.947	20.78
Right Girder @ Prosser Crown	158 + 81.290	553.763	546.731		7 + 34.621	526.040	0.000	0.0000	526.040	**20.69**
Left Girder @ Forward Prosser Curb	158 + 91.990	553.538	546.393		7 + 20.827	525.976	16.500	-0.0224	525.606	20.79
Right Girder @ Forward Prosser Curb	158 + 98.710	553.397	546.393		7 + 40.881	526.069	16.500	-0.0224	525.699	20.69

Survey Data to Set Prosser Average Grade

Survey Point	Elevation	Long. Dist.	Long Grade	Sta
SV8356: SW Shoulder	525.58			7 + 00.475
SV8383: NW Shoulder	525.78	52.25	-0.38%	7 + 52.720
SV8357: South Crown	525.88			6 + 99.694
SV8379: North Crown	526.16	52.98	-0.54%	7 + 52.670
SV8358: SE Shoulder	525.54			6 + 99.180
SV8380: NE Shoulder	525.79	53.10	-0.47%	7 + 52.280

NOTE: Prosser will be assumed to be a straight grade, utilizing nearby survery points to set the straight line grade.

Average Long. Grade:	**-0.46%**
South Cross Slope 1	2.05%
North Cross Slope 2	2.43%
Average Cross Slope:	**2.24%**

No VC Data At this time

G_1 =	0.0046
G_2 =	0.0046
VC =	0.00 ft.
PVC Sta. =	6 + 99.694
PVT Sta. =	7 + 52.670
PVC Elev.	525.878
PVT Elev.	526.162

Gannett Fleming	GF JOB # 54682	CALC NO.: **HAM075_PROS_01**	REV. NO.: **00**
		PROJECT: **HAM-75-7.85**	

ODOT PID NO.:	77889 77889	DESIGNED: *EFD*	DATE: *06/03/14*
BRIDGE NO:	*HAM-75-Prosser*	CHECKED: *CTM*	DATE: *07/09/14*
SUBJECT:	*Elevations*	UPDATED: *CTM*	DATE: *11/17/14*
	SCREED AND FINAL DECK ELEVATIONS	VERIFIED: -	DATE: -

Deck Crown Height: 1.00 in.
Span Length: 64.00 ft

Rear Low Point Elevation: 552.218
Forward Low Point Elevation: 550.874

Girder Slope: -2.099%

FINAL DECK ELEVATION TABLE

LINE NO.	LOCATION	Rear Abut. 0.000 L (0.00 ft)	0.125 L (8.00 ft)	0.250 L (16.00 ft)	0.375 L (24.00 ft)	Mid Span 0.500 L (32.00 ft)	0.625 L (40.00 ft)	0.750 L (48.00 ft)	0.875 L (56.00 ft)	Fwd. Abut. 1.000 L (64.00 ft)	Offset from Low Point
LINE 1	LEFT CURB LINE	552.30	552.13	551.97	551.80	551.63	551.46	551.29	551.13	550.96	3.75 ft
LINE 2	LEFT LOW POINT	552.22	552.05	551.88	551.71	551.55	551.38	551.21	551.04	550.87	0.00 ft
LINE 3	DECK CROWN	552.30	552.13	551.97	551.80	551.63	551.46	551.29	551.13	550.96	7.00 ft
LINE 4	RIGHT LOW POINT	552.22	552.05	551.88	551.71	551.55	551.38	551.21	551.04	550.87	0.00 ft
LINE 5	RIGHT CURB LINE	552.30	552.13	551.97	551.80	551.63	551.46	551.29	551.13	550.96	3.75 ft

SCREED ELEVATION TABLE

LINE NO.	LOCATION	Rear Abut. 0.000 L (0.00 ft)	0.125 L (8.00 ft)	0.250 L (16.00 ft)	0.375 L (24.00 ft)	Mid Span 0.500 L (32.00 ft)	0.625 L (40.00 ft)	0.750 L (48.00 ft)	0.875 L (56.00 ft)	Fwd. Abut. 1.000 L (64.00 ft)
LINE 1	LEFT CURB LINE	552.30	552.14	551.98	551.82	551.65	551.48	551.31	551.13	550.96
LINE 2	LEFT LOW POINT	552.22	552.06	551.90	551.74	551.57	551.40	551.23	551.05	550.87
LINE 3	DECK CROWN	552.30	552.14	551.98	551.82	551.65	551.48	551.31	551.13	550.96
LINE 4	RIGHT LOW POINT	552.22	552.06	551.90	551.74	551.57	551.40	551.23	551.05	550.87
LINE 5	RIGHT CURB LINE	552.30	552.14	551.98	551.82	551.65	551.48	551.31	551.13	550.96

DL1 + DL2 Deflections (Excludes Steel Self Weight)	0.00	0.11	0.20	0.26	0.29	0.26	0.20	0.11	0.00

DESIGN CALCULATION COVER SHEET

Gannett Fleming
Engineers and Architects, P.C.
Excellence Delivered ***As Promised***

ISO 9001:2008 CERTIFIED

PROJECT:	**ODOT/EMH&T: HAM-75-7.85**	*ODOT PID NO:*	*77889*
BRIDGE NO:	**HAM-75-PROS**	*EMH&T JOB NO:*	
SUBJECT:	***Girder***	**GF JOB NO:**	**54682**
	Built-Up Section Properties	CALC NO.:	**HAM075_PROS_02**
		REV NO.:	**03**
FILE NAME(s):	*HAM075_PROS_02.xlsx*	**SHEET: 1 OF 7**	

SUPERSEDES CALC NO: 01 (Stage 2)

CALCULATION DESCRIPTION:

This will calculate the section properties of the plate girder and check stiffener requirements.

DESIGN BASIS OR REFERENCES:

NS Underpass Design Criteria
AREMA (2008) Chapter 15

MISCELLANEOUS NOTES:

The Stage 1 layout will be that of the PAVR Alternative #2.

PAVR Alternative #1 did not meet NS Requirement of 2 supports per rail.

The Stage 2 Layout and Design is updated for final design (conservative). The composite properties are added.

Stage 3: Increased top flange width by 1.5". Bottom flange width reduced by 2" (below top flange width) to account for stiffener connection plate bolts. This gives a conservative result for composite and gross properties, but will ensure the bottom flange yield stress on net section is conservative.

RECORD OF ISSUES:

REV	DESCRIPTION	BY	DATE	VERIF'D	DATE
00	Stage 1 Calculation	EFD	01/13/12		
01	Final, Stage 2 Calculation	EFD	04/09/14		
02	Stage 3: Updated Flange Thickness	EFD	05/27/15	VDT	06/02/15
03	Stage 3: Increased Flange Thickness	VDT	01/20/16	CTM	04/25/16

RECORD OF CHECK OF CURRENT CALCULATION VERSION

BY	DATE	CHKD	DATE	UPD.	DATE	VERIF'D	DATE
EFD	05/27/15	VDT	06/02/15	VDT	01/20/16	CTM	04/25/16

Gannett Fleming	GF JOB # 54682	CALC NO.: **HAM075_PROS_02**	REV. NO.: **03**
		PROJECT: **ODOT/EMH&T: HAM-75-7.85**	
ODOT PID NO.: 77889		DESIGNED: *EFD*	DATE: *01/13/12*
BRIDGE NO: *HAM-75-PROS*		UPDATED: *EFD*	DATE: *04/09/14*
SUBJECT: *Girder*		CHECKED: *CTM*	DATE: *05/16/14*
Built-Up Section Properties		UPDATED: *VDT*	DATE: *01/20/16*
Includes composite properties		VERIFIED: *CTM*	DATE: *04/25/16*

Indicates user input

Yield Strength of Steel (Fy): 50.0 ksi
TOTAL Girder Depth: 48.000 in

Assumed Fillet Weld Size: 0.3750 in
Assumed Unit Weight: 490.0 pcf

SECTION PROPERTIES ABOUT PRIMARY AXIS - Non Composite (NET)

Note: the "y" C.G. dimension is input from the bottom of the bottom fiber (all values positive)

Description	Element No.	Width (in.)	Height (in.)	Area (A) (in^2)	y (C.G.) (in.)	Ay (in^3)	Ay^2 (in^4)	I_{xo} (in^4)	Ay^2+I_{xo} (in^4)
Top Flange	1	15.000	2.750	41.250	46.6250	1923.28	89673.0	26.0	89699
Web Plate	2	0.875	42.500	37.188	24.0000	892.50	21420.0	5597.5	27017
Bottom Flange	3	13.000	2.750	35.750	1.3750	49.16	67.6	22.5	90
Total		15.000	48.000	114.188		2864.94			116806.6

Cross Sectional Area:	A =	114.188	in^2
Basic Section Weight:	Steel =	388.6	plf
Weld Weight:	Weld =	1.503	plf (includes four fillet welds)
Design Dead Load:	W =	391	plf (Rounded up to nearest pound)

Distance from Bottom Edge to N.A.:	d_y =	25.090	in.
Moment of Inertia:	I_x =	44926.0	in^4
Radius of Gyration:	R_x =	19.84	in.
Section Modulus:	Bot S_x =	1790.61	in^3

Top S_x = 1960.96 in^3

Gannett Fleming	GF JOB # 54682	CALC NO.: **HAM075_PROS_02**	REV. NO.: **03**
		PROJECT: **ODOT/EMH&T: HAM-75-7.85**	
ODOT PID NO.: 77889		DESIGNED: *EFD*	DATE: *01/13/12*
BRIDGE NO: *HAM-75-PROS*		UPDATED: *EFD*	DATE: *04/09/14*
SUBJECT: *Girder*		CHECKED: *CTM*	DATE: *05/16/14*
Built-Up Section Properties		UPDATED: *VDT*	DATE: *01/20/16*
Includes composite properties		VERIFIED: *CTM*	DATE: *04/25/16*

Indicates user input

SECTION PROPERTIES ABOUT PRIMARY AXIS - COMPOSITE (n=8)

Note: The deck thickness is taken as 1" less, to allow for construction variation (0.5") as well as a top sacrificial layer (0.5")

Description	Element No.	Width (in.)	Height (in.)	Area (A) (in^2)	y (C.G.) (in.)	Ay (in^3)	Ay^2 (in^4)	I_{xo} (in^4)	Ay^2+I_{xo} (in^4)
Top Flange	1	15.000	2.750	41.250	46.6250	1923.28	89673.0	26.0	89699
Web Plate	2	0.875	42.500	37.188	24.0000	892.50	21420.0	5597.5	27017
Bottom Flange	3	15.000	2.750	41.250	1.3750	56.72	78.0	26.0	104
Deck *(3'-6" spa, n=8)*	4	5.250	8.500	44.625	52.2500	2331.66	121829.0	268.7	122098
Total		15.000	56.500	164.313		5204.16			238918.2

Distance from Bottom Edge to N.A.:	d_y =	31.672	in.		
Moment of Inertia:	I_x =	74090.5	in^4		
Radius of Gyration:	R_x =	21.23	in.		
Section Modulus:	Bot S_x =	2339.28	in^3	Top S_x =	2984.19 in^3

Gannett Fleming	GF JOB # 54682	CALC NO.: **HAM075_PROS_02**	REV. NO.: **03**
		PROJECT: **ODOT/EMH&T: HAM-75-7.85**	
ODOT PID NO.: 77889		DESIGNED: *EFD*	DATE: *01/13/12*
BRIDGE NO: *HAM-75-PROS*		UPDATED: *EFD*	DATE: *04/09/14*
SUBJECT: *Girder*		CHECKED: *CTM*	DATE: *05/16/14*
Built-Up Section Properties		UPDATED: *VDT*	DATE: *01/20/16*
Includes composite properties		VERIFIED: *CTM*	DATE: *04/25/16*

Indicates user input

SECTION PROPERTIES ABOUT SECONDARY AXIS

Note: the "x" C.G. dimension is input about the centerline of web. Negative = Left, Positive = Right

Description	Element No.	Width (in.)	Height (in.)	Area (A) (in^2)	x (C.G.) (in.)	Ax (in^3)	Ax^2 (in^4)	I_{yo} (in^4)	Ax^2+I_{yo} (in^4)
Top Flange	1	15.000	2.750	41.250	0.0000	309.38	2320.3	773.4	3094
Web Plate	2	0.875	42.500	37.188	0.0000	278.91	2091.8	2.4	2094
Bottom Flange	3	15.000	2.750	41.250	0.0000	309.38	2320.3	773.4	3094
Total		15.000	48.000	119.688		897.66			8281.7

Total Surface Perimeter:	P.tot =	154.250	in
Reduced Perimeter:	P.red	133.8	in (does not include top or sides of top flange)
Total Surface Area:	SA.tot	1.071	ft^2/ft
Reduced Surface Area:	SA.red	0.929	ft^2/ft

Distance from Left Edge to N.A.:	d_x =	7.500	in.		
Moment of Inertia:	I_y =	1549.2	in^4		
Radius of Gyration:	R_y =	3.60	in.		
Section Modulus:	Left S_y =	206.57	in^3	Right S_y =	206.57 in^3

Gannett Fleming	GF JOB # 54682	CALC NO.: HAM075_PROS_02	REV. NO.: 03
		PROJECT: ODOT/EMH&T: HAM-75-7.85	
ODOT PID NO.: 77889		DESIGNED: EFD	DATE: 01/13/12
BRIDGE NO: HAM-75-PROS		UPDATED: EFD	DATE: 04/09/14
SUBJECT: Girder		CHECKED: CTM	DATE: 05/16/14
Built-Up Section Properties		UPDATED: VDT	DATE: 01/20/16
Includes composite properties		VERIFIED: CTM	DATE: 04/25/16

Indicates user input

AREMA 15-1.7 Cross-Sectional Checks

AREMA 15-1.7.1 Compression Flange Bracing

Distance between bracing points:	D.b=	0.000	in	*Continously laterally braced*
Allowable distance between bracing points:	D.b-a =	133.662	in	*AREMA 15-1.7.1.b*
Check compliance:	**BRACING OK**			*Continously laterally braced*

AREMA 15-1.6.2 Outstanding Compression Flange

Thickness of Flange:	T.f=	2.750	in	
Outstanding Width:	W.comp =	7.063	in	
Allowable Outstanding Width:	W.cmp.a =	28.478	in	*AREMA 15-1.6.2.a.(1)*
Check compliance:	**FLANGE OK**			*^^Ties are not directly upon the flange ^^*

AREMA 15-1.7.3 Web Plate Thickness (Without Longitudinal Stiffeners)

Clear distance between flanges:	D.fl=	42.500	in	
Thickness ratio of allowable clear distance between flanges:	D.fl.r=	0.007		*AREMA 15-1.7.3.a* *(Assumes fully stressed top flange)*
Minimum allowable web ratio thickness:	t.w-a1=	0.318	in	
1/6 of flange thickness check:	t.w-a1=	0.458	in	*AREMA 15-1.7.3.c*
Actual web thickness:	t.w=	0.875	in	*NS Requires 1/2" Minimum*
Check compliance:	**WEB OK**			

AREMA 15-1.7.7 Minimum Bearing and Jacking Stiffener Thickness - Preliminary Size

Outstanding Leg:	ℓ.s=	7.063	in	
Minimum Thickness:	ℓ.s=	0.587	in	*AREMA 15-1.6.2.a(1)* *(Bracing and secondary members)* *^^ Equation re-arranged to solve for "t" ^^*
Minimum Bearing Stiffener Thickness:	**0.625 in**			

Gannett Fleming	GF JOB # 54682	CALC NO.: **HAM075_PROS_02**	REV. NO.: **03**
		PROJECT: **ODOT/EMH&T: HAM-75-7.85**	
ODOT PID NO.: 77889		DESIGNED: *EFD*	DATE: *01/13/12*
BRIDGE NO: *HAM-75-PROS*		UPDATED: *EFD*	DATE: *04/09/14*
SUBJECT: *Girder*		CHECKED: *CTM*	DATE: *05/16/14*
Built-Up Section Properties		UPDATED: *VDT*	DATE: *01/20/16*
Includes composite properties		VERIFIED: *CTM*	DATE: *04/25/16*

Indicates user input

AREMA 15-1.7.8.a-c Intermediate Transverse Web Stiffeners - Preliminary Size

Preliminary stiffener thickness:	T.st	0.500	in	*Enter Data (3/8" minimum per AREMA)*
Web depth limit of required stiffeners:	D.fl-ns=	44.674	in	*AREMA 15-1.7.8.a (equation in paragraph)*
Maximum Spacing Equation:	d.st.eq	42.870	in	*AREMA 15-1.7.8.a ("d" equation, assumes S=0.20*Fy)*
				0.3 Fy is based upon Final analysis (conservative)*
Requirement Check:	**Unstiffened**			
Maximum Spacing from paragraph:	d.st.text	42.500	in	*AREMA 15-1.7.8.a (text in paragraph)*
Minimum Spacing Considered:	d.prelim	42.000	in	*Rounded down to the nearest 6"*
Minimum Moment of Inertia:	I.st.min	22.787	in^3	*AREMA 15-1.7.8.a ("I" equation)*
Preliminary stiffener width:	W.st	5.000	in	*Enter Data*
Minimum allowable stiffener width:	W.min	3.600	in	*AREMA 15-1.7.8.b*
Maximum allowable stiffener width:	W.max	8.000	in	*AREMA 15-1.7.8.b*
Single Plate Moment of Inertia:	I.st-plt	5.208	in^4	
Distance, Plate centroid to web centroid:	R.st-plt	2.938	in	
Area of single stiffener plate:	A.st-plt	2.500	in^2	
Moment of Inertia of Stiffeners:	I.st	53.561	in^4	*Moment of I about CL web, using Parallel Axis Theorem*
Moment of Inertia Check:	**Stiff. Size OK**			

Intermediate Stiffener Details: **NO INTERMEDIATE STIFFENERS REQUIRED**

AREMA 15-1.7.8.f Longitudinal Web Stiffeners - Check that they are not required

NOTE: It is desireable to not include longitudinal web stiffeners. Re-proportion girder if longitudinal stiffers are required

Web depth limit of required stiffeners:	D.ls-max	118.773	in	*AREMA 15-1.7.8.f (assumes f = 0.55 *Fy)*
Actual web depth between flanges:	D.fl=	42.500	in	
Requirement Check:	**Long. Unstiffened**			

SECTION PROPERTIES ABOUT PRIMARY AXIS - Non Composite (GROSS)

Note: the "y" C.G. dimension is input from the bottom of the bottom fiber (all values positive)

Description	Element No.	Width (in.)	Height (in.)	Area (A) (in^2)	y (C.G.) (in.)	Ay (in^3)	Ay^2 (in^4)	I_{xo} (in^4)	Ay^2+I_{xo} (in^4)
Top Flange	1	15.000	2.750	41.250	46.6250	1923.28	89673.0	26.0	89699
Web Plate	2	0.875	42.500	37.188	24.0000	892.50	21420.0	5597.5	27017
Bottom Flange (2 holes)	3	15.000	2.750	41.250	1.3750	56.72	78.0	26.0	104
Total		15.000	48.000	119.688		2872.50			116820.5

Cross Sectional Area:	A =	119.688	in^2
Basic Section Weight:	Steel =	407.3	plf
Weld Weight:	Weld =	1.503	plf (includes four fillet welds)
Design Dead Load:	W =	409	plf (Rounded up to nearest pound)

Distance from Bottom Edge to N.A.:	d_y =	24.000	in.
Moment of Inertia:	I_x =	47880.5	in^4
Radius of Gyration:	R_x =	20.00	in.
Section Modulus:	Bot S_x =	1995.02	in^3

Top S_x = 1995.02 in^3

DESIGN CALCULATION COVER SHEET

Gannett Fleming
Engineers and Architects, P.C.
Excellence Delivered ***As Promised***

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PROJECT:	**ODOT/EMH&T: HAM-75-7.85**	*ODOT PID NO:*	*77889*
BRIDGE NO:	**HAM-75-PROS**	*EMH&T JOB NO:*	
SUBJECT:	***Loads***	**GF JOB NO:**	**54682**
	Girders	CALC NO.:	**HAM-75-PROS_03**
	Stresses and Deflection Checks	REV NO.:	**03**
FILE NAME(s):	*HAM075_PROS_03.xlsx*	**SHEET: 1 OF 20**	

SUPERSEDES CALC NO: 02 (Stage 3)

CALCULATION DESCRIPTION:

This will calculate all loads applied to the superstructure.
Vertical beam stresses and deflections will be calculated and checked.

DESIGN BASIS OR REFERENCES:

NS Underpass Design Criteria
AREMA (2013) Chapter 15
Calculation: HAM075_PROS_02 (Beam Properties)
Bridge Layout per 075_Prosser_BS003.dgn

MISCELLANEOUS NOTES:

EFD Updates made per CTM Comments; VDK re-check to ensure that this master template follows the current AREMA code and is fully functional before it is used repeatedly on all 3 HAM-75 bridges (4 unique spans)

EFD Updates made at stage 2. Design span length was increased to 64' to allow the final design to proceed while the stage 1 alignment tweaks are being made. The composite deck allows the increased spann to work without changing girder size (from stage 1)

Stage 3 update to reflect net section properties (non-composite) and an increased flange thickness

Stage 3: Increased LL distribution results in 0.5" thicker flanges; deck increased per NSRR comments

RECORD OF ISSUES:

REV	DESCRIPTION	BY	DATE	VERIF'D	DATE
00	Stage 1 Calculation	CTM	10/09/13		
01	Final, Stage 2 Calculations	EFD	04/09/14		
02	Stage 3, Modified for net section	EFD	05/27/15	VDT	06/02/15
03	Stage 3 Update	VDT	01/20/16	CTM	04/25/16

RECORD OF CHECK OF CURRENT CALCULATION VERSION

BY	DATE	CHKD	DATE	UPD.	DATE	VERIF'D	DATE
EFD	05/27/15	VDT	06/02/15	VDT	01/20/16	CTM	04/25/16

Gannett Fleming	GF JOB # 54682	CALC NO.: HAM-75-PROS_03	REV. NO.: 03
		PROJECT: ODOT/EMH&T: HAM-75-7.85	
ODOT PID NO.: 77889		DESIGNED: CTM	DATE: 10/09/13
BRIDGE NO: HAM-75-PROS		UPDATED: EFD	DATE: 04/09/14
SUBJECT: Loads		CHECKED CTM	DATE: 05/16/14
Girders		UPDATED:	DATE:
Load Combinations		VERIFIED:	DATE:

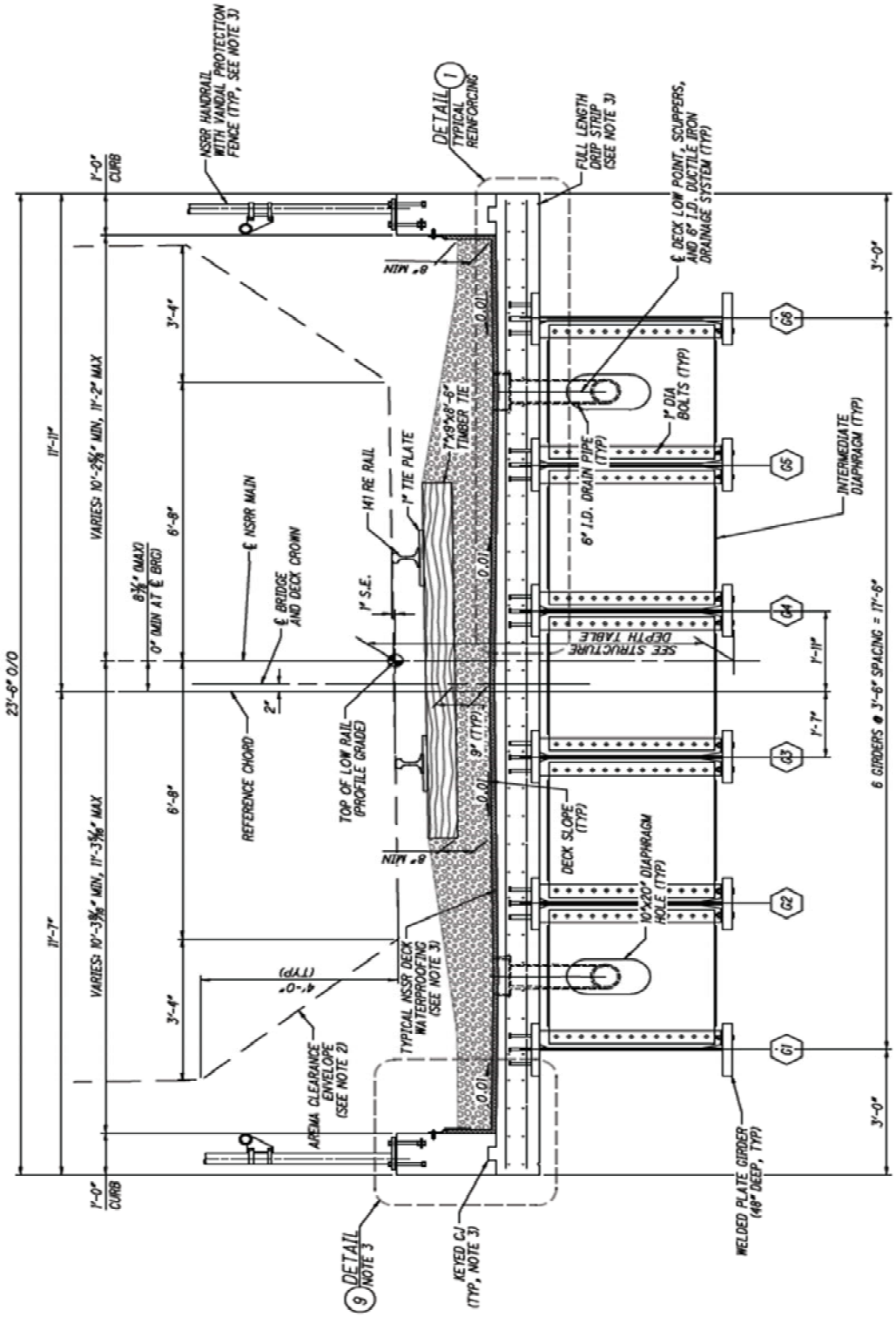
Load Combinations Considered (NSRR Factors included)

Comb.	DL	LL	I	CF	LF	N	W	WU	SL	Qg	Qb	EQ	CWR	DF	O.Stress Allowed
A1	1	1	1	1											100%
B1	1	1	1		1	1	1						1		125%
B1A	1	1	1		1	1	1						1		100%
C		1	1												f_{fat}
D1	1			1		1	1		1						150%
D2	**1**									**1**					**170%**
E1	1											1			150%
F	1							1							100%
G											1			1	150%
H1	1														150%
H2	*1*														*125%*

Comb.	Description of Use
A1	All members
B1	All members, except floorbeam hangers and high strength bolts
B1A	Floorbeam hangers and high strength bolts
C	All members with Fatigue Details
D1	Members resisting overall instability
D2	**Members resisting overall instability (NSRR)**
E1	All members - Earthquake
F	Members loaded by wind only (Bracing)
G	Cross frames, diaphragms, anchor rods
H1	Members stressed during lifting or jacking
H2	*Members stressed during erection*

DL	Dead loads (self weight,superimposed dead loads, erection loads)
LL	Live loads
I	Impact (dynamic amplification)
CF	Centrifugal force
LF	Longitudinal forces from equipment (braking and locomotive traction)
N	Lateral forces from equipment (nosing)
W	Wind forces loaded bridge
WU	Wind forces on unloaded bridge
SL	Live load on leeward track of 1200 lb/ft without impact
Qg	Derailment load for girder design
Qb	Derailment load for lateral bracing design
EQ	Forces from earthquake (combined transverse and longitudinal)
CWR	Forces from CWR (lateral and longitudinal)
DF	Lateral forces from out-of-plane bending and from load distribution effects (Differential Deflection due to derailment)
f_{fat}	Allowable stress based on member loaded length and fatigue detail category

Typical Section



General Layout Data - Span 1 of 1

Span length is increased to 64' for final design to allow for lower-disturbance construction sequencing

Span Length Data

C/C Bearings (Max):	64.000 ft	
Front Face/Face of Abut. Backwall:	4.000 ft	
Min. Clearance from Backwall:	3.000 in	0.250 ft
Maximum Beam Length:	66.500 ft	
Beam Extension past C/L Brg.	1.250 ft	

Track Alignment

Degree of Curvature:	7.75 deg	
Radius of Curvature:	739.86 ft	
Superelevation:	0.500 in	0.042 ft
Maximum Design Speed:	15.0 mph	

NOTE: Design Speed increased and super decreased to be conservative

Superstructure Cross Section Data

Out/Out Deck	23.500 ft
Standard Track Gage:	4.708 ft
C/C Rail Heads	4.932 ft
Number of Beams:	6
Standard Beam Spacing:	3.500 ft
Standard Overhang:	3.000 ft
Max Eccentricity,	
CL Deck/CL Alignment:	6.000 in
Deck Cross-Slope:	1.000%
Cross-Slope Width	7.000 ft
Tie Length:	8.500 ft
Curb Wall Thickness:	1.000 ft
Crossframe Spacing	12.000 ft
Stiffener Spacing	12.000 ft
Bridge Deck Length:	66.500 ft

Will be used to calculate total longitudinal forces

Vertical Tie-In to Top of Top Flange

Rail Height:	6.625 in
Tie Plate:	1.000 in
Tie Thickness:	7.000 in
Average Ballast:	9.250 in
Waterproofing:	1.125 in
Min Deck Thickness:	9.500 in
Max Additional Cross-Slope:	0.840 in
Rail Head to Top of Beam:	d_{top} = 35.340 in
Height of Curb:	H_{curb} = 25.840 in
Bottom of Tie to Top of Beam:	d_b = 20.205 in
Beam Lateral Distribution Length:	DB_{Lat}= 11.87 ft
AREMA: 15-1.3.4.2.4.a	
Deck Lateral Distribution Length:	DDk_{Lat}= 10.18 ft
Deck Long. Distribution Length:	DDK_{Long}= 4.68 ft
AREMA: 15-1.3.4.2.2	

Number of beam spaces in lateral distribution length: 3.391

Assumes beam layout centered on alignment
Assumes maximum eccentricity in one direction

Number of beams to distribute live load to: 4

Standard live load and track/rail dead load will be equally divided amongst this many beams

Material & Girder Data

Reference Calculation: HAM075_0834C for beam properties

Basic Girder Data

XS Area:	119.688	in^2
Dead Load:	409	*plf*
Total Depth:	48.000	*in*
Web Depth:	42.500	*in*
Web Thickness:	0.875	*in*
Flange Width:	15.000	*in*
Flange Thickness:	2.750	*in*
Diaphragm Weight:	250.6	*lb*
Stiffener Weight:	30.1	*lb*

Material Densities and Unit Loads

Track (Rails and fastenings):	200.0	*plf*
Steel:	490.0	*pcf*
Reinforced and Prestressed concrete:	150.0	*pcf*
Plain (unreinforced) concrete:	145.0	*pcf*
Timber:	60.0	*pcf*
Sand and Gravel, Compacted (ballast):	120.0	*pcf*
Sand and Gravel, Loose:	100.0	*pcf*
SIP Formwork (including flute concrete):	15.0	*psf*
Deck Waterproofing:	10.0	*psf*
Handrail Weight:	15.0	*plf*

Extra Steel Weight: 5%

Bolts, Gusset Plates, (LLB NOT REQUIRED PER AREMA)

Diaphragm Flanges	45.9	*lb*
Diaphragm Web	204.7	*lb*
Int. Stiff. Width	5.0	*in*
Int. Stiff. Thick	0.5	*in*

Steel Properties (ASTM A588, Grade 50)

Yeild Stress:	50.0	*ksi*
Ultimate Stress:	70.0	*ksi*
Modulus of Elasticity:	29000.0	*ksi*

Girder Properties

Non-Composite is based on net section, composite is based on gross section

Primary Axis (non-composite)				n=8 Composite
Distance from Bottom Edge to N.A.:	d_y =	25.090	in.	31.7
Moment of Inertia:	I_x =	44926.0	in^4	74090.5
Radius of Gyration:	R_x =	19.84	in.	21.2
Section Modulus:	Bot S_x =	1790.61	in^3	2339.3
	Top S_x =	1960.96	in^3	2984.2

Secondary Axis			
Distance from Bottom Edge to N.A.:	d_x =	7.500	in.
Moment of Inertia:	I_y =	1549.2	in^4
Radius of Gyration:	R_y =	3.60	in.
Section Modulus:	Left S_y =	206.57	in^3
	Right S_y =	206.57	in^3

Basic Live Load Data

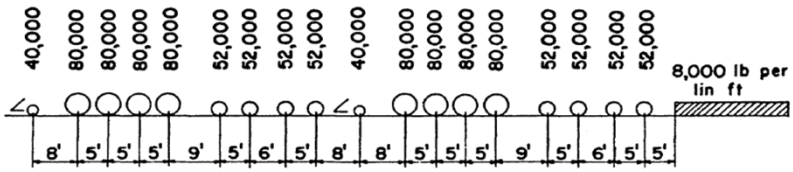


Figure 15-1-2. Cooper E 80 Load

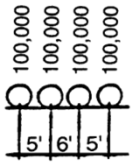


Figure 15-1-3. Alternate Live Load on 4 Axles

Unloaded Leeward Train:

SL= 1.200 klf

AREMA 1.3.10.a

Interpolated Data from AREMA Table 15-1-15 (Single Rail Loads)

Span Length (ft)	Maximum Moment (ft*kips)	Maximum Moment Quarter Point (ft*kips)	Maximum Shears (Kips)			Maximum Pier Reaction (Kips)	
			At End	At Quarter Point	At Center		
60	2597.80	2010.00	196.00	123.33	73.33	306.42	
64.00	2816.7	2249.3	206.0	126.8	74.9	325.5	40.0%
70	3145.00	2608.20	221.04	131.89	77.14	354.08	
64.00	614.40	460.80	38.40	19.20	0.00	38.40	**Unloaded Train (SL=1200 plf)**

Live Load Due to Load Eccentricity

AREMA 15-1.3.4.2.4.a states axle loads may be equally distributed to all girders within the lateral width prescribed above.

AREMA 15-1.3.4.2.1.d states that the effects of eccentricity and centrifugal load shall be included for ballasted decks.

Eccentricity due to Horizontal Curvature of Alignment

Eccentricity of load will apply a greater percentage of total load to a pair of girders if 4 are in the zone of influence.

Eccentricity load increase will be applied to a single girder if only 2-3 beams are in the zone of influence.

Standard number of beams to distribute live load to: 4

Percentage increase due to eccentricity: 7.1%

Un-Eccentric % of **rail** load to each beam: 50.0%

Eccentric % of rail load to girders on outside of curve: 60.0%

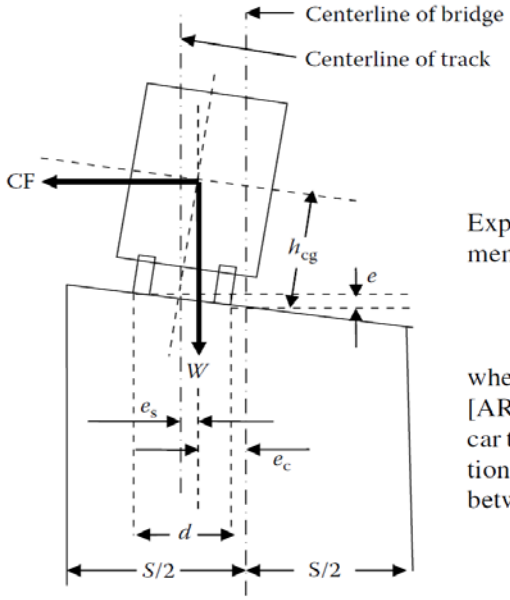
Note: 1/2 of the increase is taken even though as little as 1/3 could be assumed due to parabolically varying eccentricity. This is conservative.

Eccentricity is calculated by {ECCENTRICITY OF LOAD } / {BEAM SPACING}

A maximum value of 50% is allowed. An eccentricity larger than 50% will begin to distribute load to the next equally space girder.

The eccentric % of rail load to girders on outside of curve was based on the max possible load any girder could see. Using the 11.87 feet of lateral distribution length calculated above, girder spacing (3.5') divided by the lateral distribution length gives us 3.5 / 11.87 = 29.4% of the total load, or 58.9% of any rail load. To be conservative, 60% of a rail load will be applied.

Eccentricity due to Superelevation



$$e_s = \frac{h_{cg} e}{d}$$

Expressed as a percentage, this will affect the magnitude of forces to supporting members each side of the track in the following proportion:

$$\frac{2e_s}{s}(100), \qquad (3.29)$$

where h_{cg} is the distance from the center of gravity of the rail car to the base of the track [AREMA (2008) recommends 8 ft for the distance from the center of gravity of the car to the top of the rail]; e is the superelevation of the track; d is the horizontal projection of track gage distance (generally taken as 4.9 ft); s is the width of the span (distance between the center of gravity of longitudinal members supporting the curved track).

Known Variables & Calculations

Deg. of curve, D = 7.75 *deg*

Train Center of gravity, h_{cg} = 8.00 *ft*

Superelevation, SE = 0.500 *in*

Track c/c, d = 4.932 *ft*

C/C of resisting force couple: 7.000 *ft*

Note: Resisting force couple distance is the centroid of the supporting girder pairs for a 4-girder support system. 3 or 2 girder systems will have rocking applied to the outer most supporting girder.

Superelevation Eccentricity, e_s = 0.811 *in*

Superelevation Load +/-, SE = 1.93%

Increases inside of curve, decreases outside of curve

SE Applied Inside of Curve (per Beam) = 0.97%

SE Applied Outside of Curve (per Beam) = -0.97%

Outside of Curve Ecentricity

Vertical Cent. Factor, CF_v = 59.52%

Inside of Curve Eccentricity

Vertical Cent. Factor, CF_v = 40.48%

Applied Loads (Beams Outside of Curve)

	Span Length (ft)	Maximum Moment (ft*kips)	Maximum Moment Quarter Point	Maximum Shears (Kips)			Maximum Pier Reaction (Kips)
				At End	At Quarter Point	At Center	
Standard Live Load	64.00	1676.41	1338.71	122.62	75.44	44.55	193.72
Unloaded Train (SL)	64.00	365.67	274.26	22.85	11.43	0.00	22.85

Applied Loads (Beams Inside of Curve)

	Span Length (ft)	Maximum Moment (ft*kips)	Maximum Moment Quarter Point	Maximum Shears (Kips)			Maximum Pier Reaction (Kips)
				At End	At Quarter Point	At Center	
Standard Live Load	64.00	1140.27	910.57	83.40	51.31	30.30	131.76
Unloaded Train (SL)	64.00	248.73	186.54	15.55	7.77	0.00	15.55

Dead Load Geometry

Note: All dead loads are to be uniformly distributed

Concrete Sectional Area

Deck, Min Thickness:	18.60	sq. ft.
Deck, Cross-Slope:	0.49	sq. ft.
Curb:	4.31	sq. ft.
Between Top Flanges:	3.38	sq. ft.
Total Concrete:	**26.78**	**sq. ft.**

Deck Waterproofing

Total Length Across:	**23.65**	**ft**

Ballast Area

Thick, Max (top of tie):	1.35	ft
Thick, Min (at curb):	0.75	ft
NS Criteria, Future Ballast:	0.50	ft
Width, Ballast:	21.50	ft
Area, Max Ballast:	11.51	sq. ft.
Area, Side Ballast:	13.68	sq. ft.
Area, Future Ballast:	**10.75**	**sq. ft.**
Total Placed Ballast:	**25.19**	**sq. ft.**

Total Steel Weight (Interior Beam)

This will also be used for exterior beam DL

Beam:	409	plf
Diaphragm	20.89	plf
Stiffeners:	2.51	plf
Miscellaneous:	21.62	plf
Total Interior Beam:	**455**	**plf**

DL - Dead Load

Moments, Forces, and Stresses (All Beams)

Load	W (klf)	V_{DL} (kip)	M_{DL} (kip-in)	σ_{DL} (ksi)	Δ_{DL} (inches)	Notes
Steel	0.46	15.13	2795.52	1.56	0.13183	Erection Load
NON_COMPOSITE Concrete	0.56	18.68	3452.06	1.93	0.16279	Deck Pouring
COMPOSITE Concrete	0.10	3.46	639.45	0.36	0.01829	Composite
Waterproofing	0.04	1.31	242.21	0.14	0.00693	
Placed Ballast	0.50	16.75	3095.04	1.73	0.08850	
Future Ballast	0.22	7.15	1320.96	0.74		**NO CAMBER**
Handrail	0.01	0.17	30.72	0.02	0.00088	
Rail/Ties/Clips	0.05	1.66	307.20	0.17	0.00878	Divided by live load # of beams
Total Unfactored Dead Load	**1.93**	**64.31**	**11883.17**	**6.64**	**0.56039**	Track Supporting Beams
Total Unfactored Dead Load	**1.88**	**62.65**	**11575.97**	**6.46**	**0.54590**	Exterior Beams Only

Calculated Natural Frequency

Unloaded Natural Frequency				Loaded Natural Frequency			
Mass per linial foot:	m=	29.801721	slug/in	Loading (Alt. Load):	P	400000	pounds
First Mode:	ω_1	39.02497	rad/s	First Mode:	ωL_1	27.01666	rad/s
	ω_1	6.2110169	hertz		ωL_1	4.299835	hertz
	t_1	0.1610	seconds		tL_1	0.2326	seconds

TABLE 4.2
Undamped Natural Frequencies of Various Beams

Beam	ω_{ni} (rad/s)
	$\frac{i^2\pi^2}{L^2}\sqrt{\frac{EI}{m}}$
	$\frac{(4i^2+4i+1)\pi^2}{4L^2}\sqrt{\frac{EI}{m}}$
	$\frac{(4i^2-4i+1)\pi^2}{4L^2}\sqrt{\frac{EI}{m}}$
	$\frac{(16i^2+8i+1)\pi^2}{16L^2}\sqrt{\frac{EI}{m}}$

for the span of Example 4.3, $\omega_1 = 66.58$ rad/s $= 10.6$ Hz, is slightly greater than the estimate of 9.4 Hz for railway ballasted girder spans in Figure 4.8.

Due to the inertial effects of the relatively large railway live load on steel spans, the loaded simply supported beam natural frequencies are required in the dynamic analysis of steel railway superstructures. Approximate equations for the loaded simply supported beam fundamental frequency, ω_{L1}, have been proposed (Fryba, 1972) as

$$\omega_{L1} = \omega_1\sqrt{\frac{1}{1+(2P/mgL)+(m_w/2mg)}} \quad \text{for moving uniform continuous loads} \tag{4.16}$$

and

$$\omega_{L1} = \omega_1\sqrt{\frac{1}{1+(2P/mgL)}} \quad \text{for moving constant concentrated loads.} \tag{4.17}$$

A similar equation for the loaded simply supported beam fundamental frequency, ω_{L1}, was proposed for a moving harmonically varying concentrated force

TABLE 4.3
Unloaded Fundamental Frequencies of Steel Railway Bridges
(Empirical Equations from Fryba, 1996)

Superstructure	Unloaded Fundamental Frequency, f_1 (Hz)
Steel truss spans	$1135(L)^{-1.1}$
Ballasted girder spans	$135(L)^{-0.7}$
Open deck girder spans	$680/L$

Note: L = length in feet.

LL - Max Live Load

Moments, Forces, and Stresses

Beams Outside of Curve

Load	Equiv. W (klf)	V_{LL} (kip)	M_{LL} (kip-in)	σ_{LL} (ksi)	Δ_{LL} (inches)	Notes
Standard AREMA Live Load	3.27	122.62	20116.93	11.23	0.57524	Inc. For Deflection
Unloaded Train (SL)	0.71	22.85	4388.09	2.45	0.12548	Composite

Beams Inside of Curve

Load	Equiv. W (klf)	V_{LL} (kip)	M_{LL} (kip-in)	σ_{LL} (ksi)	Δ_{LL} (inches)	Notes
Standard AREMA Live Load	2.23	83.40	13683.23	7.64	0.39127	Inc. For Deflection
Unloaded Train (SL)	0.49	15.55	2984.71	1.67	0.08535	Composite

I - Vertical Live Load Impact

Moments, Forces, and Stresses

Impact Factor

AREMA 15-1.3.5.c (1)

Impact Factor Equation Used: (a)

Impact Factor Applied: 32.32%

Rocking Effect

AREMA 15-1.3.5.d

Rocking Effect Loading	20.0%	
C/C Tracks of applied force couple:	4.932	ft
Number of beams resisting force couple:	4	
C/C of resisting force couple:	7.000	ft
Additional Rocking Impact Applied:	14.1%	

Note: Resisting force couple distance is the centroid of the supporting girder pairs for a 4-girder support system. 3 or 2 girder systems will have rocking applied to the outer most supporting girder.

Total Impact Factor

Total Calculated Impact Factor:	46.4%	
Impact Factor Considered:	90.0%	*AREMA 15-1.3.5(b) allows reduction for ballasted deck.*
Total Applied Impact Factor:	41.8%	

Beams Outside of Curve

Load	Equiv. W (klf)	V_I (kip)	M_I (kip-in)	σ_I (ksi)	Δ_I (inches)	Notes
Standard AREMA Live Load	1.37	51.22	8403.05	4.69	0.24029	Inc. for Deflection
Unloaded Train (SL)	0.30	9.55	1832.95	1.02	0.05241	Composte

Beams Inside of Curve

Load	Equiv. W (klf)	V_I (kip)	M_I (kip-in)	σ_I (ksi)	Δ_I (inches)	Notes
Standard AREMA Live Load	0.93	34.84	5715.63	3.19	0.16344	Inc. for Deflection
Unloaded Train (SL)	0.20	6.49	1246.74	0.70	0.03565	Composite

Δ_v - Vertical Deflection Check

(Live Load + Impact)

AREMA 15-1.2.5.b

AREMA Allowable

L/640 1.200 *in*

MAX Live Load and Impact Deflections

Max Live Load Deflection:	0.575 *in*
Corresponding Impact Deflection:	0.240 *in*
Total Live Load + Impact Deflection:	0.816 *in*

	Actual		Allowable	
Deflection Check:	0.816	≤	1.200	**OK**
Deflection Ratio:	L / 942	≤	L / 640	

CF - Centrifugal Force

Moments, Forces, and Stresses

AREMA 15-1.3.6

Known Variables & Calculations

Deg. of curve, D =	7.75 *deg*	Speed, S =	15.0 *mph*
Train Center of gravity, h_{cg} =	8.00 *ft*	Track c/c, d =	4.932 *ft*
Cent. Factor, CF =	2.04%		
Horizontal Cent. Factor, CF_h =	2.04%	Vertical Cent. Factor, CF_v =	3.31%
Applied to each rail head		*Increases outside of curve, decreases inside of curve*	

Outside of Curve (per beam)		Inside of Curve (per beam)	
Vertical Cent. Factor, CF_v =	**1.65%**	**Vertical Cent. Factor, CF_v =**	**-1.65%**

Beams Outside of Curve

Load	Equiv. W (klf)	V_{CF} (kip)	M_{CF} (kip-in)	σ_{CF} (ksi)	Δ_{CF} (inches)	Notes
Standard AREMA Live Load	0.054	2.03	332.85	0.19	0.00952	Composite
Unloaded Train (SL)	0.012	0.38	72.60	0.04	0.00208	

Horizontal Cent. Load = 2.50 kip / Girder

Beams Inside of Curve

Load	Equiv. W (klf)	V_{CF} (kip)	M_{CF} (kip-in)	σ_{CF} (ksi)	Δ_{CF} (inches)	Notes
Standard AREMA Live Load	-0.037	-1.38	-226.40	-0.13	-0.00647	Will reduce stresses
Unloaded Train (SL)	-0.008	-0.26	-49.38	-0.03	-0.00141	Will reduce stresses

Horizontal Cent. Load = 1.70 kip / Girder

LF - Longitudinal Force

Moments, Forces, and Stresses

AREMA 15-1.3.12

The relative stiffness of substructures and related fixity will play a large role in longitudinal force distribution throughout a long structure.

LF herein will be for the worst case for this span.

The force for this individual span will be used to design individual items on this span.

Longitudinal force for the entire bridge length should be calculated and investigated seperately for substructure design and analysis.

Average Carriage Length = 40.0 ft

$$LF_B = 45 + 1.2L$$

Braking - Fully Loaded Bridge

Loaded Length:	66.500 *ft*
Full LF_{B_All}	124.80 *kips*
This Spans Portion of Full $LF_{B_All_Span}$	120.11 *kips*

Front axle to rear axle

$$LF_T = 25\sqrt{L}$$

Traction - Fully Loaded Bridge

Loaded Length:	66.500 *ft*
Full LF_{T_All}	203.87 *kips*
This Spans Portion of Full $LF_{T_All_Span}$	196.20 *kips*

Braking - This Span Only		
LF_{B_Span}	121.80	*kips*
Center of Action	8.00	*ft*
Vertical LF_{BV_Span}	24.36	*kips*

Traction - This Span Only		
LF_{T_Span}	200.00	*kips*
Center of Action	3.00	*ft*
Vertical LF_{TV_Span}	15.00	*kips*

Controling Load - Total			
Maximum Longitudinal Force:	LF_{Max}	200.00	*kips*
Max Shear from LF:	LF_V	24.36	*kips*
Equally distributed shear:	w_{LF}	0.38	*klf*
Moment created from LF_v:	LF_M	2338.56	*kip-in*

Applied Load - Per Rail			
Max Shear:	LF_V	12.18	*kips*
Moment created from LF_v:	LF_M	1169.28	*kip-in*

Beams Outside of Curve

Load	Equiv. W (klf)	V_{LF} (kip)	M_{LF} (kip-in)	σ_{LF} (ksi)	Δ_{LF} (inches)	Notes
Standard AREMA Live Load	0.11	7.25	695.92	0.39	0.01990	Composite
Unloaded Train (SL)	0.02	1.35	151.80	0.08	0.00434	

Beams Inside of Curve

Load	Equiv. W (klf)	V_{LF} (kip)	M_{LF} (kip-in)	σ_{LF} (ksi)	Δ_{LF} (inches)	Notes
Standard AREMA Live Load	0.08	4.93	473.36	0.26	0.01354	Composite
Unloaded Train (SL)	0.02	0.92	103.25	0.06	0.00295	

N - Lateral Forces (Nosing)

Moments, Forces, and Stresses

AREMA 15-1.3.9

Not used for girder design. *Will be used for bracing system design.*

a. A single moving concentrated lateral force equal to one-quarter of the weight of the heaviest axle of the specified live load, without impact, shall be applied at the base of rail in either direction and at any point along the span in addition to the other lateral forces specified (Reference 20). On spans supporting multiple tracks, the force shall be applied on one track only.

b. The only resulting stresses to be considered are axial stresses in members bracing the flanges of stringer, beam and girder spans, axial stresses in the chords of truss spans and in members of cross frames of such spans, and stresses from lateral bending of flanges of longitudinal flexural members having no bracing system. The effects of lateral bending between braced points of flanges, axial forces in flanges, vertical forces and forces transmitted to bearings shall be disregarded.

Heaviest Axle: Alt. Load = 100 kip
Notional Lateral Design Load, N = 25.0 kip

W - Wind on Loaded Bridge

Moments, Forces, and Stresses

AREMA 15-1.3.7

In general, the wind force shall be considered as a moving load acting in any horizontal direction. As a minimum, the bridge shall be designed for laterally and longitudinally applied wind forces acting independently as follows:

a. On the train, the lateral wind force shall be taken at 300 lb. per linear foot applied normal to the train on one track at a distance of 8 feet above top of rail.

b. On the bridge, lateral wind pressure shall be taken at 30 lb. per square foot normal to the following surfaces:

(1) For girder spans, 1.5 times the vertical projection of the span.

c. The lateral wind force on girder and truss spans, however, shall not be taken as less than 200 lb. per foot for the loaded chord or flange and 150 lb. per foot for the unloaded chord or flange, neglecting the wind force on the floor system.

d. The longitudinal wind force on spans shall be taken as:

(1) For girder spans, 25 percent of the lateral wind force.

Loaded Length: 66.500 *ft*

Wind on Superstructure

Girder Height:	48.000	*in*
Deck Thickness:	9.50	*in*
Curb Height:	25.84	*in*
Total Height:	6.95	*ft*
1.5 x Total Height:	10.42	*ft*
Wind Pressure:	30.00	*psf*
Wind Load:	312.53	*plf*

Not less than 200 plf, will be used for lateral bracing

Total Wind Force:	20.78	*kip*
Force Centroid:	5.21	*ft*
Resisting Couple	17.50	*ft*
Resisting Force:	6.19	*kip*
Resisting Load/Girder:	46.51	*plf* (up/down)

50% is assumed to be resisted by a single beam

OUTER MOST (windward = up, leeward = down)

Girder Moment from Vertical Component 23.81 *kip*ft*

(Worst Beam)

Wind on Live Load

Lateral Load Applied:	300.000	*plf*
Load Centroid:	8.00	*ft*
Vertical Component:	486.59	*plf*

*Assumed to act down on rail at the **outside of curve***

*Assumed to act up on rail at the **inside of curve***

Total Wind Force: 19.95 *kip*

Girder Moment from Vertical Component 249.13 *kip*ft*

(Total)

NOTE: Lateral Loads will also be used for lateral bracing system design and stability analysis of substructures.

Total Girder Moment and shears are multiplied by eccentricity factors above.

Beams Outside of Curve

Load	Equiv. W (klf)	V_W (kip)	M_W (kip-in)	σ_W (ksi)	Δ_W (inches)	Notes
Standard AREMA Live Load	0.34	11.18	2065.09	1.15	0.05905	
Unloaded Train (SL)	0.34	11.18	2065.09	1.15	0.05905	

Beams Inside of Curve

Load	Equiv. W (klf)	V_W (kip)	M_W (kip-in)	σ_W (ksi)	Δ_W (inches)	Notes
Standard AREMA Live Load	-0.24	-8.10	-1496.03	-0.84	-0.04278	Will reduce stress
Unloaded Train (SL)	-0.24	-8.10	-1496.03	-0.84	-0.04278	Will reduce stress

WU - Wind on Unloaded Bridge

Moments, Forces, and Stresses

AREMA 15-1.3.8

a. The lateral wind force on the unloaded bridge shall be taken as 50 lb per square foot of surface as defined in Article 1.3.7b.

b. The longitudinal wind force on the unloaded spans shall be taken as:

(1) For girder spans, 25 percent of the lateral wind force.

Loaded Length: 66.500 *ft*

Lateral on Superstructure

1.5 x Total Height:	10.42	*ft*
Wind Pressure:	50.00	*psf*
Wind Load:	520.88	*plf*

Not less than 200 plf, will be used for lateral bracing

Total Wind Force: 34.64 *kip*

Vertical on Superstructure

Force Centroid:	5.21	*ft*
Resisting Couple	17.50	*ft*
Resisting Force:	10.31	*kip*
Resisting Load/Girder:	77.52	*plf*

OUTER MOST (windward = up, leeward = down)

Girder Moment from Vertical Component 39.69 *kip*ft*

(Worst Beam)

NOTE: Lateral Loads will also be used for lateral bracing system design and stability analysis of substructures.

Total Girder Moment and shears are multiplied by eccentricity factors above

Beams Outside of Curve

Load	Equiv. W (klf)	V_{WU} (kip)	M_{WU} (kip-in)	σ_{WU} (ksi)	Δ_{WU} (inches)	Notes
Standard AREMA Live Load	0.08	2.58	476.27	0.27	0.01362	
Unloaded Train (SL)	0.08	2.58	476.27	0.27	0.01362	

Beams Inside of Curve

Load	Equiv. W (klf)	V_{WU} (kip)	M_{WU} (kip-in)	σ_{WU} (ksi)	Δ_{WU} (inches)	Notes
Standard AREMA Live Load	-0.08	-2.58	-476.27	-0.27	-0.01362	Will reduce stress
Unloaded Train (SL)	-0.08	-2.58	-476.27	-0.27	-0.01362	Will reduce stress

Qg - Derailment Load on Girders

Moments, Forces, and Stresses

NFS_UP2 V.C.2 (Beams - Used for D2 Loadcase)

C. **Design of Superstructure for Derailment:**

2. The supporting steel members, whose centerlines measure not more than 11'-0" from the outside face of curb shall be considered as carrying the derailed load. An impact load, equal to 0.8x live load, shall be provided. Allowable unit stresses in steel shall be not more than 70 percent greater than the normal allowable unit stresses.

Total Impact Factor

Total Applied Impact Factor: 80.0%

Note: AREMA Rail load will be applied to an individual girder (derail directly atop girder) and multiplied by the impact factor listed above)

Girder Forces below are the total LL+I for the derailed condition - Used for Load Case D2

Derailed Beam

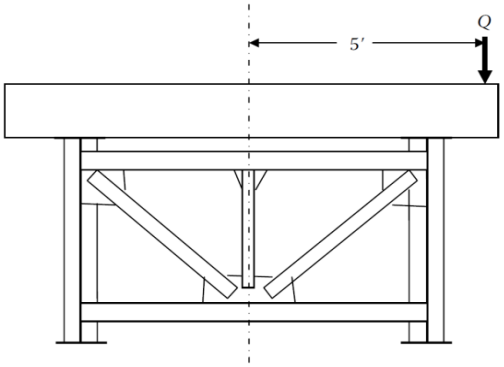
Load	Equiv. W (klf)	V_Q (kip)	M_Q (kip-in)	σ_Q (ksi)	Δ_Q (inches)	Notes
Standard AREMA Live Load	9.90	220.71	60840.29	33.98	1.73973	Stress Check
Unloaded Train (SL)	2.16	41.14	13271.04	7.41	0.37949	Composite

Qb - Derailment Load on lateral bracing

Moments, Forces, and Stresses

AREMA 15-1.3.10 .b (Cross Frames Only - Used for G Loadcase) - Max Differential Deflections Considered in Connection Calculations

.... "A single line of wheel loads (Q) equal to the design load per rail (Article 1.3.3) including full design impact is to be applied at an eccentricity of 5 feet from the centerline of track as shown in Figure 15-1-4, but no further than the edge of the deck or, for open decks, the bridge ties."



Geometry Considered

The entire load will be applied to a single girder and the vertical force will need to be transmitted to the adjacent girder through the cross frame or diaphragm within the allowable stress (150%).

Total Impact Factor		Cooper E-80 Impact Load for Design	
Total Applied Impact Factor:	41.8%	Vertical Point Load:	56.7 kip

Will be applied to cross bracing in lieu of differential deflection analysis.
Overstress allowance applies.

CWR - Continously Welded Rail

Not Required on Ballasted Deck

128 Design of Modern Steel Railway Bridges

4.4.3 Forces From the CWR on Steel Railway Bridges

The magnitude of the CWR–bridge thermal interaction is governed by the following conditions:

- The type of bridge (open deck, ballasted‡) and bridge materials.

‡ For ordinary ballast deck bridges, the differential thermal movements are generally accommodated by the ballast section.

EQ - Earthquake Loads

Not explicity required for a single span bridge

AREMA Chapter 9. *Multi-Span bridges should utilize the ELF Procedure.*

Table 9-1-7. Analysis Procedure Selection

Bridge Configuration	Analysis Procedure[1]
Single-span	No analysis required
Two-span	ELF or MA Procedure
Multi-span regular[2]	ELF or MA Procedure
Multi-span irregular[2]	MA Procedure

Note:
1. ELF denotes Equivalent Lateral Force Procedure, MA denotes Modal Analysis Procedure.
2. Irregular bridges are those structures with significantly irregular configuration or support stiffness.

1.4.5.3 Equivalent Lateral Force Procedure[1]

The Equivalent Lateral Force Procedure may be used for two-span bridges or multi-span regular bridges as described in Paragraph 1.4.5.2. The procedure is described below.

1.4.4.3 Seismic Response Coefficient[1]

The Seismic Response Coefficient (C_m) to be used in the methods of analysis recommended in Paragraph 1.4.5, shall be calculated from the following formula. For areas with soft soil conditions and high seismicity, or close proximity to known faults, use of a site-specific response spectrum is preferred.

$$C_m = \frac{1.2ASD}{T_m^{2/3}} \leq 2.5AD$$

C_m = Seismic Response Coefficient for the mth mode

A = Base Acceleration Coefficient determined in accordance with Paragraph 1.3.3.3 — A = 6% 475 Year Return

S = Site Coefficient determined in accordance with Paragraph 1.4.4.1 — S = 1.2 Rock > 200'

D = Damping Adjustment Factor determined in accordance with Paragraph 1.4.4.2 — D = 1.000 **AREMA 9-1.4.4.2** *(Conservative; assumes 2.5% damping)*

T_m = Period of vibration of the mth mode in seconds

Lateral Period of Vibration, $T_m = 2\pi*(W/gK)^{1/2}$ — Per AASHTO LRFD 4.7.4.3.2c

W =	765060	lbs	
g =	32.17	ft/sec^2	
I =	17753733	in^4	*(Deck)*
E =	3604997	lbs/in^2	
$v_{s,MAX}$ =	0.000083	in	
K=	9672686		
T_m =	0.311550	sec	

Seismic Response Coefficient for the First Mode

Calculated C_m =	18.8%	
Maximum C_m =	15.0%	
Used C_m =	15.0%	Note: For reference, ODOT assumes 20% across the state.

Horizontal Seismic Load

Distributed Weight of Bridge:	11504.66	*plf*	
Distributed horizontal Load, p(x):	1725.70	*plf*	
Total horizontal Load:	114.76	*kip*	
Total Longitudinal Load at each end:	114.76	*kip*	(Use for anchor bolt and foundation analysis)
Total Lateral Load at each end:	57.38	*kip*	(Use for end diaphragms)

Gannett Fleming	GF JOB # 54682	CALC NO.: **HAM-75-PROS_03**	REV. NO.: **03**
		PROJECT: **ODOT/EMH&T: HAM-75-7.85**	

ODOT PID NO.: 77889		DESIGNED: *CTM*	DATE: *09/27/13*
BRIDGE NO: *HAM-75-PROS*		UPDATED: *EFD*	DATE: *04/09/14*
SUBJECT: *Loads*		CHECKED *CTM*	DATE: *05/16/14*
Girders		UPDATED: *VDT*	DATE: *01/20/16*
		VERIFIED: *CTM*	DATE: *04/25/16*

Δ_L - Lateral Deflection Check

AREMA 15-1.2.5.c

Allowable Chord Deflection =	0.250	*in*	*0.25" = Curved, 0.375 "= Tangent*
Chord Length =	31	*feet*	*31' = Curved, 62' = Tangent*
Allowable Span Deflection =	1.06556	*in*	*Squared Proportional to Span Length*

Lateral Centrifugal Force on Fully Loaded Bridge

Horizontal Cent. Factor, CF_h = 4.08%

Total (Applied to both rail heads)

Horizontal Cent. Moment = 1379.17 *kip-in*

(Ratio of vertical moment - load on the total track is considered)

W - Wind Forces On fully loaded bridge

Total Lateral Load: 0.05104 *kips/inch*

Total (Applied to both rail heads)

Total Lateral Moment: 3763.354 *kip-in*

Section Properties Used

"D" of the outer most beams	63.00	*in*
"D" of the inner most beams	21.00	*in*
Girder Sectional Area:	119.69	in^2
Girder Secondary Mom. Of Inertia:	1549.25	in^4
Moment of Inertia of outer pair:	953177.87	in^4
Moment of Inertia of remaining:	108662.87	in^4
Total Lateral Mom. Of I Considered:	530920.37	in^4

N - Notional Load

Lateral Point Load: 25.0 *kips*

Total Lateral Moment: 4800.0 *kip-in*

Note: The entire bridge will not be assumed for lateral deflection (to be conservative). Only the girders supporting one track will be used and 50% of the the section properties combined using the parallel axis theorem. This assumes a largely rigid lateral system, which is required by AREMA.

Factored Moments Applied

Load Combination A1 =	1379.17	*kip-in*	with	100%	*Allowable*
Load Combination B1 =	8563.35	*kip-in*	with	125%	*Allowable*
Load Combination B1A =	8563.35	*kip-in*	with	100%	*Allowable*
Load Combination D1 =	9942.53	*kip-in*	with	150%	*Allowable*

Calculated Conservative Lateral Deflections

Load Combination A1 =	0.00550	*in*	with	1.066 in	*Allowable*	**OK**
Load Combination B1 =	0.03417	*in*	with	1.332 in	*Allowable*	**OK**
Load Combination B1A =	0.03417	*in*	with	1.066 in	*Allowable*	**OK**
Load Combination D1 =	0.03968	*in*	with	1.598 in	*Allowable*	**OK**

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		PROJECT: ODOT/EMH&T: HAM-75-7.85	
ODOT PID NO.: 77889		DESIGNED: CTM	DATE: 10/09/13
BRIDGE NO: HAM-75-PROS		UPDATED: EFD	DATE: 04/09/14
SUBJECT: Loads		CHECKED CTM	DATE: 05/16/14
Girders		UPDATED: EFD	DATE: 05/27/15
		VERIFIED: VDT	DATE: 06/02/15

Unfactored Girder Bending Stresses

Comb.	DL	LL	I	CF	LF	N	W	WU	SL	Qg
Beams Outside of Curve										
Standard AREMA Live Load	6.64 ksi	11.23 ksi	4.69 ksi	0.19 ksi	0.39 ksi		1.15 ksi	0.27 ksi	0.00 ksi	33.98 ksi
Unloaded Train (SL)	6.64 ksi	0.00 ksi	1.02 ksi	0.04 ksi	0.08 ksi		1.15 ksi	0.27 ksi	2.45 ksi	7.41 ksi
Beams Inside of Curve										
Standard AREMA Live Load	6.64 ksi	7.64 ksi	3.19 ksi	-0.13 ksi	0.26 ksi		-0.84 ksi	-0.27 ksi	0.00 ksi	33.98 ksi
Unloaded Train (SL)	6.64 ksi	0.00 ksi	0.70 ksi	-0.03 ksi	0.06 ksi		-0.84 ksi	-0.27 ksi	1.67 ksi	7.41 ksi

Bending Stresses from Girder Load Cases - Outside of Curve

Comb.	DL	LL	I	CF	LF	N	W	WU	SL	Qg	Qb	EQ	CWR	DF	Total	Allowable	Check
A1	6.64 ksi	11.23 ksi	4.69 ksi	0.19 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	22.75 ksi	27.50 ksi	OK
B1	6.64 ksi	11.23 ksi	4.69 ksi	0.00 ksi	0.39 ksi	0.00 ksi	1.15 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	24.11 ksi	34.38 ksi	OK
B1A	6.64 ksi	11.23 ksi	4.69 ksi	0.00 ksi	0.39 ksi	0.00 ksi	1.15 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	24.11 ksi	27.50 ksi	OK
C - FATIGUE	0.00 ksi	11.23 ksi	4.69 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	15.93 ksi	16.00 ksi	OK
D1	6.64 ksi	0.00 ksi	0.00 ksi	0.19 ksi	0.00 ksi	0.00 ksi	1.15 ksi	0.00 ksi	2.45 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	10.43 ksi	41.25 ksi	OK
D2	6.64 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	33.98 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	40.61 ksi	46.75 ksi	OK
E1	6.64 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	6.64 ksi	41.25 ksi	OK
F	6.64 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.27 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	6.90 ksi	27.50 ksi	OK
G	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	41.25 ksi	OK
H1	6.64 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	6.64 ksi	41.25 ksi	OK
H2	6.64 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	6.64 ksi	34.38 ksi	OK

Bending Stresses from Girder Load Cases - Inside of Curve

Comb.	DL	LL	I	CF	LF	N	W	WU	SL	Qg	Qb	EQ	CWR	DF	Total	Allowable	Check
A1	6.64 ksi	7.64 ksi	3.19 ksi	-0.13 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	17.34 ksi	27.50 ksi	OK
B1	6.64 ksi	7.64 ksi	3.19 ksi	0.00 ksi	0.26 ksi	0.00 ksi	-0.84 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	16.90 ksi	34.38 ksi	OK
B1A	6.64 ksi	7.64 ksi	3.19 ksi	0.00 ksi	0.26 ksi	0.00 ksi	-0.84 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	16.90 ksi	27.50 ksi	OK
C - FATIGUE	0.00 ksi	7.64 ksi	3.19 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	10.83 ksi	16.00 ksi	OK
D1	6.64 ksi	0.00 ksi	0.00 ksi	-0.13 ksi	0.00 ksi	0.00 ksi	-0.84 ksi	0.00 ksi	1.67 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	7.34 ksi	41.25 ksi	OK
D2	6.64 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	33.98 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	40.61 ksi	46.75 ksi	OK
E1	6.64 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	6.64 ksi	41.25 ksi	OK
F	6.64 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	-0.27 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	6.37 ksi	27.50 ksi	OK
G	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	41.25 ksi	OK
H1	6.64 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	6.64 ksi	41.25 ksi	OK
H2	6.64 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	6.64 ksi	34.38 ksi	OK

Gannett Fleming	GF JOB # 54682	CALC NO.: HAM-75-PROS_03	REV. NO.: 03
		PROJECT: ODOT/EMH&T: HAM-75-7.85	
ODOT PID NO.: 77889		DESIGNED: CTM	DATE: 10/09/13
BRIDGE NO: HAM-75-PROS		UPDATED: EFD	DATE: 04/09/14
SUBJECT: Loads		CHECKED CTM	DATE: 05/16/14
Girders		UPDATED: EFD	DATE: 05/27/15
		VERIFIED: VDT	DATE: 06/02/15

Unfactored Girder Shear Loads (At Supports)

Comb.	DL	LL	I	CF	LF	N	W	WU	SL	Qg
Beams Outside of Curve										
Standard AREMA Live Load	64.31 kip	122.62 kip	51.22 kip	2.03 kip	7.25 kip		11.18 kip	2.58 kip	0.00 ksi	220.71 kip
Unloaded Train (SL)	64.31 kip	0.00 ksi	9.55 kip	0.38 kip	1.35 kip		11.18 kip	2.58 kip	22.85 kip	41.14 kip
Beams Inside of Curve										
Standard AREMA Live Load	64.31 kip	83.40 kip	34.84 kip	-1.38 kip	4.93 kip		-8.10 kip	-2.58 kip	0.00 ksi	220.71 kip
Unloaded Train (SL)	64.31 kip	0.00 ksi	6.49 kip	-0.26 kip	0.92 kip		-8.10 kip	-2.58 kip	15.55 kip	41.14 kip

Shear Area
42.00 in^2

Shear Stresses from Girder Load Cases - Outside of Curve

Comb.	DL	LL	I	CF	LF	N	W	WU	SL	Qg	Qb	EQ	CWR	DF	Total	Allowable	Check
A1	1.53 ksi	2.92 ksi	1.22 ksi	0.05 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	5.72 ksi	17.50 ksi	OK
B1	1.53 ksi	2.92 ksi	1.22 ksi	0.00 ksi	0.17 ksi	0.00 ksi	0.27 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	6.11 ksi	21.88 ksi	OK
B1A	1.53 ksi	2.92 ksi	1.22 ksi	0.00 ksi	0.17 ksi	0.00 ksi	0.27 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	6.11 ksi	17.50 ksi	OK
C - FATIGUE	0.00 ksi	2.92 ksi	1.22 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	4.14 ksi		
D1	1.53 ksi	0.00 ksi	0.00 ksi	0.05 ksi	0.00 ksi	0.00 ksi	0.27 ksi	0.00 ksi	0.54 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	2.39 ksi	26.25 ksi	OK
D2	1.53 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	5.25 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	6.79 ksi	29.75 ksi	OK
E1	1.53 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	1.53 ksi	26.25 ksi	OK
F	1.53 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.06 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	1.59 ksi	17.50 ksi	OK
G	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	26.25 ksi	OK
H1	1.53 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	1.53 ksi	26.25 ksi	OK
H2	1.53 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	1.53 ksi	21.88 ksi	OK

Shear Stresses from Girder Load Cases - Inside of Curve

Comb.	DL	LL	I	CF	LF	N	W	WU	SL	Qg	Qb	EQ	CWR	DF	Total	Allowable	Check
A1	1.53 ksi	1.99 ksi	0.83 ksi	-0.03 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	4.31 ksi	17.50 ksi	OK
B1	1.53 ksi	1.99 ksi	0.83 ksi	0.00 ksi	0.12 ksi	0.00 ksi	-0.19 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	4.27 ksi	21.88 ksi	OK
B1A	1.53 ksi	1.99 ksi	0.83 ksi	0.00 ksi	0.12 ksi	0.00 ksi	-0.19 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	4.27 ksi	17.50 ksi	OK
C - FATIGUE	0.00 ksi	1.99 ksi	0.83 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	2.82 ksi		
D1	1.53 ksi	0.00 ksi	0.00 ksi	-0.03 ksi	0.00 ksi	0.00 ksi	-0.19 ksi	0.00 ksi	0.37 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	1.68 ksi	26.25 ksi	OK
D2	1.53 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	5.25 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	6.79 ksi	29.75 ksi	OK
E1	1.53 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	1.53 ksi	26.25 ksi	OK
F	1.53 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	-0.06 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	1.47 ksi	17.50 ksi	OK
G	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	26.25 ksi	OK
H1	1.53 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	1.53 ksi	26.25 ksi	OK
H2	1.53 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	0.00 ksi	1.53 ksi	21.88 ksi	OK

DESIGN CALCULATION COVER SHEET

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PROJECT:	**ODOT/EMH&T: HAM-75-7.85**	*ODOT PID NO:*	*77889*
BRIDGE NO:	**HAM-PROSSER AVE**	*EMH&T JOB NO:*	
SUBJECT:	***Loads***	**GF JOB NO:**	**54682**
	Abutment Loads for	CALC NO.:	**HAM075_PROS_04**
	Abut5 Program	REV NO.:	**01**
FILE NAME(s):	*HAM075_PROS_04.xlsx*	**SHEET:**	**1 OF 6**

SUPERSEDES CALC NO: 00 (Stage 1 Design)

CALCULATION DESCRIPTION:

This will calculate all loads applied to the abutment on a per-foot basis for use in the Abut5 program.

Vertical Loads: The max girder reaction will be divided by the girder spacing to produce the worst loading scenario on an abutment.
Horizontal Loads: The horizontal girder reactions are divided by the a width equal to 66% of the substructure and 33% of the total out-out of the beams. This recognizes the fact that the deck diaphragm action will distribute the loads to all of teh girders, and that the entire stem will resist bending by the time the force is distributed to the foundation. The vertical/longitudinal components will be used as input into Abut5. A single fixed abutment design will be completed.

The forward abutment will be fixed, but the taller rear abutment dimensions will be analyzed to ensure the analysis is conservative and is applicable for both abutments.

DESIGN BASIS OR REFERENCES:

NS Underpass Design Criteria
AREMA (2013) Chapter 8, Section 5
AREMA (2014) Chapter 15, Section 5
Calculation: HAM075_PROSC_03 (Girder Loads and Reactions)
Calculation: HAM075_PROSC_01 (Elevation Data)
Revised Stage 1 Plans (after addressing comments)

MISCELLANEOUS NOTES:

Abut5 Calculates the abutment self-weight and earth loads automatically. This calculation will determine earth loads based upon rankine.

Bearing stiffness calculations will be completed assuming sliding steel plates.

Loads input into Abut5 will be grouped according to AASHTO.

RECORD OF ISSUES:

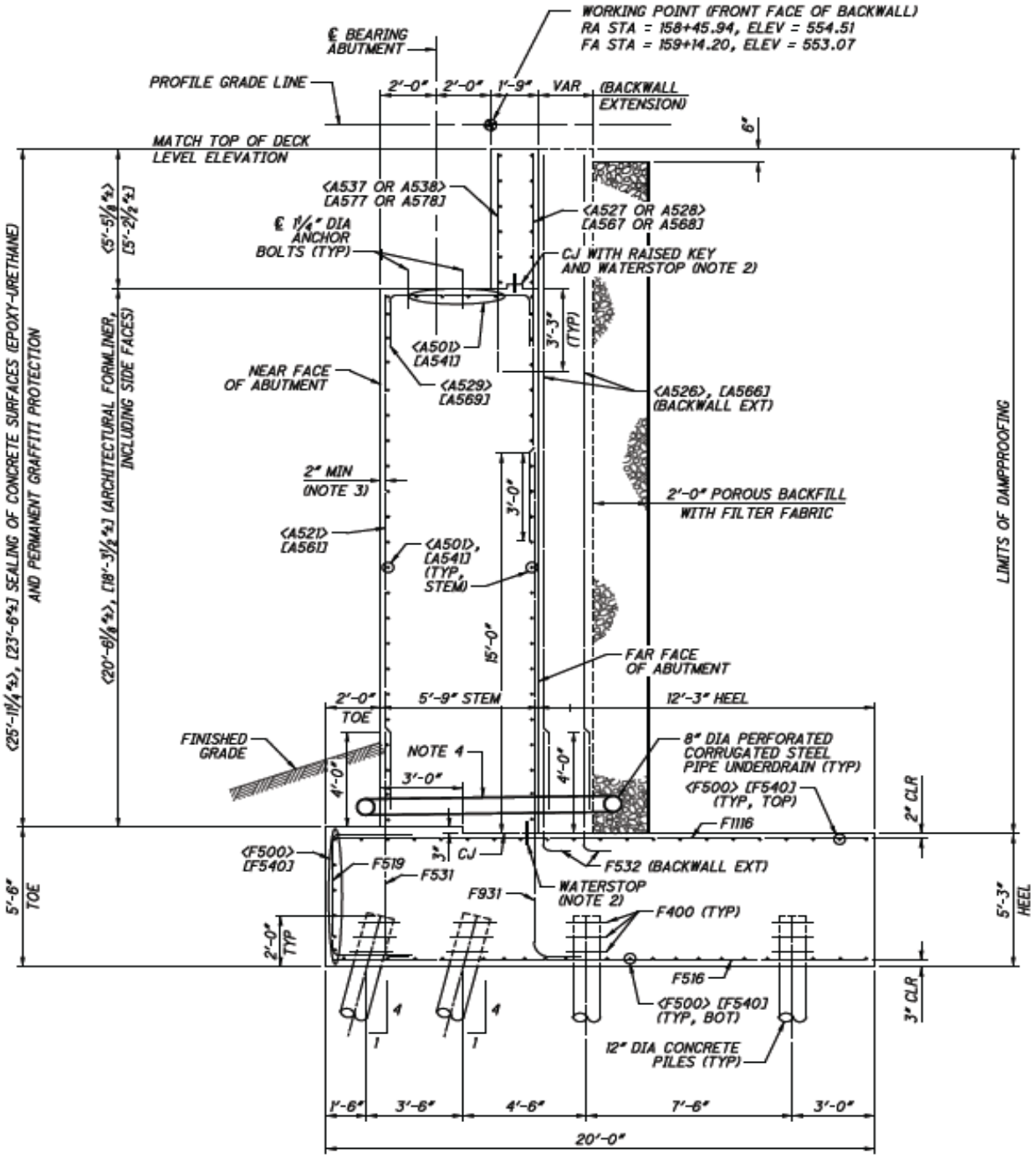
REV	DESCRIPTION	BY	DATE	VERIF'D	DATE
00	Stage 1 Calculation	CTM	11/13/13		
01	Stage 2 Calculation	EFD	05/29/14	VDK	06/16/14

RECORD OF CHECK OF CURRENT CALCULATION VERSION

BY	DATE	CHKD	DATE	UPD.	DATE	VERIF'D	DATE
EFD	05/29/14	VDK	05/29/14	EFD	06/16/14	VDK	06/16/14

Gannett Fleming	GF JOB # 54682	CALC NO.: **HAM075_PROS_04**	REV. NO.: **01**
		PROJECT: **ODOT/EMH&T: HAM-75-7.85**	
ODOT PID NO.: 77889		DESIGNED: *CTM*	DATE: *11/13/13*
BRIDGE NO: *HAM-PROSSER AVE*		Stg. 2 UPDATED: *EFD*	DATE: *05/29/14*
SUBJECT: *Loads*		CHECKED: *VDK*	DATE: *05/29/14*
Abutment Loads for		UPDATED: *EFD*	DATE: *06/16/14*
Rear Abutment (Worst Case)		VERIFIED: *VDK*	DATE: *06/16/14*

Typical Section (Preliminary Stage 2 Detail)



TYPICAL ABUTMENT SECTION

Gannett Fleming	GF JOB # 54682	CALC NO.: **HAM075_PROS_04**	REV. NO.: **01**
		PROJECT: **ODOT/EMH&T: HAM-75-7.85**	
ODOT PID NO.: 77889		DESIGNED: *CTM*	DATE: *11/13/13*
BRIDGE NO: *HAM-PROSSER AVE*		Stg. 2 UPDATED: *EFD*	DATE: *05/29/14*
SUBJECT: *Loads*		CHECKED: *VDK*	DATE: *05/29/14*
Abutment Loads for		UPDATED: *EFD*	DATE: *06/16/14*
Rear Abutment (Worst Case)		VERIFIED: *VDK*	DATE: *06/16/14*

General Layout Data (Fixed, Forward Abutment Analysis.. dimensions are for the rear abutment to be conservative)

Girder Layout (For Distribution)

Typical Girder Spacing:	3.500 ft	
^^ *Normal to the C/L of the deck* ^^		
Girder Spacing Used:	3.743 ft	
		Along Skew
C/C Track Supporting Girders:	10.500 ft	11.230 ft
C/C Lateral Girders:	17.500 ft	18.717 ft
Abutment Stem Length:	32.000 ft	
Number of LL Girders:	4	*Vertical Load Distribution*

Trigonometry multipliers to make loads and dimensions normal/parallel to abutment face

Skew Angle:	20.776 deg	0.36 rad
^^ *Normal to the C/L of the girder* ^^		
Girder Spacing (1/cos):	1.070	
Lateral Loads (sin):	0.355	
Longitudinal Loads (cos):	0.935	
Considered Lateral Girders:	6	*Horiz. Load Distrib.*
Bottom/Ftg. ELEV:	521.00	*Elevations At Back of Backwall of rear abutment*
PGL ELEV:	554.59	

Reaction Data from calculation HAM075_PROSC_03

Girder Reactions Considered - Span 2

Worst Case Reactions - Standard AREMA Live Load (Girder Outside of Curve)

DL Vert	LL Vert	CF Lat	LF Long	WL Lat	WS Lat	EQ Normal	Qg Vert	Impact Load *(Used to Check Bearing Size)*
64.31 kip	122.62 kip	10.01 kip	200.00 kip	9.98 kip	10.39 kip	114.76 kip	220.71 kip	51.22 kip
		(2)	(2)	(2)	(2)	(1)	(4)	

(1) The total earthquake load is assumed to be in any direction. Normal to the face of abutment is considered herein. (parallel to the face of abutment will not control by inspection)

*(2) The horizontal loads (lateral and longitudinal) are the **total force PER abutment.***

(4) This derail load includes impact for substructure design (pier stability)

Abut 5 Load Inputs from Girder Reactions

Abutment Reactions / LF of Abutment

Worst Case Reactions - Standard AREMA Live Load (Girders Outside of Curve)

DL Vert	LL Vert	CF Lat	LF Long	WL Lat	WS Lat	EQ Normal	Qg Vert	Upward Wind
17.18 kip/ft	32.75 kip/ft	0.19 kip/ft	6.85 kip/ft	0.19 kip/ft	0.20 kip/ft	4.32 kip/ft	58.96 kip/ft	0.69 kip/ft
			(1)	(3)	(2)	(4)		

(1) AREMA Commentary 9.1.3.12 explains that there is only a negligible reduction in longitudinal force on a ballasted deck bridge. As such, all of the longitudinal force will be transferred to the fixed abutment. The width that the longitudinal force is distributed across is 66% of the stem width plus 33% of the superstructure width. This recongnizes that stem outside of the superstructure bearing width will resist the bending moment (worst bending moment at the bottom of the stem = more room to dissipate across stem height).

(2) The total reaction given above is per abutment; this load is over the C/C exterior girder width

(3) The total reaction given above is due to a live load per abutment; this load is over the C/C track supporting girders

(4) The earthquake load is reduced by thermal load (calculated below) which allows for the bearing to absorb and transfer some of the ground motion to the expansion abutment. The dynamic coefficient of friction is assumed to be 33% of static (conservative per bronze/steel published values). The EQ load is distributed laterally in the same manner as the longitudinal loads

Train Surcharge Loading

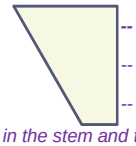
Cooper E-80 Linear Load: 8000 plf
Assumed Soil Density: 110 pcf
^^ Type 2 Soil, per AREMA Table 8-5-2
Typical Tie Width: 8.500 ft

Bottom/Tie to Bottom/Footing: 32.342 ft
Abut5 TOP FTG TO TOP EMBANK --> 27.092 ft
Bottom of Tie to Top of Backwall: 0.833 ft
Footing Thickness 5.250 ft
Stem Height (to top of backwall) --> 26.258 ft

Lateral distribution of surcharge at top of backwall:	9.33 ft	*AREMA 8-5.3.1.b*	
Lateral distribution of surcharge at bottom of footing:	32.00 ft	*AREMA 8-5.3.1.b*	*40.84157*
Equivalent surcharge at the top of backwall:	7.79 ft	*Per foot of abutment width*	
Equivalent surcharge at the bottom of footing:	2.27 ft	*Per foot of abutment width*	

Soil surcharge below is per foot of abutment width, rounded up to the nearest 0.1'

Soil surcharge at a point 1/3 Down from ties	**6.00 ft**	***-- Worst case for overturning & Bending***
Soil surcharge at a point 1/2 Down from ties	5.10 ft	*-- Average Eq. Surcharge (Sliding)*
Soil surcharge at a point 2/3 Down from ties	4.20 ft	*-- Equivilant at the base of the footing*



The worst case surcharge (1/3 down) is used for the Abut5 Analysis because it will give the worst case bending in the stem and the worst case overturning (vertical force on piles as well as bending in the foundation). However, ***this will overestimate the required sliding resistance.*** *Due to that, there is no need to be overly conservative on the sliding resistance of the Abut5 output.*

Additional Longitudinal Backwall Forces

Ref: AREMA 8-5.3.1.c - Increase backwall loads by 100% for earth and surcharge pressures.

No additional loading will be added to Abut5; the required reinforcing will be increased by the factor below:

Multiplier due to increased surcharge on backwall: ***2.30***

*^^ Includes a 100% increase to satisfy AREMA 8-5.3.1.c and a ratio of (top of backall)/(applied surcharge)^^*

Thermal Force & Rough Bearing Size - Span #1 (Used for Abutment Design)

A sliding steel expansion bearing will be assumed. Therefore, the maximum thermal force will be dependent upon the force it takes to overcome the static friction of the steel interfaces. It will be assumed that the sliding plate will be a self-lubricating bronze bearing plate. Reference AREMA 15-5 for design parameters.

Steel Expansion Rate 6.50E-06 *in/in/degF*

Max Thermal Change: 90 degF

Span Length: 64.000 ft

Calculated Max Thermal Movement: 0.449 in

Max Thermal Movement Per AREMA 15-5.1.2.c: 0.800 in *(1.25" per 100 feet in Zone 2)*

Max Thermal Movement Considered: 0.800 in *Per foot of abutment width*

Note: AREMA 15-5.1.3.c will require an additional 1" (+/- 0.5") of movement for construction tolerance.

Bronze Bearing Contact Length: 7.00 in *Parallel to Girder*

Bronze Bearing Contact Width: 17.00 in *Normal to Girder*

Bronze Bearing Contact Area 119.0 sq. in

Static coefficient of friction: 0.10

*^^ (AREMA 15-5.4.3.b) ^^*

Allowable Bronze Bearing Pressure: 2.00 ksi

*^^ (AREMA 15-5.2.3) ^^*

Masonry Plate Length: 15.00 in *Parallel to Girder*

Masonry Plate Width: 25.00 in *Normal to Girder*

Masonry Plate Area 375.0 sq. in

Allowable Conc. Bearing Press.: 1.05 ksi

*^^ (AREMA 15-5.2.7 w/ 4ksi concrete) ^^*

Bearing Forces Considered - Span #1

Note: Friction forces will be tabulated upon dead load forces only. Sizing will be checked upon DL+LL+I+CF (Load Case A)

Dead Load: 64.31 kip Impact Load: 51.22 kip

Live Load: 122.62 kip Vert Cent. Load: 2.03 kip *(Ref: HAM075_PROSC_03)*

Max Static Friction Force: 6.43 kip *From Dead Load*

Thermal Bearing Force used in Abut5: 1.61 kip *Normal to Abutment Face*

Once this force is overcome, the thermal load will simply slide across the pier expansion bearing and induce no further load.

Rough Bearing Size Check

Total Bearing Force: 240.17 kip

Bronze Bearing Stress: 2.018 ksi < 2.00 ksi

Masonry Bearing Stress: 0.640 ksi < 1.05 ksi

Granular Backfill

A select granular backfill will be used behind the abutment.

For the design of the abutment, Backfill Type 2 or 1 of AREMA Table 8-5-1 (Section 8-5.2.5) is assumed.

Using Rankine theory (to match ABUT5 Routine) for a vertical wall, the equivilant fluid pressure is as follows:

Friction btwn backfill and stem concrete δ:	16.0 deg	0.2793	*Half of backfill angle*
Angle of backfill β:	1.4 deg	0.0250	*Assumes a 2.5% profile grade behind the abutment*
Conservative Angle of Internal Friction ϕ:	32.0 deg	0.5585	*AREMA Table 8-5-2 (Type 1), Project Geotech Report uses 33*
Angle of Stem (Fill-Face) θ:	90.0 deg	1.5708	*Angle from horizontal (90 degrees… less than 90 for a batter*

	Coulomb	***Rankine***
K_a Numerator:	**0.719**	**0.719**
K_a denominator Sqrt Term:	0.627	2.308
K_a denominator Outside Term:	0.961	1.000
K_a Denominator:	**2.545**	**2.308**
Coefficient of Active Earth Pressure K_a:	**0.283**	**0.312**

NOTE: Coulomb is calculated simply for reference should it ever be needed.

Rankine formulast are per "All-In-One Civil Engineering PE Exam Guide, Goswami"

Equivalent Fluid Pressure Dry: 34.3 psi *(AREMA Chart C-8-5-6 is conservative and for walls less than 20'. It approximates 31.0 psi for type 1 soil)*

DESIGN CALCULATION COVER SHEET

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PROJECT: **ODOT/EMH&T: HAM-75-7.85**
BRIDGE NO: **HAM-75-PROS**
SUBJECT: ***Analysis***
Abutment, Backwall, Stability
Abut5 Program
FILE NAME(s): *AB5_PROS.dat & AB5_PROS.out*

ODOT PID NO: *77889*
EMH&T JOB NO:
GF JOB NO: 54682
CALC NO.: **HAM075_PROS_05**
REV NO.: **02**
SHEET: 1 OF 10

SUPERSEDES CALC NO: 01 (Stage 2)

CALCULATION DESCRIPTION:

This will analyze the proposed abutment utilizing previously calculated load, geometric, and soil input within the Abut5 program.

The elevations of the abutment analyzed are for the tallest (expansion) bearing, but the fixed loads are applied to it. This should result in an acceptably conservative design for both abutments.

DESIGN BASIS OR REFERENCES:

NS Underpass Design Criteria
AREMA (2013) Chapter 8, Section 5
Calculation: HAM075_PROS_04 (Abutment Loads)
Calculation: HAM075_PROS_01 (Elevation Data)
Stage 1 Plans and Comments
Abut5 Users Manual (PennDOT)

MISCELLANEOUS NOTES:

Abut5 Calculates the abutment self-weight and earth loads automatically. Rankine lateral earth pressures are input into the program (as calculated in PROS_04)

Loads input into Abut5 are be grouped according to AASHTO but are very similar to AREMA Load Cases.

No tension is allowed in piles while vertical surcharge load is applied. Due to the conservative nature, minimal tension will be allowed with NO VERTICAL SURCHARGE in extreme even scenarios only (5% of allowable pile force max)

Stage 3 Update: Re-ran analysis with increased loads due to increased girder section, increased LL distribution, and increased deck thickness

RECORD OF ISSUES:

REV	DESCRIPTION	BY	DATE	VERIF'D	DATE
00	Stage 1 Calculation	EFD	02/08/12		
01	Stage 2, Final Calculation	EFD	05/29/14	VDK	06/16/14
02	Stage 3 Update	VDT	01/20/16	CTM	04/25/16

RECORD OF CHECK OF CURRENT CALCULATION VERSION

BY	DATE	CHKD	DATE	UPD.	DATE	VERIF'D	DATE
EFD	05/29/14	VDK	05/30/14	VDT	01/20/16	CTM	04/25/16

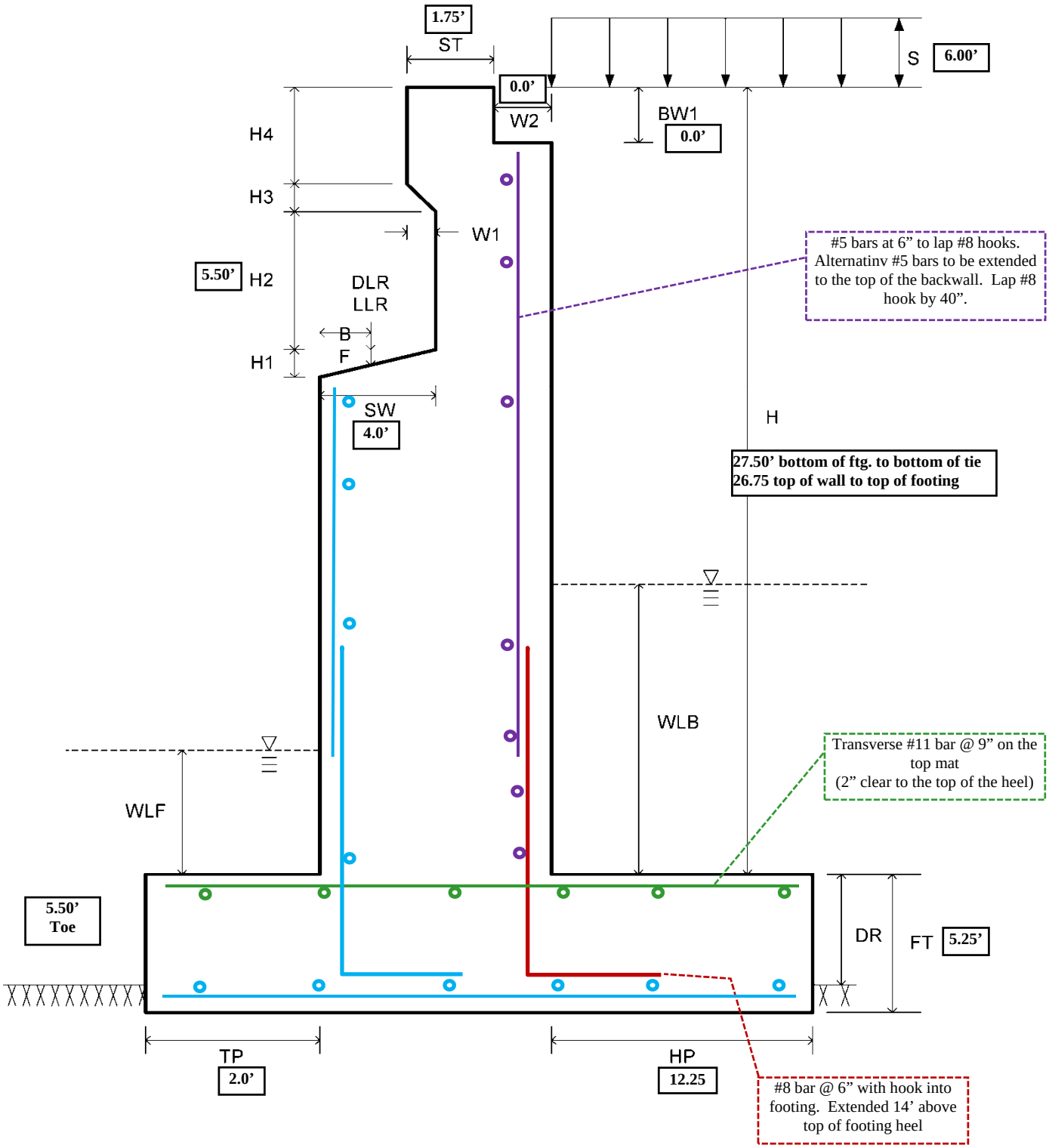

Figure 5.2.1 Input Dimensions - Abutment Type I

NOTE: Heights "H" are rounded up (approximately 6") in this analysis to ensure conservative design will still apply subject to any potential minor elevation changes during final detailing.

Reinforcing Note: All other bars shall be #5's at 12". 3" Clear to bottom of footing. 2" clear to all other surfaces. Vertical stem rebar should be outside of horizontal rebar. Transverse footing rebar should be on the top/bottom of longitudinal rebar.

```
   ---------------------------------------------------------------------------
                                ECHO OF INPUT FILE
   ---------------------------------------------------------------------------
**PROJECT: ODOT/EMH&T: HAM-75-7.85, GF Job # 054682
*CALC: EFD, 5/29/2014
**NSRR Over Prosser Forward (Fixed) Abutment
**Heights are of the rear (exp.) abutment to be conservative
**ASD Design as per AREMA Chapter 8, Part 5 and NS Standards
* ** Created by EngAsst **
* EngAsst Information: [Program=ABUT5] [Version=5.4.0.0]
* ** Data Records Start Here **
*Pressure as per Rankine Equation
SLA12120Y4 343     97519000    2750        0600     40004000400060   000   YYN  ! Criteria
267517502001225         00005500000000000000   0000    02000400      525        ! Dimensions
*EFD Preliminary Numbers
0506080912YSSS000102000300020002750300020004000300                              ! Rebar
*NOTE: Seismic Not Considered for a Single Span as per AREMA
1582282300190019    0066068500160148                       039700950            ! Loading
013000150003001000                                                              ! Pile
023000350003001000                                                              ! Pile
03  0045000600                                                                  ! Pile
04  0075000600                                                                  ! Pile
```

```
                    LFD Abutment and Retaining Wall                  333067

PROGRAM P4354040                                          06/16/2014 09:52
VERSION 5.4.0.0          LAST UPDATED 05/05/2005        DOCUMENTATION 05/2005

INPUT: AB5_PROS.dat

*PROJECT: ODOT/EMH&T: HAM-75-7.85, GF JOB # 054682
CALC: EFD, 5/29/2014
*NSRR OVER PROSSER FORWARD (FIXED) ABUTMENT


                            ANALYSIS OF
                     ABUTMENT WITH BACKWALL TYPE I

                      TYPE OF FOOTING : ON PILES

                                                                EQUIV FLUID
DESIGN                   FTG    PILE            PILE             PRESSURE
METHOD   A OR D   TYPE   TYPE     D   EMBEDDED  ROWS   R OR S     DRY   SAT
  SL       A       1      2     12.0     Y       4               34.3

                      ALLOW SOIL   WATER  TOP FTG   WATER    TOP WALL   LIVE
COEFF OF  BACKFILL  PRESS OR AXIAL LEVEL  TO TOP    LEVEL     TO TOP    LOAD
FRICTION    SLOPE     PILE CAPAC    BACK  EMBANK    FRONT    BACKFILL   SURCH
            97.5        190.00             27.50                        6.00

TOP FTG  F'C                                  LATERAL                     PILE
TO ROCK  BACK   F'C    F'C  REBAR   PILE  PILE  PILE      REBAR  OVR  80%   ROW
SURFACE  WALL   STEM   FTG  GRADE  BATTER  OPT  CAPAC  Kv  DES   STR  RULE  OPT
        4000.  4000.  4000.   60.                0.0        Y     Y    N

  TOP OF
(BACK)WALL                                          MAX
TO TOP OF     TOP      PROJECTION          MAX    FTG
 FOOTING   THICKNESS   TOE   HEEL  T OR H  PROJ  WIDTH   H1     H2    H3    H4
  26.75      1.75      2.00 12.25                       0.00   5.50  0.00  0.00

                           FRONT           HEIGHT
                           FACE   BRIDGE     OF
                           TO DL   SEAT   BACKWALL  BACK   FOOTING  PILE  FTG
 W1   W2   W3   BW1   BW2   REACT   WIDTH   BATTER  BATTER THICKNESS COST  COST
0.00 0.00       0.00         2.00    4.00                    5.25

     TRIAL
 REBAR SPACING  BAR   BW    STEM    FTG    MIN
 1  2  3  4  5  LOC  EXPOS  EXPOS  EXPOS   As
 5  6  8  9 12   Y     S      S      S    0.001

     STEM REBAR COVER           FOOTING REBAR COVER
 VERT  HORIZ   VERT  HORIZ    LONG  TRANS   LONG  TRANS
 2.00   3.00   2.00   2.75    3.00   2.00   4.00   3.00

                                         LONG
                      WIND   WIND        FORCE
 DL     LL    WIND    ON     ON   UPWARD  FROM   CENTR  TEMP
REACT  REACT  ON LL  SUPER   SUB   WIND    LL    FORCE  FORCE
17.18  32.75   0.19   0.20          0.69   6.85   0.19   1.61

                            BACKWALL            ALLOW
    PARAPET OR EXTERNAL     LIVE LOAD  SEISMIC   PILE
 HORZ   DIST   VERT   DIST   VERT   HORZ   LOAD   UPLIFT
                                           4.32    9.50
```

Since piles are friction; it is recognized that some allowable tension can be realistically expected. A conservative 5% of the allowable vertical capacity is assumed.

ROW NO	PILE BATTER	DISTANCE BETWEEN ROWS	PILE SPACING	PERCENT ROW BATTERED
1	3.0	1.50	3.00	100.0
2	3.0	3.50	3.00	100.0
3		4.50	6.00	
4		7.50	6.00	

HP12x53 Piles with ODOT allowable 95T per pile
HP piles are chosen for their higher capacity and lower displacement. Lower displacement is desirable due to the proximity of underground utilities and residential foundations.

```
                        REINFORCEMENT SUMMARY
                        ---------------------
                         REBAR SPACINGS (in):
                                  5.
                                  6.
                                  8.
                                  9.
                                 12.

              REBAR COVERS (in):   (C.G. OF BAR TO OUTER FACE)
                      STEM                      FOOTING
              BACK          FRONT          TOP         BOTTOM
           VERT  HORIZ   VERT  HORIZ  LONG   TRANS  LONG   TRANS
           2.00   3.00   2.00   2.75  3.00    2.00  4.00    3.00

                         BACK WALL -- SEVERE
           EXPOSURES:    STEM ------- SEVERE
                         FOOTING ---- SEVERE

           MINIMUM AREA OF STEEL PER FOOT: 0.001 SQUARE INCHES


***  NOTE  ***
 - THE REBARS IN THE BOTTOM OF THE FOOTING ARE BELOW THE TOP OF THE PILES.


                            PILE FOOTING ANALYSIS
                            ---------------------

***  WARNING  ***
 - THERE IS UPLIFT IN THE FIRST OR LAST PILE ROW.
```

Minimal uplift is present only under extreme event load case without the vertical component of live load surcharge. Given that load case 6 includes thermal along with longitudinal live load and wind, and the fact that those items will never be fully realized at the same time (due to the friction slip nature of thermal loads on sliding plate bearings), minimal uplift of friction piles was deemed acceptable in these limited cases.

```
***  NOTE  ***
 - SINCE Kv WAS NOT ENTERED,  0.0 WAS COMPUTED AND USED.

 - FOR THE TEMPORARY CONSTRUCTION LOADING CONDITION (GROUP T) - FILL HEIGHT
   TO ABUTMENT SEAT, LIVE LOAD SURCHARGE, BUOYANCY AND WIND ON SUBSTRUCTURE
   (IF APPLICABLE):
    > PILE CAPACITY HAS BEEN INCREASED BY A FACTOR OF 1.25.

 - FOR PILE ROWS WITH BOTH VERTICAL AND BATTERED PILES, PILE LOADS ARE
   FOR THE MAXIMUM CONDITION (AXIAL LOAD IN BATTERED PILES).


***  SEISMIC DESIGN - DESIGNER NOTES  ***
 1. BRIDGE BEARING SEAT LENGTH MUST BE CHECKED VERSUS THE MINIMUM REQUIRED
    BEARING SUPPORT LENGTHS AS DETERMINED BY SECTION 4.8, APPENDIX A, DM-4.
 2. VERTICAL REINFORCEMENT ON BACK FACE OF STEM MUST BE HOOKED INTO THE
    FOOTING.
 3. TOP REINFORCEMENT IN THE FOOTING, BOTH LONGITUDINAL AND TRANSVERSE,
    SHALL BE A MINIMUM OF #5's @ 12" SPACING.
 4. VERTICAL REINFORCEMENT, #4's @ 4' SPACING, ARE REQUIRED TO TIE THE
    TOP AND BOTTOM MATS OF REINFORCING IN THE FOOTING TOGETHER.
```

STABILITY ANALYSIS (SERVICE LOADS) - WITH VERTICAL COMPONENT OF LL SURCHARGE

AASHTO GROUP	SUM V	BAL. MOMENT	SUM H	O.T. MOMENT	FRONT ROW TO RESULT	PILE LOAD FRONT	PILE LOAD BACK	PILE LOAD ALLOW	LAT. RESIST	F.S. O.T.
1	**133.59**	**924.08**	**25.32**	**316.21**	**4.55**	**150.60**	**107.97**	**190.00**	**30.72**	**2.92**
2	91.33	731.33	18.59	206.11	5.75	83.12	113.26	237.50	38.40	3.55
3	**133.39**	**923.56**	**32.42**	**504.36**	**3.14**	**184.34**	**40.25**	**237.50**	**38.40**	**1.83**
4	133.59	924.08	26.93	358.87	4.23	158.32	92.61	237.50	38.40	2.57
5	91.33	731.33	20.20	248.77	5.28	90.84	97.91	266.00	43.01	2.94
6	133.39	923.56	34.03	547.02	2.82	192.06	24.90	266.00	43.01	1.69
7	92.02	733.06	22.71	315.29	4.54	103.91	74.03	438.46	70.90	2.33
T	74.14	691.44	18.34	192.09	6.74	54.25	118.21	237.50	38.40	3.60

NOTE: Max Pile Reaction under Primary Loads is 151 Kips. It is 185 kips when longitudinal live load is added.

STABILITY ANALYSIS (SERVICE LOADS) - WITHOUT VERTICAL COMPONENT OF LL SURCHARGE

AASHTO GROUP	SUM V	BAL. MOMENT	SUM H	O.T. MOMENT	FRONT ROW TO RESULT	PILE LOAD FRONT	PILE LOAD BACK	PILE LOAD ALLOW	LAT. RESIST	F.S. O.T.
1	124.77	814.93	25.32	316.21	4.00	153.15	76.01	190.00	30.72	2.58
2	91.33	731.33	18.59	206.11	5.75	83.12	113.26	237.50	38.40	3.55
3	124.57	814.42	32.42	504.36	2.49	186.89	8.30	237.50	38.40	1.61
4	124.77	814.93	26.93	358.87	3.66	160.87	60.66	237.50	38.40	2.27
5	91.33	731.33	20.20	248.77	5.28	90.84	97.91	266.00	43.01	2.94
6	124.57	814.42	34.03	547.02	2.15	194.61	-7.06	266.00	43.01	1.49
7	92.02	733.06	22.71	315.29	4.54	103.91	74.03	438.46	70.90	2.33
T	65.32	582.30	18.34	192.09	5.97	56.79	86.25	237.50	38.40	3.03

NOTE: minimal uplift without vertical component of surcharge under extreme load cases is deemed acceptable for extreme event load cases

FOOTING ANALYSIS (SERVICE LOADS) - WITH VERTICAL COMPONENT OF LL SURCHARGE

AASHTO GROUP	TOE MOMENT	TOE SHEAR @	HEEL MOMENT	HEEL SHEAR @
1	22.78	0.00 D	157.39	19.83 F
2	11.86	0.00 D	103.65	14.95 F
3	28.23	0.00 D	267.35	34.30 F
4	24.02	0.00 D	182.31	23.10 F
5	13.11	0.00 D	128.57	18.22 F
6	29.48	0.00 D	292.27	37.57 F
7	15.23	0.00 D	167.19	23.24 F
T	7.20	0.00 D	96.97	15.59 F

FOOTING ANALYSIS (SERVICE LOADS) - WITHOUT VERTICAL COMPONENT OF LL SURCHARGE

AASHTO GROUP	TOE MOMENT	TOE SHEAR @	HEEL MOMENT	HEEL SHEAR @
1	23.19	0.00 D	157.10	18.88 F
2	11.86	0.00 D	103.65	14.95 F
3	28.64	0.00 D	267.07	33.35 F
4	24.44	0.00 D	182.02	22.16 F
5	13.11	0.00 D	128.57	18.22 F
6	29.89	0.00 D	291.98	36.63 F
7	15.23	0.00 D	167.19	23.24 F
T	7.61	0.00 D	96.68	14.65 F

FOOTING DESIGN - WITH AND WITHOUT VERTICAL COMPONENT OF LL SURCHARGE

FOOTING WIDTH	FOOTING THICKNESS	TOE PROJ	HEEL PROJ	EFFECTIVE DEPTH TOE	EFFECTIVE DEPTH HEEL
20.00	5.25	2.00	12.25	5.000	5.083

SHEAR STR TOE	ALLOWABLE SHEAR TOE	SHEAR STR HEEL	ALLOWABLE SHEAR HEEL	**TRANS REINFORCEMENT TOE-BOT**	**TRANS REINFORCEMENT HEEL-TOP**	LONGITUDINAL REINFORCEMENT
0.000	0.060	0.051	0.060	**0.26**	**1.87**	0.14

LONGITUDINAL MOMENT CALCULATIONS - WITH VERTICAL COMPONENT OF LL SURCHARGE
GROUP 1 GOVERNS

MAX UNIFORM LOAD (w) = 6.680 K/FT
MAX UNIFORM SERVICE LOAD (w) = 6.680 K/FT
MAX PILE SPACING = 6.00 FEET
MAX LONGITUDINAL MOMENT = 12.02 K-FT
MAX LONG. MOMENT (SERVICE) = 12.02 K-FT

PILE PATTERN

ROW	BATTER	DISTANCE	SPACING	% BATTERED
1	3.0 ON 12	1.50	3.00	100.0
2	3.0 ON 12	5.00	3.00	100.0
3	VERT	9.50	6.00	0.0
4	VERT	17.00	6.00	0.0

C.G. OF PILES 6.58 FT. FROM TOE I OF PILES PER FOOT 28.951
PILE DENSITY 1.0000 PILES PER FOOT

STEM ANALYSIS

DESIGN OF STEM SECTION AT 2.75 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	7
MOMENT	0.90	0.12	0.90	0.90	0.12	0.90	0.12
AXIAL FORCE	0.72	0.72	0.72	0.72	0.72	0.72	0.72
SHEAR	0.70	0.13	0.70	0.70	0.13	0.70	0.13

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	BACK FACE REINF
1.750	0.003	0.060	0.010

DESIGN OF STEM SECTION AT 5.50 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	7
MOMENT	4.06	0.95	4.06	4.06	0.95	4.06	0.95
AXIAL FORCE	1.44	1.44	1.44	1.44	1.44	1.44	1.44
SHEAR	1.65	0.52	1.65	1.65	0.52	1.65	0.52

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	BACK FACE REINF
1.750	0.007	0.060	0.104

NOTE: 2.30 x the backwall reinforcing per AREMA 8-5.3.1.c and to account for higher effective surcharge at the top of the backwall
REQUIRED: 0.104 in^2/ft * 2.30 = 0.24 in^2/ft
AREMA Minimum is 0.250 in^2/ft

Proposed: #5's @ 12" = 0.31 in^2/ft

DESIGN OF STEM SECTION AT 10.81 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	7	T
MOMENT	69.39	22.58	106.89	77.94	31.13	115.44	33.71	3.76
AXIAL FORCE	55.96	22.52	55.75	55.96	22.52	55.75	23.21	4.58
SHEAR	4.42	2.21	11.52	6.03	3.82	13.13	4.17	1.58

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	BACK FACE REINF
5.750	0.016	0.060	0.001

DESIGN OF STEM SECTION AT 16.12 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	7	T
MOMENT	101.87	40.38	177.09	118.97	57.49	194.19	61.93	18.47
AXIAL FORCE	60.54	27.10	60.33	60.54	27.10	60.33	27.79	9.16
SHEAR	7.97	4.66	15.07	9.58	6.27	16.68	6.62	4.12

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	BACK FACE REINF
5.750	0.021	0.060	0.311

DESIGN OF STEM SECTION AT 21.44 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	7	T
MOMENT	155.76	73.80	268.70	181.42	99.46	294.36	105.75	49.28
AXIAL FORCE	65.12	31.68	64.91	65.12	31.68	64.91	32.37	13.75
SHEAR	12.48	8.08	19.58	14.09	9.69	21.19	10.04	7.64

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	BACK FACE REINF
5.750	0.026	0.060	1.012

DESIGN OF STEM SECTION AT 26.75 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	7	T
MOMENT	236.22	127.96	386.88	270.43	162.18	421.09	170.33	101.32
AXIAL FORCE	69.70	36.26	69.49	69.70	36.26	69.49	36.95	18.33
SHEAR	17.97	12.47	25.07	19.58	14.08	26.68	14.43	12.12

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	**BACK FACE REINF**
5.750	0.033	0.060	**1.524**

Interpolate between section 16.12 and 21.44 to find the point where #5 bars @ 6” (0.62 sq.in) is OK:
(21.44-16.12)/(1.012-0.311) **= 7.6 feet/SqIn**
(1.012 – 0.62) * 7.6 **= 3.0 Feet**
21.44 – 3.0 feet = **18.44 Feet**
Therefore… 18.44 feet down, or 8.31 feet from the base (26.75-18.04), it is OK to switch to #5 bars @ 6”. Alternate every other one into the backwall to provide backwall reinforcing (#5 @ 12”).

Extend the #8 bars the length of development, so 10 feet (extra foot to be conservative) + Development = 10’ + 4.00’ development (less than 3D cover) = 14’ above base.

SUMMARY OF STEEL DESIGN

FOOTING DESIGN

Top longitudinal reinforcement:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
5	5	0.744
6	5	0.620
8	5	0.465
9	5	0.413
12	5	0.310

Bottom longitudinal reinforcement:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
5	5	0.744
6	5	0.620
8	5	0.465
9	5	0.413
12	5	0.310

Top transverse reinforcement:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
5	8	1.896
6	9	2.000
8	10	1.905
9	11	2.080
12	** NO AVAILABLE DESIGN FOR THIS SPACING **	

Bottom transverse reinforcement:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
5	5	0.744
6	5	0.620
8	5	0.465
9	5	0.413
12	5	0.310

STEM DESIGN

Vertical reinforcement on back face:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
At Section #1, 2.75 feet from the top:		
5	4	0.480
6	4	0.400
8	4	0.300
9	4	0.267
12	4	0.200
At Section #2, 5.50 feet from the top:		
5	4	0.480
6	4	0.400
8	4	0.300
9	4	0.267
12	4	0.200
At Section #3, 10.81 feet from the top:		
5	4	0.480
6	4	0.400
8	4	0.300
9	4	0.267
12	4	0.200
At Section #4, 16.12 feet from the top:		
5	4	0.480
6	4	0.400
8	5	0.465
9	5	0.413
12	6	0.440
At Section #5, 21.44 feet from the top:		
5	6	1.056
6	7	1.200
8	8	1.185
9	8	1.053
12	10	1.270
At Section #6, 26.75 feet from the top:		
5	8	1.896
6	8	1.580
8	10	1.905
9	10	1.693
12	11	1.560

```
          OPTIMUM REBAR DESIGN
          --------------------


FOOTING:


 Top longitudinal reinforcement:
     Spacing       Rebar

       12.             5
        9.             5
        8.             5
        6.             5
        5.             5
Bottom longitudinal reinforcement:
     Spacing       Rebar

       12.             5
        9.             5
        8.             5
        6.             5
        5.             5
  Top transverse reinforcement:
     Spacing       Rebar

        6.             9
        8.            10
        5.             8
        9.            11
 Bottom transverse reinforcement:
     Spacing       Rebar

       12.             5
        9.             5
        6.             5
        8.             5
        5.             5
```

Proposed Footing Reinforcing

Top Mat (2” Clear to Transverse Bars)
#5’s @ 12” Longitudinally
#11’s # 9” Transverse
Bottom Mat (3” Clear to Transverse Bars)
#5’s @ 12” Longitudinally
#5’s # 12” Transverse

```
STEM:


  Here is the design of the vertical reinforcement
  in the back face of the stem.   For this design,
  two  complementary rebar spacings are used;  one
  for the bottom and one for the rest of the stem.



Design #  1:
============
The theoretical cutoff for area of steel = 1.580
sq.in.
using a linear interpolation between design
sections
is  7.32 feet from the top of the footing.

Reqd. extension of reinf. per AASHTO 8.24.1.2.1 :
  1.  15 bar diameters  = 15.000 inches
  2.  effective depth   = 67.000 inches
  3.  1 ft. minimum

      USE               = 67.000 inches


Use # 8 bars at  6. in. up to a height of 12.90
ft.
Above this height use # 8 bars at 12. in.


STEEL DESIGNS LISTED FROM MOST DESIRABLE TO LEAST
OPTIMAL
```

Proposed Primary Backwall Reinforcing

Rear Mat Vertical Bars:
#5’s # 12” (continued into the backwall from the stem)

Proposed Primary Stem Reinforcing

Rear Mat Vertical Bars:
#8’s # 6” hooked into foundation and extending 14’-0” above top of footing (10’-0” Effective after 4’ development)

#5’s # 6” lapping the footing dowel bars. Alternating #5’s will extend into the backwall. The lap should be 40” (Class C Splice to be conservative)

*Note – Plans state #9s @ 6″ for stem reinforcing to be conservative.

DESIGN CALCULATION COVER SHEET

Gannett Fleming
Engineers and Architects, P.C.
Excellence Delivered ***As Promised***

ISO 9001:2008 CERTIFIED

PROJECT:	**ODOT/EMH&T: HAM-75-7.85**	*ODOT PID NO:*	*77889*
BRIDGE NO:	**HAM-075-PROSSER**	*EMH&T JOB NO:*	
SUBJECT:	***Loads***	**GF JOB NO:**	**54682**
	Wingwall Loads for	CALC NO.:	**HAM075_PROS_06**
	Abut5 Program	REV NO.:	**01**
FILE NAME(s):	*HAM075_PROS_06.xlsx*	**SHEET:**	**1 OF 7**

SUPERSEDES CALC NO: 00 (Stage 1 Design)

CALCULATION DESCRIPTION:

This will calculate all loads applied to the wingwall on a per-foot basis for use in the Abut5 program.

DESIGN BASIS OR REFERENCES:

NS Underpass Design Criteria
AREMA (2013) Chapter 8, Section 5
AREMA (2014) Chapter 15, Section 10
Calculation: HAM075_PROSC_03 (Girder Loads and Reactions)
Calculation: HAM075_PROSC_01 (Elevation Data)
Revised Stage 1 Plans (after addressing comments)

MISCELLANEOUS NOTES:

Abut5 Calculates the wingwall self-weight and earth loads automatically.

Loads input into Abut5 will be grouped according to AASHTO.

STAGE 2: Coulomb's equation is used for equivilant pressure.

RECORD OF ISSUES:

REV	DESCRIPTION	BY	DATE	VERIF'D	DATE
00	Stage 1 Calculation	CTM	11/13/13		
01	Stage 2 Calculation	VDT	05/26/14	VDT	06/16/14

RECORD OF CHECK OF CURRENT CALCULATION VERSION

BY	DATE	CHKD	DATE	UPD.	DATE	VERIF'D	DATE
VDT	05/26/14	CTM	05/30/14	CTM	06/12/14	VDT	06/16/14

Gannett Fleming	GF JOB # 54682	CALC NO.: HAM075_PROS_06	REV. NO.: 01
		PROJECT: ODOT/EMH&T: HAM-75-7.85	
ODOT PID NO.: 77889		DESIGNED: VDT	DATE: 05/26/14
BRIDGE NO: HAM-075-PROSSER		CHECKED: CTM	DATE: 05/30/14
SUBJECT: Loads		BACKCHECKED: EFD	DATE: 06/12/14
Wingwall Loads for		UPDATED: CTM	DATE: 06/12/14
SE Wingwall (Worst Case)		VERIFIED: VDT	DATE: 06/16/14

Section 1: SE Wingwall, Station: 157+98.28, Offset: -29.0087

A select granular backfill will be used behind the abutment. For the design of the wingwall, Backfill Type 1 of AREMA Table 8-5-1 (Section 8-5.2.5) is will be specified. Type 2 will be used to be conservative. Coulomb's theory for a vertical wall, the equivilant fluid pressure will be used

Distance from Top of Rail to Top of Wall:		9.00 ft		*Round up… must coincide with a bousenesque level.*
Distance from Fill Face of wall to CL Rail:		30.00 ft		*Normal to the wall, this distance will be a CLT distance in bousensque*
Friction btwn backfill and stem concrete δ:		15.0 deg	0.2618 rad	*1/2 of internal friction angle (conservative)*
Angle of backfill β:	3.3 : 1	17.1 deg	0.2985 rad	*Enter 1000:1 for horz. slope (Top/wall to Bot/tie)*
Angle of Internal Friction ϕ:		30.0 deg	0.5236 rad	*AREMA Table 8-5-2*
Angle of Stem (Fill-Face) θ:	1 : 8.0	82.9 deg	1.4469 rad	*Angle from horizontal (less than 90 for a batter) (Enter 1:100 for a vertical face)*
AREMA Type 2 Soil Density		110.0 pcf		

	Coulomb	Rankine	(Equation from "All-In-One Civil Engineering PE")
K_a Numerator:	***0.849***	**0.849**	
K_a denominator Γ Term:	*2.005*	1.790	
K_a denominator Outside Term:	*0.912*	0.977	
K_a Denominator:	***1.829***	**1.749**	
Coefficient of Active Earth Pressure K_a:	***0.464***	**0.485**	*It should be noted, that the equivilant surcharge is meant to correctly model the moment in the stem. In general, this will overestimate the thrust (sliding). Therefore, as long as the analysis is OK for sliding, there is no need to be extra conservative.*
	For Comparison Only		
Equivalent Fluid Pressure Dry:	***51.0 psi***	**53.4 psi**	
Live Load Surcharge:	**2.6 ft**		*A minimum of 2.5' will be considered for construction loads*
Vertical Fluid Pressure (K_v):	***15.7 psi***	**16.4 psi**	*LRFD Equation 3.11.5.5-2*

Section 2: SE Wingwall, Station: 158+24.72, Offset: -20.21

A select granular backfill will be used behind the abutment. For the design of the wingwall, Backfill Type 1 of AREMA Table 8-5-1 (Section 8-5.2.5) is will be specified. Type 2 will be used to be conservative. Coulomb's theory for a vertical wall, the equivilant fluid pressure will be used

Distance from Top of Rail to Top of Wall:		5.00 ft		*Round up… must coincide with a bousenesque level.*
Distance from Fill Face of wall to CL Rail:		21.50 ft		*Normal to the wall, this distance will be a CLT distance in bousensque*
Friction btwn backfill and stem concrete δ:		15.0 deg	0.2618 rad	*1/2 of internal friction angle (conservative)*
Angle of backfill β:	4.4 : 1	12.7 deg	0.2217 rad	*Enter 1000:1 for horz. slope (Top/wall to Bot/tie)*
Angle of Internal Friction ϕ:		30.0 deg	0.5236 rad	*AREMA Table 8-5-2*
Angle of Stem (Fill-Face) θ:	1 : 8.0	82.9 deg	1.4469 rad	*Angle from horizontal (less than 90 for a batter) (Enter 1:100 for a vertical face)*
AREMA Type 1/2 Soil Density		110.0 pcf		

	Coulomb	Rankine	(Equation from "All-In-One Civil Engineering PE")
K_a Numerator:	***0.849***	**0.849**	
K_a denominator Γ Term:	*2.183*	1.927	
K_a denominator Outside Term:	*0.912*	0.977	
K_a Denominator:	***1.992***	**1.883**	
Coefficient of Active Earth Pressure K_a:	***0.426***	**0.451**	
	For Comparison Only		
Equivalent Fluid Pressure Dry:	***46.9 psi***	**49.6 psi**	
Live Load Surcharge:	**3.5 ft**		*A minimum of 2.5' will be considered for construction loads*
Vertical Fluid Pressure (K_v):	***10.6 psi***	**11.2 psi**	*LRFD Equation 3.11.5.5-2*

Gannett Fleming	GF JOB # 54682	CALC NO.: HAM075_PROS_06	REV. NO.: 01
		PROJECT: ODOT/EMH&T: HAM-75-7.85	
ODOT PID NO.: 77889		DESIGNED: VDT	DATE: 05/26/14
BRIDGE NO: HAM-075-PROSSER		CHECKED: CTM	DATE: 05/30/14
SUBJECT: Loads		BACKCHECKED: EFD	DATE: 06/12/14
Wingwall Loads for		UPDATED: CTM	DATE: 06/12/14
SE Wingwall (Worst Case)		VERIFIED: VDT	DATE: 06/16/14

Section 3: SE Wingwall, Station: 158+33.38, Offset: -17.13

A select granular backfill will be used behind the abutment. For the design of the wingwall, Backfill Type 1 of AREMA Table 8-5-1 (Section 8-5.2.5) is will be specified. Type 2 will be used to be conservative. Coulomb's theory for a vertical wall, the equivilant fluid pressure will be used

Distance from Top of Rail to Top of Wall:	3.00 ft		*Round up… must coincide with a bousenesque level.*
Distance from Fill Face of wall to CL Rail:	18.00 ft		*Normal to the wall, this distance will be a CLT distance in bousensque*
Friction btwn backfill and stem concrete δ:	15.0 deg	0.2618 rad	*1/2 of internal friction angle (conservative)*
Angle of backfill β: 7.2 : 1	7.9 deg	0.1379 rad	*Enter 1000:1 for horz. slope (Top/wall to Bot/tie)*
Angle of Internal Friction ϕ:	30.0 deg	0.5236 rad	*AREMA Table 8-5-2*
Angle of Stem (Fill-Face) θ: 1 : 8.0	82.9 deg	1.4469 rad	*Angle from horizontal (less than 90 for a batter) (Enter 1:100 for a vertical face)*
AREMA Type 1/2 Soil Density	110.0 pcf		

	Coulomb	*Rankine*	*(Equation from "All-In-One Civil Engineering PE")*
K_a Numerator:	***0.849***	**0.849**	
K_a denominator Γ Term:	*2.359*	2.060	
K_a denominator Outside Term:	*0.912*	0.977	
K_a Denominator:	***2.152***	**2.013**	
Coefficient of Active Earth Pressure K_a:	***0.394***	**0.421**	
	For Comparison Only		

Equivalent Fluid Pressure Dry:	***43.4 psi***	**46.4 psi**	
Live Load Surcharge:	**3.9 ft**		*A minimum of 2.5' will be considered for construction loads*
Vertical Fluid Pressure (K_v):	***6.0 psi***	**6.4 psi**	*LRFD Equation 3.11.5.5-2*

Section 4: Caisson Wall Station: 159+10.79

A select granular backfill will be used behind the abutment. For the design of the wingwall, Backfill Type 1 of AREMA Table 8-5-1 (Section 8-5.2.5) is will be specified. Type 2 will be used to be conservative. Coulomb's theory for a vertical wall, the equivilant fluid pressure will be used

Distance from Top of Rail to Top of Wall:	1.00 ft		*Round up… must coincide with a bousenesque level.*
Distance from Fill Face of wall to CL Rail:	18.90 ft		*Normal to the wall, this distance will be a CLT distance in bousensque*
Friction btwn backfill and stem concrete δ:	15.0 deg	0.2618 rad	*1/2 of internal friction angle (conservative)*
Angle of backfill β: 7.0 : 1	8.1 deg	0.1414 rad	*Enter 1000:1 for horz. slope (Top/wall to Bot/tie)*
Angle of Internal Friction ϕ:	30.0 deg	0.5236 rad	*AREMA Table 8-5-2*
Angle of Stem (Fill-Face) θ: 1 : 100.0	90.0 deg	1.5708 rad	*Angle from horizontal (less than 90 for a batter) (Enter 1:100 for a vertical face)*
AREMA Type 1/2 Soil Density	110.0 pcf		

	Coulomb	*Rankine*	*(Equation from "All-In-One Civil Engineering PE"*
K_a Numerator:	***0.750***	**0.750**	
K_a denominator Γ Term:	*2.326*	2.056	
K_a denominator Outside Term:	*0.966*	1.000	
K_a Denominator:	***2.247***	**2.056**	
Coefficient of Active Earth Pressure K_a:	***0.334***	**0.365**	
	For Comparison Only		

Equivalent Fluid Pressure Dry:	***36.7 psi***	**40.1 psi**	
Live Load Surcharge:	**2.5 ft**		*A minimum of 2.5' will be considered for construction loads*
Vertical Fluid Pressure (K_v):	***5.2 psi***	**5.7 psi**	*LRFD Equation 3.11.5.5-2*

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Check:	CTM	Date:	5/30/2014
Verified:	VDT	Date:	6/16/2014

Lateral Pressure from Parallel Strip Loading

4 Different wall distances/heights considered.

Calculations as per Norfolk Southern Design Standards

Min. Depth of Resulting Table (Hs_{min}):	1.00 ft	Centerline Track Distance 1, CLT_1:	30.0 ft
Max Depth of Resulting Table (Hs_{max}):	41.00 ft	Centerline Track Distance 2, CLT_2:	21.5 ft
Spacing of Max Axle Load:	5.0 ft	Centerline Track Distance 3, CLT_3:	18.0 ft
Max Axle Load:	100.0 kip	Centerline Track Distance 4, CLT_4:	18.9 ft
Width of Tie (TL):	8.5 ft	Applied Pressure (q):	2.35 ksf

Hs	**β** *(Radians)*				**α** *(Radians)*				*Active Pressure,* **Ps** *(psf)*			
	β_1	β_2	β_3	β_4	α_1	α_2	α_3	α_4	Ps_1	Ps_2	Ps_3	Ps_4
0.00 ft	0.0000	0.0000	0.0000	0.0000	1.5708	1.5708	1.5708	1.5708	0.00	0.00	0.00	0.00
1.00 ft	0.0096	0.0191	0.0277	0.0250	1.5368	1.5224	1.5120	1.5151	28.81	57.06	82.65	74.61
2.00 ft	0.0192	0.0379	0.0548	0.0495	1.5029	1.4743	1.4538	1.4599	57.21	112.51	161.88	146.45
3.00 ft	0.0286	0.0562	0.0808	0.0731	1.4691	1.4267	1.3964	1.4054	84.83	164.88	234.63	213.01
4.00 ft	0.0378	0.0737	0.1052	0.0954	1.4356	1.3798	1.3403	1.3520	111.31	212.89	298.51	272.27
5.00 ft	0.0468	0.0903	0.1277	0.1162	1.4024	1.3338	1.2859	1.3000	136.32	255.56	351.98	322.83
6.00 ft	0.0555	0.1058	0.1481	0.1351	1.3696	1.2890	1.2334	1.2496	159.60	292.23	394.32	363.97
7.00 ft	0.0638	0.1201	0.1661	0.1521	1.3373	1.2453	1.1829	1.2011	180.92	322.58	425.60	395.55
8.00 ft	0.0718	0.1330	0.1818	0.1671	1.3055	1.2031	1.1347	1.1545	200.13	346.57	446.49	417.97
9.00 ft	0.0793	0.1446	0.1952	0.1801	1.2742	1.1622	1.0888	1.1100	217.11	364.42	458.06	431.99
10.00 ft	0.0863	0.1549	0.2064	0.1912	1.2435	1.1229	1.0452	1.0674	231.83	376.55	461.62	438.58
11.00 ft	0.0930	0.1639	0.2156	0.2005	1.2136	1.0851	1.0039	1.0270	244.28	383.51	458.55	438.86
12.00 ft	0.0991	0.1717	0.2229	0.2081	1.1842	1.0488	0.9647	0.9885	254.51	385.91	450.18	433.94
13.00 ft	0.1048	0.1783	0.2286	0.2141	1.1557	1.0141	0.9277	0.9521	262.60	384.40	437.74	424.88
14.00 ft	0.1100	0.1838	0.2328	0.2188	1.1278	0.9809	0.8928	0.9175	268.66	379.64	422.34	412.66
15.00 ft	0.1147	0.1883	0.2357	0.2223	1.1007	0.9492	0.8598	0.8847	272.83	372.23	404.88	398.12
16.00 ft	0.1190	0.1918	0.2375	0.2247	1.0743	0.9189	0.8286	0.8537	275.27	362.72	386.12	381.97
17.00 ft	0.1228	0.1946	0.2383	0.2261	1.0487	0.8900	0.7992	0.8243	276.12	351.63	366.69	364.82
18.00 ft	0.1262	0.1966	0.2383	0.2268	1.0238	0.8624	0.7715	0.7965	275.57	339.39	347.06	347.15
19.00 ft	0.1292	0.1979	0.2376	0.2267	0.9997	0.8361	0.7452	0.7702	273.77	326.35	327.59	329.34
20.00 ft	0.1318	0.1987	0.2363	0.2261	0.9763	0.8111	0.7204	0.7452	270.88	312.84	308.55	311.69
21.00 ft	0.1341	0.1990	0.2346	0.2249	0.9537	0.7872	0.6970	0.7216	267.06	299.11	290.15	294.41
22.00 ft	0.1361	0.1988	0.2324	0.2234	0.9318	0.7644	0.6748	0.6992	262.45	285.36	272.50	277.67
23.00 ft	0.1377	0.1982	0.2300	0.2215	0.9106	0.7426	0.6538	0.6779	257.18	271.75	255.70	261.58
24.00 ft	0.1390	0.1974	0.2273	0.2193	0.8901	0.7219	0.6339	0.6577	251.38	258.42	239.79	246.21
25.00 ft	0.1401	0.1962	0.2244	0.2169	0.8702	0.7021	0.6151	0.6385	245.16	245.45	224.78	231.60
26.00 ft	0.1409	0.1948	0.2214	0.2144	0.8510	0.6832	0.5971	0.6203	238.61	232.91	210.68	217.78
27.00 ft	0.1415	0.1932	0.2182	0.2116	0.8325	0.6651	0.5801	0.6030	231.83	220.86	197.46	204.74
28.00 ft	0.1419	0.1914	0.2150	0.2088	0.8145	0.6479	0.5640	0.5865	224.89	209.31	185.10	192.47
29.00 ft	0.1421	0.1895	0.2117	0.2059	0.7972	0.6314	0.5486	0.5707	217.85	198.30	173.56	180.95
30.00 ft	0.1421	0.1875	0.2084	0.2030	0.7804	0.6156	0.5340	0.5557	210.78	187.82	162.79	170.16
31.00 ft	0.1420	0.1854	0.2051	0.2000	0.7642	0.6005	0.5200	0.5415	203.73	177.87	152.76	160.05
32.00 ft	0.1418	0.1832	0.2017	0.1970	0.7485	0.5860	0.5067	0.5278	196.72	168.44	143.42	150.60
33.00 ft	0.1414	0.1810	0.1984	0.1939	0.7333	0.5721	0.4940	0.5148	189.81	159.53	134.73	141.77
34.00 ft	0.1409	0.1787	0.1952	0.1909	0.7186	0.5589	0.4819	0.5023	183.01	151.11	126.64	133.52
35.00 ft	0.1403	0.1764	0.1919	0.1880	0.7044	0.5461	0.4703	0.4904	176.35	143.16	119.11	125.82

36.00 ft	0.1396	0.1741	0.1887	0.1850	0.6907	0.5339	0.4592	0.4790	169.85	135.67	112.11	118.63
37.00 ft	0.1388	0.1717	0.1856	0.1821	0.6774	0.5221	0.4486	0.4680	163.52	128.61	105.59	111.91
38.00 ft	0.1380	0.1694	0.1825	0.1792	0.6645	0.5108	0.4384	0.4576	157.38	121.96	99.52	105.64
39.00 ft	0.1371	0.1671	0.1795	0.1764	0.6521	0.5000	0.4287	0.4475	151.42	115.70	93.87	99.79
40.00 ft	0.1361	0.1648	0.1765	0.1736	0.6400	0.4896	0.4194	0.4379	145.66	109.81	88.60	94.32
41.00 ft	0.1351	0.1625	0.1736	0.1708	0.6284	0.4795	0.4104	0.4286	140.09	104.26	83.69	89.21

Note: The following two pages contain numerical and graphical representations of these loads over a selected area.

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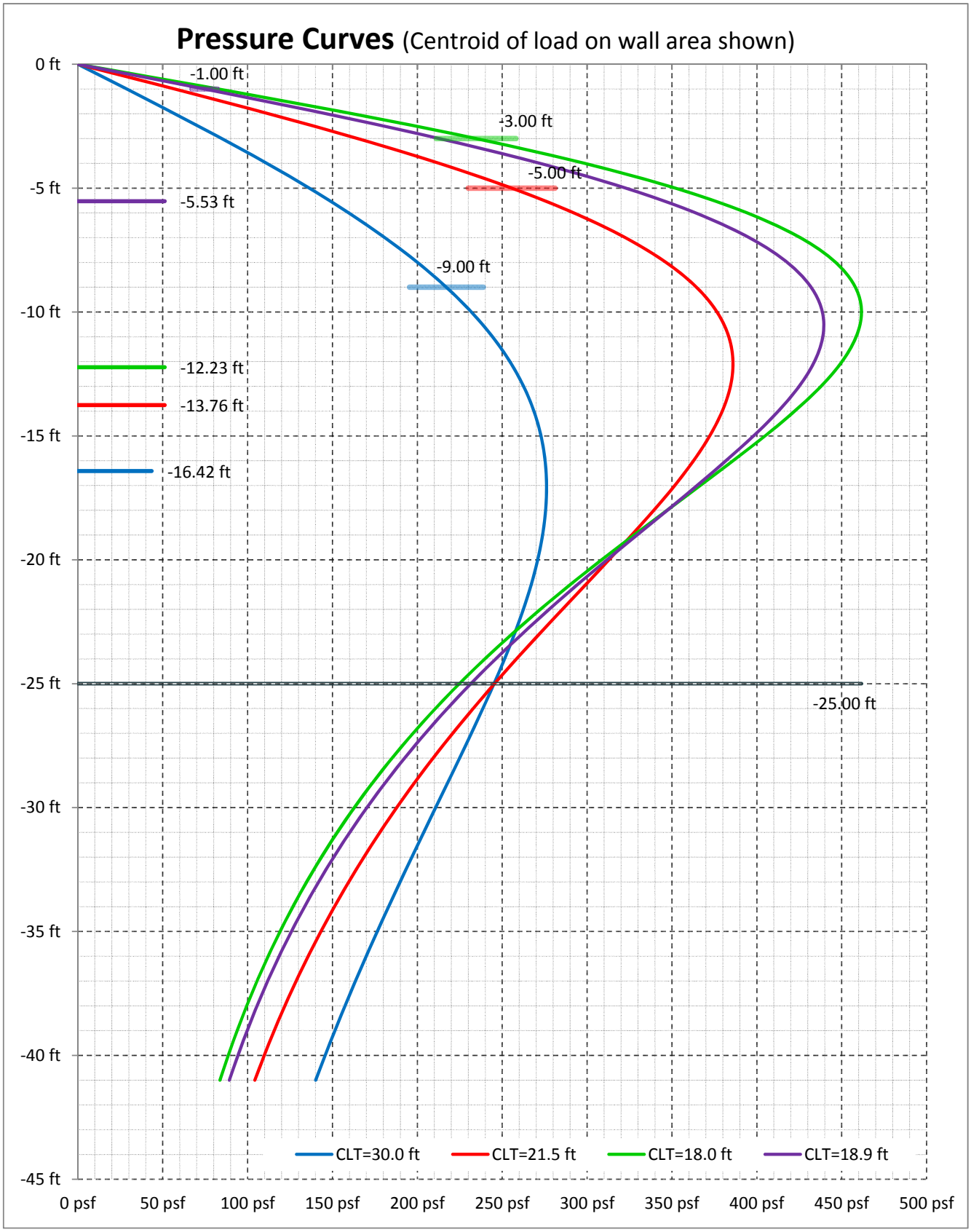
Design:	EFD	Date:	4/15/2014
Update:	VDT	Date:	5/29/2014
Check:	CTM	Date:	5/30/2014
Verified:	VDT	Date:	6/16/2014

Numerical Summary of Forces

To gain total forces, the area under the graph is required. The area under the graph for the considered depth is divided by the total considered depth to arrive at a uniformly applied lateral pressure on any surface in question. The total lateral force is the total area under the graph. Simpsons Rule is used to arrive at the total area under the graph (median value force over a height is used)

Select the Top and Bottom of wall depths for each CLT under consideration. The ZERO elevation is the PGL Elevation. The bottom of wall elevation is the **bottom of stem**.	CLT=30.0 ft	CLT=21.5 ft	CLT=18.0 ft	CLT=18.9 ft
Top of Wall (Min. Depth Considered):	9.00 ft	5.00 ft	3.00 ft	1.00 ft
Bottom of Wall (Max. Depth Considered):	25.00 ft	25.00 ft	25.00 ft	9.00 ft
Height Considered:	**16.00 ft**	**20.00 ft**	**22.00 ft**	**8.00 ft**
Top and Bottom Applied Lateral Pressure on limits of the wall				
	Values at CLT Distances			
Data Below is intended to give summary values that can be used in the design of walls and surfaces affected by the surcharge. A graphical output is on the next page	CLT=30.0 ft	CLT=21.5 ft	CLT=18.0 ft	CLT=18.9 ft
Top of Wall Pressure	217.1 psf	255.6 psf	234.6 psf	74.6 psf
Bottom of Wall Pressure	245.2 psf	245.4 psf	224.8 psf	432.0 psf
Maximum Applied Lateral Pressure on limits of the wall				
Maximum Applied Lateral Pressure	276.1 psf	385.9 psf	461.6 psf	432.0 psf
Total Applied Lateral Pressure on limits of the wall *(as an equivilant point load)*				
Total Lateral Load (Per foot)	4.28 klf	6.79 klf	8.15 klf	2.39 klf
Centroid of Total Lateral Load (Distance down from zero)	16.42 ft	13.76 ft	12.23 ft	5.53 ft
Total Applied Moment due to Surcharge *(assumes fixed at max depth)*	36.8 k-ft	76.4 k-ft	104.1 k-ft	8.3 k-ft
Uniformly Applied Lateral Pressure on limits of the wall (For Lateral Stability) *(Total Applied Lateral Pressure* ***Thrust Controls;*** *Total lateral moment is not correct)*				
Uniformly Applied Lateral Load	267.8 psf	339.7 psf	370.5 psf	298.2 psf
Moment due to Uniform Load *(assumes fixed at max depth)*	34.3 k-ft	67.9 k-ft	89.7 k-ft	9.5 k-ft
Difference in moment from that of the Total Point Load	-7%	-11%	-14%	15%
Uniformly Applied Lateral Pressure on limits of the wall (For Moments/Deflectoins) *(Total Applied Lateral Pressure* ***Moment Controls;*** *Total lateral thrust is not correct)*				
Uniformly Applied Lateral Load	287.3 psf	382.0 psf	430.3 psf	258.9 psf
Moment due to Uniform Load *(assumes fixed at max depth)*	36.8 k-ft	76.4 k-ft	104.1 k-ft	8.3 k-ft
Difference in moment from that of the Total Point Load	0%	0%	0%	0%
Total of Uniformly Applied Lateral Load	4.60 klf	7.64 klf	9.47 klf	2.07 klf
Difference in Load from that of the Total Point Load	7%	12%	16%	-13%
Equivalent Live Load Surcharge (Assumes 110 PCF, AREMA Type 2)	2.6 ft	3.5 ft	3.9 ft	2.4 ft

A minimum of 2.5' of surcharge will be applied to the wingwalls, regardless of the pressure curve. Abut5 just simply takes the surcharge load (does not apply it through boussinesq.


Pressure Curves (Centroid of load on wall area shown)
0 ft
-5 ft
-10 ft
-15 ft
-20 ft
-25 ft
-30 ft
-35 ft
-40 ft
-45 ft
-1.00 ft
-3.00 ft
-5.00 ft
-5.53 ft
-9.00 ft
-12.23 ft
-13.76 ft
-16.42 ft
-25.00 ft
CLT=30.0 ft
CLT=21.5 ft
CLT=18.0 ft
CLT=18.9 ft
0 psf
50 psf
100 psf
150 psf
200 psf
250 psf
300 psf
350 psf
400 psf
450 psf
500 psf

DESIGN CALCULATION COVER SHEET

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ISO 9001:2008 CERTIFIED

PROJECT:	**ODOT/EMH&T: HAM-75-7.85**	*ODOT PID NO:*	*77889*
BRIDGE NO:	**HAM-075-PROSSER**	*EMH&T JOB NO:*	
SUBJECT:	***Loads***	**GF JOB NO:**	**54682**
	Wingwall Loads for	CALC NO.:	**HAM075_PROS_06C**
	Caisson Design MathCad	REV NO.:	**01**
FILE NAME(s):	*HAM075_PROS_06C.xlsx*	**SHEET:**	**1 OF 7**

SUPERSEDES CALC NO: 00 (Stage 1 Design)

CALCULATION DESCRIPTION:

This will calculate all loads applied to the wingwall on a per-foot basis for use in the Abut5 program.

DESIGN BASIS OR REFERENCES:

NS Underpass Design Criteria
AREMA (2013) Chapter 8, Section 5
AREMA (2014) Chapter 15, Section 10
Calculation: HAM075_PROSC_03 (Girder Loads and Reactions)
Calculation: HAM075_PROSC_01 (Elevation Data)
Revised Stage 1 Plans (after addressing comments)

MISCELLANEOUS NOTES:

Abut5 Calculates the wingwall self-weight and earth loads automatically.

RECORD OF ISSUES:

REV	DESCRIPTION	BY	DATE	VERIF'D	DATE
00	Stage 1 Calculation	CTM	11/13/13		
01	Stage 2 Calculation	VDT	06/03/14	CTM	06/16/14

RECORD OF CHECK OF CURRENT CALCULATION VERSION

BY	DATE	CHKD	DATE	UPD.	DATE	VERIF'D	DATE
VDT	06/03/14	CTM	06/12/14	VDT	06/16/14	CTM	06/16/14

Gannett Fleming	GF JOB # 54682	CALC NO.: HAM075_PROS_06C	REV. NO.: 01
		PROJECT: ODOT/EMH&T: HAM-75-7.85	
ODOT PID NO.: 77889		DESIGNED: VDT	DATE: 06/03/14
BRIDGE NO: HAM-075-PROSSER		CHECKED: CTM	DATE: 06/12/14
SUBJECT: Loads		BACKCHECKED: VDT	DATE: 06/16/14
Wingwall Loads for		UPDATED: VDT	DATE: 06/16/14
SW Tangent Pile Wingwall		VERIFIED: CTM	DATE: 06/16/14

Section 1: Temporary Tangent Wall, Exposed Face

Existing Soil Parameters are from Geotechnical Report

Coulomb's theory for a vertical wall, the equivilant fluid pressure will be used

Distance from Top of Rail to Top of Wall:		9.00 ft		Round up… must coincide with a bousenesque level.
Distance from Fill Face of wall to CL Rail:		25.50 ft		Normal to the wall, this distance will be a CLT distance in bousensque
Friction btwn backfill and stem concrete δ:		15.5 deg	0.2705 rad	1/2 of internal friction angle (conservative)
Angle of backfill β:	2.7 : 1	20.4 deg	0.3560 rad	Enter 1000:1 for horz. slope (Top/wall to Bot/tie)
Angle of Internal Friction ϕ:		31.0 deg	0.5411 rad	
Angle of Stem (Fill-Face) θ:	1 : 100.0	90.0 deg	1.5708 rad	Angle from horizontal (less than 90 for a batter) (Enter 1:100 for a vertical face)
AREMA Type 1/2 Soil Density		125.0 pcf		

	Coulomb	Rankine	(Equation from "All-In-One Civil Engineering PE")
K_a Numerator:	***0.735***	**0.735**	
K_a denominator Γ Term:	*1.916*	1.737	
K_a denominator Outside Term:	*0.964*	1.000	
K_a Denominator:	***1.847***	**1.737**	
Coefficient of Active Earth Pressure K_a:	***0.398***	**0.423**	It should be noted, that the equivilant surcharge is meant to correctly model the moment in the stem. In general, this will overestimate the thrust (sliding). Therefore, as long as the analysis is OK for sliding, there is no need to be extra conservative.
	For Comparison Only		
Equivalent Fluid Pressure Dry:	***49.7 psi***	**52.9 psi**	
Live Load Surcharge:	**3.2 ft**		A minimum of 2.5' will be considered for construction loads
Vertical Fluid Pressure (K_v):	***18.5 psi***	**19.7 psi**	LRFD Equation 3.11.5.5-2

Section 2: Temporary Tangent Wall, Buried Portion

Existing Soil Parameters are from Geotechnical Report

Coulomb's theory for a vertical wall, the equivilant fluid pressure will be used

Distance from Top of Rail to Top of Wall:		25.00 ft		Round up… must coincide with a bousenesque level.
Distance from Fill Face of wall to CL Rail:		25.50 ft		Normal to the wall, this distance will be a CLT distance in bousensque
Friction btwn backfill and stem concrete δ:		15.5 deg	0.2705 rad	1/2 of internal friction angle (conservative)
Angle of backfill β:	0.9 : 1	48.4 deg	0.8447 rad	Enter 1000:1 for horz. slope (Top/wall to Bot/tie)
Angle of Internal Friction ϕ:		31.0 deg	0.5411 rad	AREMA Table 8-5-2 (Average of Type 1 & 2… Type 1 will be specified)
Angle of Stem (Fill-Face) θ:	1 : 100.0	90.0 deg	1.5708 rad	Angle from horizontal (less than 90 for a batter) (Enter 1:100 for a vertical face)
AREMA Type 1/2 Soil Density		110.0 pcf		

	Coulomb	Rankine	(Equation from "All-In-One Civil Engineering PE")
K_a Numerator:	***0.735***	**0.735**	
K_a denominator Γ Term:	*#NUM!*	#NUM!	
K_a denominator Outside Term:	*0.964*	1.000	
K_a Denominator:	***#NUM!***	**#NUM!**	
Coefficient of Active Earth Pressure K_a:	***#NUM!***	**#NUM!**	
	For Comparison Only		
Equivalent Fluid Pressure Dry:	***#NUM!***	**#NUM!**	
Live Load Surcharge:	**2.5 ft**		A minimum of 2.5' will be considered for construction loads
Vertical Fluid Pressure (K_v):	***#NUM!***	**#NUM!**	LRFD Equation 3.11.5.5-2

Section 3: Final Tangent Wall, Exposed Face

A select granular backfill will be used behind the abutment. For the design of the wingwall, Backfill Type 1 of AREMA Table 8-5-1 (Section 8-5.2.5) will be specified. Type 2 will be used to be conservative. Coulomb's theory for a vertical wall, the equivilant fluid pressure will be used

Distance from Top of Rail to Top of Wall:	5.00 ft		*Round up… must coincide with a bousenesque level.*
Distance from Fill Face of wall to CL Rail:	19.00 ft		*Normal to the wall, this distance will be a CLT distance in bousensque*
Friction btwn backfill and stem concrete δ:	15.0 deg	0.2618 rad	*1/2 of internal friction angle (conservative)*
Angle of backfill β: 3.8 : 1	14.8 deg	0.2583 rad	*Enter 1000:1 for horz. slope (Top/wall to Bot/tie)*
Angle of Internal Friction ϕ:	30.0 deg	0.5236 rad	*AREMA Table 8-5-2*
Angle of Stem (Fill-Face) θ: 1 : 100.0	90.0 deg	1.5708 rad	*Angle from horizontal (less than 90 for a batter) (Enter 1:100 for a vertical face)*
AREMA Type 1/2 Soil Density	110.0 pcf		

	Coulomb	*Rankine*	*(Equation from "All-In-One Civil Engineering PE")*
K_a Numerator:	***0.750***	**0.750**	
K_a denominator Γ Term:	*2.090*	1.872	
K_a denominator Outside Term:	*0.966*	1.000	
K_a Denominator:	***2.018***	**1.872**	
Coefficient of Active Earth Pressure K_a:	***0.372***	**0.401**	
	For Comparison Only		
Equivalent Fluid Pressure Dry:	***40.9 psi***	**44.1 psi**	
Live Load Surcharge:	**4.2 ft**		*A minimum of 2.5' will be considered for construction loads*
Vertical Fluid Pressure (K_v):	***10.8 psi***	**11.6 psi**	*LRFD Equation 3.11.5.5-2*

Section 4: Final Tangent Wall, Buried Portion

A select granular backfill will be used behind the abutment. For the design of the wingwall, Backfill Type 1 of AREMA Table 8-5-1 (Section 8-5.2.5) is will be specified. Type 2 will be used to be conservative. Coulomb's theory for a vertical wall, the equivilant fluid pressure will be used

Distance from Top of Rail to Top of Wall:	17.00 ft		*Round up… must coincide with a bousenesque level.*
Distance from Fill Face of wall to CL Rail:	19.00 ft		*Normal to the wall, this distance will be a CLT distance in bousensque*
Friction btwn backfill and stem concrete δ:	15.5 deg	0.2705 rad	*1/2 of internal friction angle (conservative)*
Angle of backfill β: 0.9 : 1	47.1 deg	0.8221 rad	*Enter 1000:1 for horz. slope (Top/wall to Bot/tie)*
Angle of Internal Friction ϕ:	31.0 deg	0.5411 rad	*AREMA Table 8-5-2*
Angle of Stem (Fill-Face) θ: 1 : 100.0	90.0 deg	1.5708 rad	*Angle from horizontal (less than 90 for a batter) (Enter 1:100 for a vertical face)*
AREMA Type 1/2 Soil Density	110.0 pcf		

	Coulomb	*Rankine*	*(Equation from "All-In-One Civil Engineering PE")*
K_a Numerator:	***0.735***	**0.735**	
K_a denominator Γ Term:	*#NUM!*	#NUM!	
K_a denominator Outside Term:	*0.964*	1.000	
K_a Denominator:	***#NUM!***	**#NUM!**	
Coefficient of Active Earth Pressure K_a:	***#NUM!***	**#NUM!**	
	For Comparison Only		
Equivalent Fluid Pressure Dry:	***#NUM!***	**#NUM!**	
Live Load Surcharge:	**2.6 ft**		*A minimum of 2.5' will be considered for construction loads*
Vertical Fluid Pressure (K_v):	***#NUM!***	**#NUM!**	*LRFD Equation 3.11.5.5-2*

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Verified:	CTM	Date:	6/16/2014

Lateral Pressure from Parallel Strip Loading

4 Different wall distances/heights considered.

Calculations as per Norfolk Southern Design Standards

Min. Depth of Resulting Table (Hs_{min}):	1.00 ft	Centerline Track Distance 1, CLT_1:	25.5 ft
Max Depth of Resulting Table (Hs_{max}):	41.00 ft	Centerline Track Distance 2, CLT_2:	25.5 ft
Spacing of Max Axle Load:	5.0 ft	Centerline Track Distance 3, CLT_3:	19.0 ft
Max Axle Load:	100.0 kip	Centerline Track Distance 4, CLT_4:	19.0 ft
Width of Tie (TL):	8.5 ft	Applied Pressure (q):	2.35 ksf

Hs	**β** *(Radians)*				**α** *(Radians)*				*Active Pressure,* **Ps** *(psf)*			
	β_1	β_2	β_3	β_4	α_1	α_2	α_3	α_4	Ps_1	Ps_2	Ps_3	Ps_4
0.00 ft	0.0000	0.0000	0.0000	0.0000	1.5708	1.5708	1.5708	1.5708	0.00	0.00	0.00	0.00
1.00 ft	0.0134	0.0134	0.0247	0.0247	1.5305	1.5305	1.5155	1.5155	40.15	40.15	73.79	73.79
2.00 ft	0.0267	0.0267	0.0490	0.0490	1.4903	1.4903	1.4605	1.4605	79.51	79.51	144.87	144.87
3.00 ft	0.0397	0.0397	0.0723	0.0723	1.4504	1.4504	1.4063	1.4063	117.35	117.35	210.79	210.79
4.00 ft	0.0524	0.0524	0.0944	0.0944	1.4109	1.4109	1.3532	1.3532	152.99	152.99	269.56	269.56
5.00 ft	0.0646	0.0646	0.1150	0.1150	1.3720	1.3720	1.3015	1.3015	185.86	185.86	319.80	319.80
6.00 ft	0.0762	0.0762	0.1338	0.1338	1.3337	1.3337	1.2514	1.2514	215.53	215.53	360.79	360.79
7.00 ft	0.0871	0.0871	0.1507	0.1507	1.2961	1.2961	1.2030	1.2030	241.68	241.68	392.38	392.38
8.00 ft	0.0974	0.0974	0.1656	0.1656	1.2594	1.2594	1.1566	1.1566	264.13	264.13	414.93	414.93
9.00 ft	0.1069	0.1069	0.1785	0.1785	1.2236	1.2236	1.1122	1.1122	282.83	282.83	429.17	429.17
10.00 ft	0.1156	0.1156	0.1896	0.1896	1.1887	1.1887	1.0698	1.0698	297.83	297.83	436.06	436.06
11.00 ft	0.1235	0.1235	0.1989	0.1989	1.1549	1.1549	1.0294	1.0294	309.29	309.29	436.67	436.67
12.00 ft	0.1307	0.1307	0.2065	0.2065	1.1221	1.1221	0.9911	0.9911	317.43	317.43	432.11	432.11
13.00 ft	0.1371	0.1371	0.2126	0.2126	1.0903	1.0903	0.9547	0.9547	322.51	322.51	423.40	423.40
14.00 ft	0.1427	0.1427	0.2173	0.2173	1.0596	1.0596	0.9201	0.9201	324.84	324.84	411.51	411.51
15.00 ft	0.1477	0.1477	0.2208	0.2208	1.0300	1.0300	0.8874	0.8874	324.73	324.73	397.28	397.28
16.00 ft	0.1519	0.1519	0.2233	0.2233	1.0014	1.0014	0.8564	0.8564	322.52	322.52	381.41	381.41
17.00 ft	0.1556	0.1556	0.2248	0.2248	0.9739	0.9739	0.8270	0.8270	318.51	318.51	364.51	364.51
18.00 ft	0.1587	0.1587	0.2255	0.2255	0.9473	0.9473	0.7992	0.7992	312.99	312.99	347.06	347.06
19.00 ft	0.1612	0.1612	0.2255	0.2255	0.9218	0.9218	0.7729	0.7729	306.26	306.26	329.44	329.44
20.00 ft	0.1632	0.1632	0.2249	0.2249	0.8973	0.8973	0.7479	0.7479	298.54	298.54	311.94	311.94
21.00 ft	0.1648	0.1648	0.2239	0.2239	0.8737	0.8737	0.7243	0.7243	290.08	290.08	294.79	294.79
22.00 ft	0.1660	0.1660	0.2224	0.2224	0.8511	0.8511	0.7018	0.7018	281.05	281.05	278.16	278.16
23.00 ft	0.1668	0.1668	0.2206	0.2206	0.8293	0.8293	0.6805	0.6805	271.64	271.64	262.15	262.15
24.00 ft	0.1673	0.1673	0.2185	0.2185	0.8083	0.8083	0.6603	0.6603	262.00	262.00	246.85	246.85
25.00 ft	0.1674	0.1674	0.2161	0.2161	0.7882	0.7882	0.6411	0.6411	252.23	252.23	232.29	232.29
26.00 ft	0.1674	0.1674	0.2136	0.2136	0.7689	0.7689	0.6228	0.6228	242.45	242.45	218.51	218.51
27.00 ft	0.1670	0.1670	0.2109	0.2109	0.7503	0.7503	0.6055	0.6055	232.74	232.74	205.49	205.49
28.00 ft	0.1665	0.1665	0.2081	0.2081	0.7324	0.7324	0.5889	0.5889	223.16	223.16	193.24	193.24
29.00 ft	0.1658	0.1658	0.2053	0.2053	0.7153	0.7153	0.5732	0.5732	213.78	213.78	181.73	181.73
30.00 ft	0.1649	0.1649	0.2024	0.2024	0.6988	0.6988	0.5581	0.5581	204.63	204.63	170.94	170.94
31.00 ft	0.1639	0.1639	0.1994	0.1994	0.6829	0.6829	0.5438	0.5438	195.75	195.75	160.83	160.83
32.00 ft	0.1628	0.1628	0.1964	0.1964	0.6676	0.6676	0.5301	0.5301	187.16	187.16	151.37	151.37
33.00 ft	0.1615	0.1615	0.1934	0.1934	0.6529	0.6529	0.5171	0.5171	178.88	178.88	142.53	142.53
34.00 ft	0.1602	0.1602	0.1905	0.1905	0.6387	0.6387	0.5046	0.5046	170.90	170.90	134.26	134.26
35.00 ft	0.1588	0.1588	0.1875	0.1875	0.6251	0.6251	0.4926	0.4926	163.25	163.25	126.54	126.54

36.00 ft	0.1574	0.1574	0.1846	0.1846	0.6119	0.6119	0.4811	0.4811	155.92	155.92	119.33	119.33
37.00 ft	0.1559	0.1559	0.1817	0.1817	0.5993	0.5993	0.4702	0.4702	148.91	148.91	112.60	112.60
38.00 ft	0.1543	0.1543	0.1788	0.1788	0.5871	0.5871	0.4597	0.4597	142.22	142.22	106.31	106.31
39.00 ft	0.1528	0.1528	0.1760	0.1760	0.5753	0.5753	0.4496	0.4496	135.83	135.83	100.44	100.44
40.00 ft	0.1512	0.1512	0.1732	0.1732	0.5639	0.5639	0.4399	0.4399	129.74	129.74	94.94	94.94
41.00 ft	0.1495	0.1495	0.1705	0.1705	0.5529	0.5529	0.4306	0.4306	123.94	123.94	89.81	89.81

Note: The following two pages contain numerical and graphical representations of these loads over a selected area.

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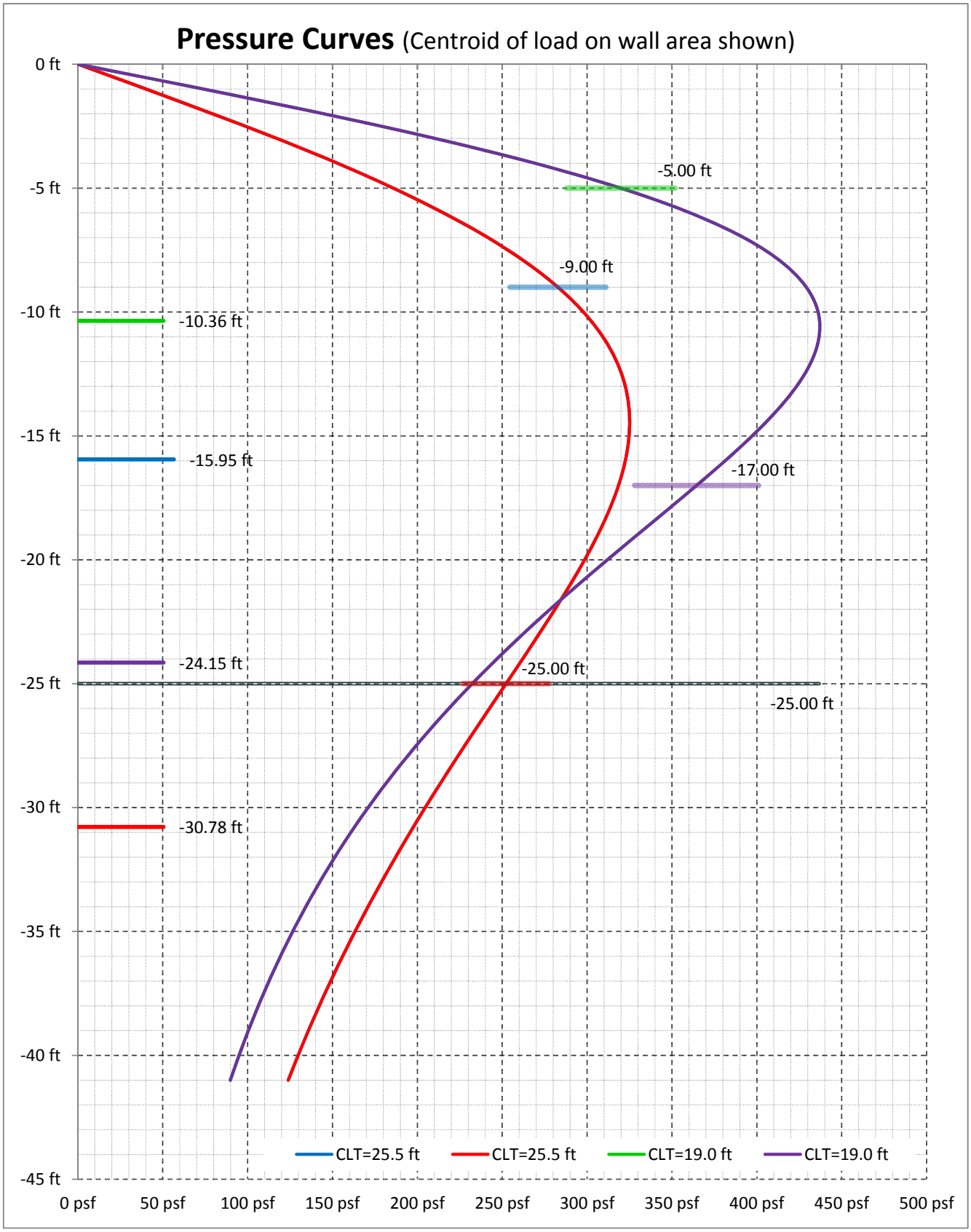
Design:	EFD	Date:	4/15/2014
Update:	VDT	Date:	5/29/2014
Check:	CTM	Date:	6/12/2014
Verified:	CTM	Date:	6/16/2014

Numerical Summary of Forces

To gain total forces, the area under the graph is required. The area under the graph for the considered depth is divided by the total considered depth to arrive at a uniformly applied lateral pressure on any surface in question. The total lateral force is the total area under the graph. Simpsons Rule is used to arrive at the total area under the graph (median value force over a height is used)

Select the Top and Bottom of wall depths for each CLT under consideration. The ZERO elevation is the PGL Elevation. The bottom of wall elevation is the **bottom of stem**.	CLT=25.5 ft	CLT=25.5 ft	CLT=19.0 ft	CLT=19.0 ft
Top of Wall (Min. Depth Considered):	9.00 ft	25.00 ft	5.00 ft	17.00 ft
Bottom of Wall (Max. Depth Considered):	25.00 ft	41.00 ft	17.00 ft	41.00 ft
Height Considered:	**16.00 ft**	**16.00 ft**	**12.00 ft**	**24.00 ft**
Top and Bottom Applied Lateral Pressure on limits of the wall				
	Values at CLT Distances			
Data Below is intended to give summary values that can be used in the design of walls and surfaces affected by the surcharge. A graphical output is on the next page	CLT=25.5 ft	CLT=25.5 ft	CLT=19.0 ft	CLT=19.0 ft
Top of Wall Pressure	282.8 psf	252.2 psf	319.8 psf	364.5 psf
Bottom of Wall Pressure	252.2 psf	123.9 psf	364.5 psf	89.8 psf
Maximum Applied Lateral Pressure on limits of the wall				
Maximum Applied Lateral Pressure	324.8 psf	252.2 psf	436.7 psf	364.5 psf
Total Applied Lateral Pressure on limits of the wall *(as an equivilant point load)*				
Total Lateral Load (Per foot)	4.97 klf	3.04 klf	5.02 klf	4.93 klf
Centroid of Total Lateral Load (Distance down from zero)	15.95 ft	30.78 ft	10.36 ft	24.15 ft
Total Applied Moment due to Surcharge *(assumes fixed at max depth)*	45.0 k-ft	31.1 k-ft	33.3 k-ft	83.1 k-ft
Uniformly Applied Lateral Pressure on limits of the wall (For Lateral Stability) *(Total Applied Lateral Pressure **Thrust Controls;** Total lateral moment is not correct)*				
Uniformly Applied Lateral Load	310.6 psf	190.0 psf	418.1 psf	205.5 psf
Moment due to Uniform Load *(assumes fixed at max depth)*	39.8 k-ft	24.3 k-ft	30.1 k-ft	59.2 k-ft
Difference in moment from that of the Total Point Load	-12%	-22%	-10%	-29%
Uniformly Applied Lateral Pressure on limits of the wall (For Moments/Deflectoins) *(Total Applied Lateral Pressure **Moment Controls;** Total lateral thrust is not correct)*				
Uniformly Applied Lateral Load	351.4 psf	242.7 psf	462.9 psf	288.5 psf
Moment due to Uniform Load *(assumes fixed at max depth)*	45.0 k-ft	31.1 k-ft	33.3 k-ft	83.1 k-ft
Difference in moment from that of the Total Point Load	0%	0%	0%	0%
Total of Uniformly Applied Lateral Load	5.62 klf	3.88 klf	5.56 klf	6.93 klf
Difference in Load from that of the Total Point Load	13%	28%	11%	40%
Equivalent Live Load Surcharge (Assumes 110 PCF, AREMA Type 2)	3.2 ft	2.2 ft	4.2 ft	2.6 ft

A minimum of 2.5' of surcharge will be applied to the wingwalls, regardless of the pressure curve. Abut5 just simply takes the surcharge load (does not apply it through boussinesq.


Pressure Curves (Centroid of load on wall area shown)
0 ft
-5 ft
-10 ft
-15 ft
-20 ft
-25 ft
-30 ft
-35 ft
-40 ft
-45 ft
0 psf
50 psf
100 psf
150 psf
200 psf
250 psf
300 psf
350 psf
400 psf
450 psf
500 psf
-5.00 ft
-9.00 ft
-10.36 ft
-15.95 ft
-17.00 ft
-24.15 ft
-25.00 ft
-25.00 ft
-30.78 ft
CLT=25.5 ft
CLT=25.5 ft
CLT=19.0 ft
CLT=19.0 ft

DESIGN CALCULATION COVER SHEET

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ISO 9001:2008 CERTIFIED

PROJECT: **ODOT/EMH&T: HAM-75-7.85**
BRIDGE NO: **HAM-75-PROS**
SUBJECT: ***Analysis***
Southeast Wingwall
Abut5 Program
FILE NAME(s): *WW1.dat, WW2.dat & WW3.dat*

ODOT PID NO: 77889
EMH&T JOB NO:
GF JOB NO: 54682
CALC NO.: **HAM075_PROS_07**
REV NO.: **01**
SHEET: 1 OF 22

SUPERSEDES CALC NO: 0 (Stage 1)

CALCULATION DESCRIPTION:

This will analyze the proposed wingwalls utilizing full surcharge loading at three sections on the wingwall. The three sections are located at Section 1: STA 157+98.28 (Shorter section on raised footing closer to the edge of the wall), Section 2 : STA 158+24.72 (Taller section on the raised footing towards the abutment) and Section 3: STA 158+33.38 (Taller section adjacent to the abutment on the common abutment footing.)

DESIGN BASIS OR REFERENCES:

NS Underpass Design Criteria
AREMA (2008) Chapter 8, Section 5
Input Load Calculation (HAM075_PROS_06)

MISCELLANEOUS NOTES:

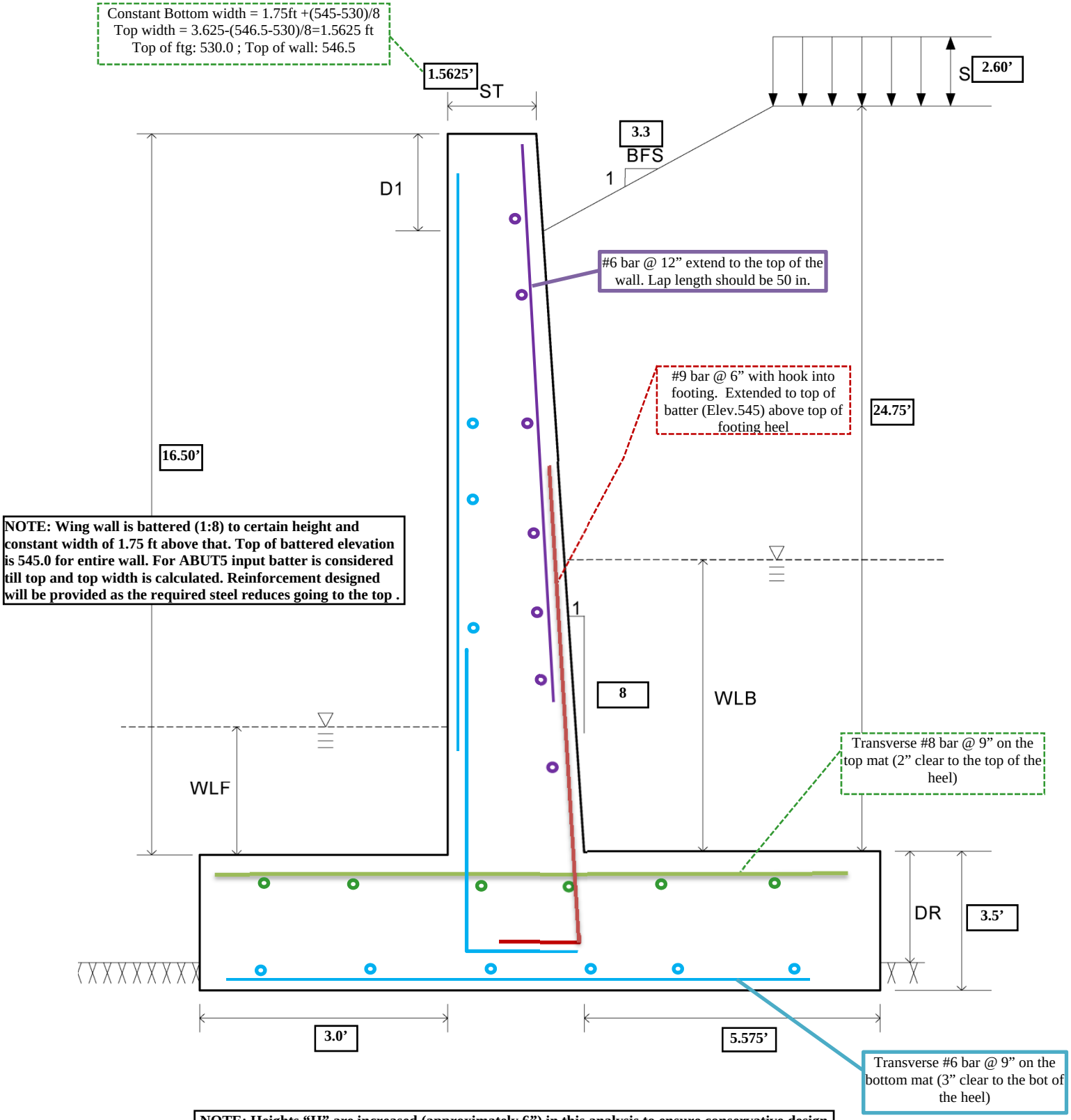
Abut5 Calculates the wingwall self-weight and earth loads automatically.

RECORD OF ISSUES:

REV	DESCRIPTION	BY	DATE	VERIF'D	DATE
00	Stage 1 Calculation	EFD	01/23/12		
01	Stage 2 Calculation	VDK	06/02/14	CTM	06/23/14

RECORD OF CHECK OF CURRENT CALCULATION VERSION

BY	DATE	CHKD	DATE	UPD.	DATE	VERIF'D	DATE
VDK	06/02/14	CTM	06/17/14	VDK	06/17/14	CTM	06/23/14



NOTE: Heights "H" are increased (approximately 6") in this analysis to ensure conservative design will still apply subject to any potential minor elevation changes during final detailing.

Reinforcing Note: All other bars shall be #5's at 12". 3" Clear to bottom of footing. 2" clear to all other surfaces. Vertical stem rebar should be outside of horizontal rebar. Transverse footing rebars should be on the top/bottom of longitudinal rebar.

```
LFD Abutment and Retaining Wall                    333067

PROGRAM P4354040                                          06/02/2014 16:03
VERSION 5.4.0.0           LAST UPDATED 05/05/2005        DOCUMENTATION 05/2005

INPUT: WW_SE1.dat

**PROJECT: ODOT/EMH&T: HAM-75-7.85, GF JOB # 054682
**NSRR OVER PROSSER: SE WINGWALL AT LAST PILE ROW BEFORE STEP;
STATION 157+98.28 OFFSET 29.0087


                              ANALYSIS OF
                             RETAINING WALL

                       TYPE OF FOOTING : ON PILES

                                                                  EQUIV FLUID
DESIGN                   FTG    PILE            PILE              PRESSURE
METHOD   A OR D   TYPE   TYPE     D   EMBEDDED  ROWS   R OR S     DRY   SAT
  SL       A       3      2     12.0     Y       3               53.4

                       ALLOW SOIL   WATER  TOP FTG   WATER    TOP WALL   LIVE
COEFF OF  BACKFILL  PRESS OR AXIAL LEVEL  TO TOP    LEVEL     TO TOP    LOAD
FRICTION    SLOPE     PILE CAPAC    BACK  EMBANK    FRONT    BACKFILL   SURCH
             3.3        190.00             24.75                        2.60

TOP FTG  F'C                                  LATERAL                     PILE
TO ROCK  BACK   F'C    F'C  REBAR   PILE  PILE  PILE      REBAR  OVR  80%   ROW
SURFACE  WALL   STEM   FTG  GRADE  BATTER  OPT  CAPAC  Kv  DES   STR  RULE  OPT
        4000.  4000.  4000.   60.                0.1  16.4  Y     Y

  TOP OF
(BACK)WALL                                          MAX
TO TOP OF     TOP      PROJECTION          MAX    FTG
 FOOTING   THICKNESS   TOE   HEEL  T OR H  PROJ  WIDTH   H1     H2    H3    H4
  16.50      1.56      3.00  5.575

                             FRONT           HEIGHT
                             FACE   BRIDGE     OF
                             TO DL   SEAT   BACKWALL  BACK   FOOTING  PILE  FTG
 W1   W2   W3   BW1   BW2   REACT   WIDTH   BATTER  BATTER THICKNESS COST  COST
                                                        8.    3.50

     TRIAL
 REBAR SPACING  BAR   BW    STEM    FTG    MIN
 1  2  3  4  5  LOC  EXPOS  EXPOS  EXPOS   As
 3  6  8  9 12   Y     S      S      S

     STEM REBAR COVER           FOOTING REBAR COVER
 VERT  HORIZ   VERT  HORIZ    LONG  TRANS   LONG  TRANS
 2.00   3.00   2.00   3.00    3.00   2.00   4.00   3.00

                                          LONG
                       WIND   WIND        FORCE
 DL     LL    WIND    ON     ON   UPWARD  FROM   CENTR  TEMP
REACT  REACT  ON LL  SUPER   SUB   WIND    LL    FORCE  FORCE


                              BACKWALL            ALLOW
    PARAPET OR EXTERNAL       LIVE LOAD  SEISMIC   PILE
 HORZ   DIST   VERT   DIST   VERT   HORZ   LOAD   UPLIFT
                                                  19.00
             DISTANCE           PERCENT
ROW   PILE   BETWEEN    PILE      ROW
NO   BATTER    ROWS    SPACING  BATTERED
 1     3.0      1.50     3.00    100.0
 2     3.0      3.00     3.00    100.0
 3              6.00     6.00
```

```
REINFORCEMENT SUMMARY
---------------------
 REBAR SPACINGS (in):
          3.
          6.
          8.
          9.
         12.

   REBAR COVERS (in):   (C.G. OF BAR TO OUTER FACE)
           STEM                    FOOTING
   BACK          FRONT          TOP         BOTTOM
VERT  HORIZ   VERT  HORIZ  LONG   TRANS  LONG   TRANS
2.00   3.00   2.00   3.00  3.00    2.00  4.00    3.00

              BACK WALL -- SEVERE
EXPOSURES:    STEM ------- SEVERE
              FOOTING ---- SEVERE

MINIMUM AREA OF STEEL PER FOOT: 0.125 SQUARE INCHES
```

HP12x53 Piles with ODOT allowable 95T per pile
HP piles are chosen for their higher capacity and lower displacement. Lower displacement is desirable due to the proximity of underground utilities and residential foundations.

```
***  NOTE  ***
 - THE REBARS IN THE BOTTOM OF THE FOOTING ARE BELOW THE TOP OF THE PILES.
```

PILE FOOTING ANALYSIS

```
***  NOTE  ***
 - FOR THE TEMPORARY CONSTRUCTION LOADING CONDITION (GROUP T) - FILL HEIGHT
   TO ABUTMENT SEAT, LIVE LOAD SURCHARGE, BUOYANCY AND WIND ON SUBSTRUCTURE
   (IF APPLICABLE):
    > PILE CAPACITY HAS BEEN INCREASED BY A FACTOR OF 1.25.

 - FOR PILE ROWS WITH BOTH VERTICAL AND BATTERED PILES, PILE LOADS ARE
   FOR THE MAXIMUM CONDITION (AXIAL LOAD IN BATTERED PILES).
```

STABILITY ANALYSIS (SERVICE LOADS) - WITH VERTICAL COMPONENT OF LL SURCHARGE

AASHTO GROUP	SUM V	BAL. MOMENT	SUM H	O.T. MOMENT	FRONT ROW TO RESULT	PILE LOAD FRONT	PILE LOAD BACK	PILE LOAD ALLOW	LAT. RESIST	F.S. O.T.
1	**33.33**	**211.72**	**16.38**	**133.30**	**2.35**	**48.63**	**25.62**	**190.00**	**30.72**	**1.59**
2	**30.95**	**195.43**	**13.28**	**98.76**	**3.12**	**36.98**	**39.68**	**237.50**	**38.40**	**1.98**
3	33.33	211.72	16.38	133.30	2.35	48.63	25.62	237.50	38.40	1.59
4	33.33	211.72	16.38	133.30	2.35	48.63	25.62	237.50	38.40	1.59
5	30.95	195.43	13.28	98.76	3.12	36.98	39.68	266.00	43.01	1.98
6	33.33	211.72	16.38	133.30	2.35	48.63	25.62	266.00	43.01	1.59
T	32.27	203.17	16.38	133.30	2.16	49.18	20.76	237.50	38.40	1.52

NOTE: Max Pile Reaction under Primary Loads is 49 Kips. It is 40 kips when longitudinal live load is added.

STABILITY ANALYSIS (SERVICE LOADS) - WITHOUT VERTICAL COMPONENT OF LL SURCHARGE

AASHTO GROUP	SUM V	BAL. MOMENT	SUM H	O.T. MOMENT	FRONT ROW TO RESULT	PILE LOAD FRONT	PILE LOAD BACK	PILE LOAD ALLOW	LAT. RESIST	F.S. O.T.
1	30.95	195.43	16.38	133.30	2.01	48.84	16.66	190.00	30.72	1.47
2	30.95	195.43	13.28	98.76	3.12	36.98	39.68	237.50	38.40	1.98
3	30.95	195.43	16.38	133.30	2.01	48.84	16.66	237.50	38.40	1.47
4	30.95	195.43	16.38	133.30	2.01	48.84	16.66	237.50	38.40	1.47
5	30.95	195.43	13.28	98.76	3.12	36.98	39.68	266.00	43.01	1.98
6	30.95	195.43	16.38	133.30	2.01	48.84	16.66	266.00	43.01	1.47
T	29.90	186.88	16.38	133.30	1.79	49.40	11.80	237.50	38.40	1.40

FOOTING ANALYSIS (SERVICE LOADS) - WITH VERTICAL COMPONENT OF LL SURCHARGE

AASHTO GROUP	TOE MOMENT	TOE SHEAR @	HEEL MOMENT	HEEL SHEAR @
1	21.23	0.00 D	40.20	14.17 F
2	15.57	0.00 D	26.31	10.10 F
3	21.23	0.00 D	40.20	14.17 F
4	21.23	0.00 D	40.20	14.17 F
5	15.57	0.00 D	26.31	10.10 F
6	21.23	0.00 D	40.20	14.17 F
T	21.49	0.00 D	43.34	14.98 F

FOOTING ANALYSIS (SERVICE LOADS) - WITHOUT VERTICAL COMPONENT OF LL SURCHARGE

AASHTO GROUP	TOE MOMENT	TOE SHEAR @	HEEL MOMENT	HEEL SHEAR @
1	21.33	0.00 D	41.19	13.93 F
2	15.57	0.00 D	26.31	10.10 F
3	21.33	0.00 D	41.19	13.93 F
4	21.33	0.00 D	41.19	13.93 F
5	15.57	0.00 D	26.31	10.10 F
6	21.33	0.00 D	41.19	13.93 F
T	21.60	0.00 D	44.33	14.74 F

FOOTING DESIGN - WITH AND WITHOUT VERTICAL COMPONENT OF LL SURCHARGE

FOOTING WIDTH	FOOTING THICKNESS	TOE PROJ	HEEL PROJ	EFFECTIVE DEPTH TOE	EFFECTIVE DEPTH HEEL
12.17	3.50	3.00	5.54	3.250	3.333

SHEAR STR TOE	ALLOWABLE SHEAR TOE	SHEAR STR HEEL	ALLOWABLE SHEAR HEEL	TRANS REINFORCEMENT TOE-BOT	TRANS REINFORCEMENT HEEL-TOP	LONGITUDINAL REINFORCEMENT
0.000	0.060	0.031	0.060	0.37	0.70	0.12

LONGITUDINAL MOMENT CALCULATIONS - WITH VERTICAL COMPONENT OF LL SURCHARGE
GROUP 1 GOVERNS

MAX UNIFORM LOAD (w) = 2.739 K/FT
MAX UNIFORM SERVICE LOAD (w) = 2.739 K/FT
MAX PILE SPACING = 6.00 FEET
MAX LONGITUDINAL MOMENT = 4.93 K-FT
MAX LONG. MOMENT (SERVICE) = 4.93 K-FT

PILE PATTERN

ROW	BATTER	DISTANCE	SPACING	% BATTERED
1	3.0 ON 12	1.50	3.00	100.0
2	3.0 ON 12	4.50	3.00	100.0
3	VERT	10.50	6.00	0.0

C.G. OF PILES 4.50 FT. FROM TOE I OF PILES PER FOOT 9.000
PILE DENSITY 0.8333 PILES PER FOOT

STEM ANALYSIS

DESIGN OF STEM SECTION AT 4.12 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	T
MOMENT	5.26	2.39	5.26	5.26	2.39	5.26	1.83
AXIAL FORCE	1.26	1.26	1.26	1.26	1.26	1.26	1.25
SHEAR	2.00	1.10	2.00	2.00	1.10	2.00	1.03

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	BACK FACE REINF
2.076	0.007	0.060	0.125

DESIGN OF STEM SECTION AT 8.25 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	T
MOMENT	18.30	10.57	18.30	18.30	10.57	18.30	9.84
AXIAL FORCE	3.10	3.10	3.10	3.10	3.10	3.10	3.08
SHEAR	4.44	2.97	4.44	4.44	2.97	4.44	2.96

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	BACK FACE REINF
2.591	0.013	0.060	0.349

DESIGN OF STEM SECTION AT 12.38 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	T
MOMENT	43.37	28.41	43.37	43.37	28.41	43.37	27.80
AXIAL FORCE	5.52	5.52	5.52	5.52	5.52	5.52	5.48
SHEAR	7.79	5.75	7.79	7.79	5.75	7.79	5.81

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	BACK FACE REINF
3.107	0.018	0.060	0.710

DESIGN OF STEM SECTION AT 16.50 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	T
MOMENT	84.22	59.67	84.22	84.22	59.67	84.22	59.45
AXIAL FORCE	8.53	8.53	8.53	8.53	8.53	8.53	8.46
SHEAR	12.05	9.44	12.05	12.05	9.44	12.05	9.56

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	BACK FACE REINF
3.623	0.024	0.060	0.978

SUMMARY OF STEEL DESIGN

FOOTING DESIGN

Top longitudinal reinforcement:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
3	5	1.240
6	5	0.620
8	5	0.465
9	5	0.413
12	5	0.310

Bottom longitudinal reinforcement:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
3	5	1.240
6	5	0.620
8	5	0.465
9	5	0.413
12	5	0.310

Top transverse reinforcement:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
3	5	1.240
6	6	0.880
8	7	0.900
9	7	0.800
12	8	0.790

Bottom transverse reinforcement:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
3	5	1.240
6	5	0.620
8	5	0.465
9	5	0.413
12	6	0.440

STEM DESIGN

Vertical reinforcement on back face:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
At Section #3, 4.12 feet from the top:		
3	4	0.800
6	4	0.400
8	4	0.300
9	4	0.267
12	4	0.200
At Section #4, 8.25 feet from the top:		
3	4	0.800
6	4	0.400
8	5	0.465
9	5	0.413
12	6	0.440
At Section #5, 12.38 feet from the top:		
3	4	0.800
6	6	0.880
8	7	0.900
9	7	0.800
12	8	0.790
At Section #6, 16.50 feet from the top:		
3	5	1.240
6	7	1.200
8	8	1.185
9	8	1.053
12	9	1.000

OPTIMUM REBAR DESIGN

FOOTING:

```
Top longitudinal reinforcement:
     Spacing       Rebar

       12.             5
        9.             5
        6.             5
        8.             5
        3.             5
Bottom longitudinal reinforcement:
     Spacing       Rebar

       12.             5
        9.             5
        6.             5
        8.             5
        3.             5
  Top transverse reinforcement:
     Spacing       Rebar

       12.             8
        6.             6
        9.             7
        8.             7
        3.             5
 Bottom transverse reinforcement:
     Spacing       Rebar

       12.             6
        9.             5
        6.             5
        8.             5
        3.             5
```

Proposed Footing Reinforcing

Top Mat:
#5's @ 12" Longitudinally
#8's @ 9" Laterally
Bottom Mat:
#5's @ 12" Longitudinally
#6's @ 9" Laterally
These reinforcement details are used to match the footing designed in the taller wingwall section closer to abutment with raised footing. (File WW2)

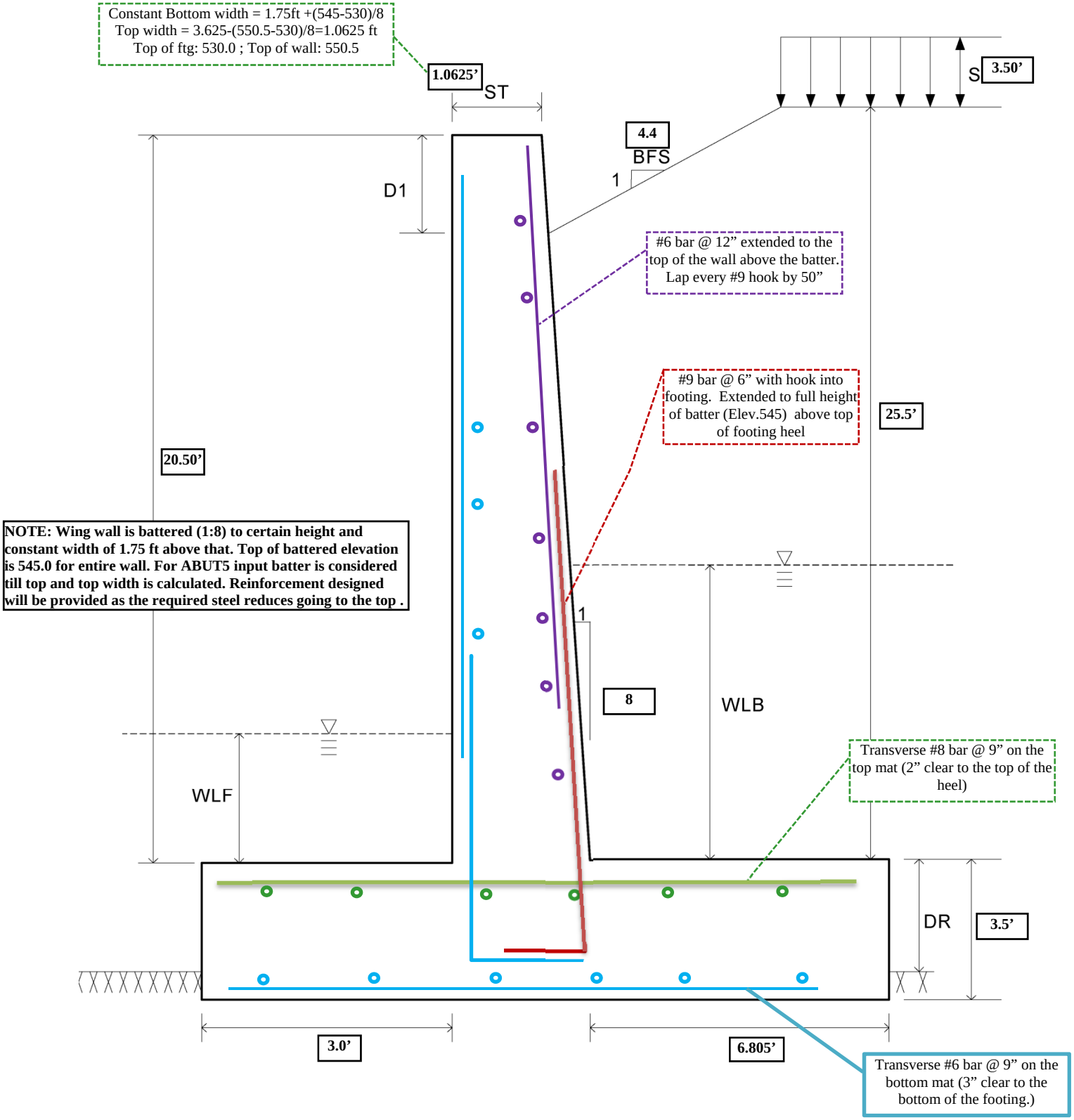
STEM:

```
Design #  1:
============
The theoretical cutoff for area of steel = 1.200
sq.in.
using a linear interpolation between design
sections
is  5.38 feet from the top of the footing.

Reqd. extension of reinf. per AASHTO 8.24.1.2.1 :
  1.  15 bar diameters  = 13.125 inches
  2.  effective depth   = 41.470 inches
  3.  1 ft. minimum

      USE               = 41.470 inches


Use # 7 bars at  6. in. up to a height of  8.84
ft.
Above this height use # 7 bars at 12. in.



Design #  2:
============
The theoretical cutoff for area of steel = 1.240
sq.in.
using a linear interpolation between design
sections
is  5.16 feet from the top of the footing.

Reqd. extension of reinf. per AASHTO 8.24.1.2.1 :
  1.  15 bar diameters  =  9.375 inches
  2.  effective depth   = 41.470 inches
  3.  1 ft. minimum

      USE               = 41.470 inches


Use # 5 bars at  3. in. up to a height of  8.61
ft.
Above this height use # 5 bars at  6. in.


STEEL DESIGNS LISTED FROM MOST DESIRABLE TO LEAST
OPTIMAL
```

Proposed Primary Stem Reinforcing

Rear Mat Vertical Bars:
#9's @ 6" hooked into foundation and extending to top of the batter (elevation 545) above top of footing.

#6's @ 12" extend to the top of the wall above the batter. These reinforcement details are used to match the stem (top of the batter) designed in the taller wingwall section closer to abutment with raised footing. (File WW2 & 3)



NOTE: Heights "H" are increased (approximately 6") in this analysis to ensure conservative design will still apply subject to any potential minor elevation changes during final detailing.

Reinforcing Note: All other bars shall be #5's at 12". 3" Clear to bottom of footing. 2" clear to all other surfaces. Vertical stem rebar should be outside of horizontal rebar. Transverse footing rebars should be on the top/bottom of longitudinal rebar.

```
LFD Abutment and Retaining Wall                    333067

PROGRAM P4354040                                          06/02/2014 17:22
VERSION 5.4.0.0           LAST UPDATED 05/05/2005        DOCUMENTATION 05/2005

INPUT: WW_SE2.dat

**PROJECT: ODOT/EMH&T: HAM-75-7.85, GF JOB # 054682
**NSRR OVER PROSSER: SE WINGWALL AT LAST PILE ROW BEFORE STEP;
STATION 157+24.70 OFFSET 20.21


                              ANALYSIS OF
                             RETAINING WALL

                       TYPE OF FOOTING : ON PILES

                                                                  EQUIV FLUID
DESIGN                   FTG    PILE            PILE              PRESSURE
METHOD   A OR D   TYPE   TYPE     D   EMBEDDED  ROWS   R OR S     DRY   SAT
  SL       A       3      2     12.0     Y       3               49.6

                       ALLOW SOIL   WATER  TOP FTG   WATER    TOP WALL   LIVE
COEFF OF  BACKFILL  PRESS OR AXIAL LEVEL  TO TOP    LEVEL     TO TOP    LOAD
FRICTION    SLOPE     PILE CAPAC    BACK  EMBANK    FRONT    BACKFILL   SURCH
             4.4        190.00             25.50                        3.50

TOP FTG  F'C                                  LATERAL                     PILE
TO ROCK  BACK   F'C    F'C  REBAR   PILE  PILE  PILE      REBAR  OVR  80%   ROW
SURFACE  WALL   STEM   FTG  GRADE  BATTER  OPT  CAPAC  Kv  DES   STR  RULE  OPT
        4000.  4000.  4000.   60.                0.1  11.2  Y     Y

  TOP OF
(BACK)WALL                                          MAX
TO TOP OF     TOP      PROJECTION          MAX    FTG
 FOOTING   THICKNESS   TOE   HEEL  T OR H  PROJ  WIDTH   H1     H2    H3    H4
  20.50      1.06      3.00  6.80

                            FRONT           HEIGHT
                            FACE   BRIDGE     OF
                            TO DL   SEAT   BACKWALL  BACK   FOOTING  PILE  FTG
 W1   W2   W3   BW1   BW2   REACT   WIDTH   BATTER  BATTER THICKNESS COST  COST
                                                      8.    3.50

     TRIAL
 REBAR SPACING  BAR   BW    STEM    FTG    MIN
 1  2  3  4  5  LOC  EXPOS  EXPOS  EXPOS   As
 3  6  8  9 12   Y     S      S      S

     STEM REBAR COVER           FOOTING REBAR COVER
 VERT  HORIZ   VERT  HORIZ    LONG  TRANS   LONG  TRANS
 2.00   3.00   2.00   3.00    3.00   2.00   4.00   3.00

                                          LONG
                      WIND   WIND         FORCE
 DL     LL    WIND    ON     ON   UPWARD  FROM   CENTR  TEMP
REACT  REACT  ON LL  SUPER   SUB   WIND    LL    FORCE  FORCE


                              BACKWALL            ALLOW
    PARAPET OR EXTERNAL       LIVE LOAD  SEISMIC   PILE
 HORZ   DIST   VERT   DIST   VERT   HORZ   LOAD   UPLIFT
                                                  19.00
             DISTANCE           PERCENT
ROW   PILE   BETWEEN    PILE      ROW
NO   BATTER    ROWS    SPACING  BATTERED
 1     3.0      1.50     3.00    100.0
 2     3.0      3.00     3.00    100.0
 3              6.00     6.00
```

```
                    REINFORCEMENT SUMMARY
                    ---------------------
                     REBAR SPACINGS (in):
                              3.
                              6.
                              8.
                              9.
                             12.

          REBAR COVERS (in):   (C.G. OF BAR TO OUTER FACE)
                  STEM                      FOOTING
          BACK          FRONT          TOP         BOTTOM
       VERT  HORIZ   VERT  HORIZ  LONG   TRANS  LONG   TRANS
       2.00   3.00   2.00   3.00  3.00    2.00  4.00    3.00

                     BACK WALL -- SEVERE
       EXPOSURES:    STEM ------- SEVERE
                     FOOTING ---- SEVERE

       MINIMUM AREA OF STEEL PER FOOT: 0.125 SQUARE INCHES


***  NOTE  ***
 - THE REBARS IN THE BOTTOM OF THE FOOTING ARE BELOW THE TOP OF THE PILES.


                        PILE FOOTING ANALYSIS
                        ---------------------

***  NOTE  ***
 - FOR THE TEMPORARY CONSTRUCTION LOADING CONDITION (GROUP T) - FILL HEIGHT
   TO ABUTMENT SEAT, LIVE LOAD SURCHARGE, BUOYANCY AND WIND ON SUBSTRUCTURE
   (IF APPLICABLE):
    > PILE CAPACITY HAS BEEN INCREASED BY A FACTOR OF 1.25.

 - FOR PILE ROWS WITH BOTH VERTICAL AND BATTERED PILES, PILE LOADS ARE
   FOR THE MAXIMUM CONDITION (AXIAL LOAD IN BATTERED PILES).
```

HP12x53 Piles with ODOT allowable 95T per pile
HP piles are chosen for their higher capacity and lower displacement. Lower displacement is desirable due to the proximity of underground utilities and residential foundations.

STABILITY ANALYSIS (SERVICE LOADS) - WITH VERTICAL COMPONENT OF LL SURCHARGE

AASHTO GROUP	BAL. SUM V	BAL. MOMENT	O.T. SUM H	O.T. MOMENT	FRONT ROW TO RESULT	PILE LOAD FRONT	PILE LOAD BACK	PILE LOAD ALLOW	LAT. RESIST	F.S. O.T.
1	**43.05**	**297.10**	**21.46**	**206.68**	**2.10**	**66.57**	**25.84**	**190.00**	**30.72**	**1.44**
2	**39.12**	**268.66**	**16.93**	**147.43**	**3.10**	**47.07**	**49.52**	**237.50**	**38.40**	**1.82**
3	43.05	297.10	21.46	206.68	2.10	66.57	25.84	237.50	38.40	1.44
4	43.05	297.10	21.46	206.68	2.10	66.57	25.84	237.50	38.40	1.44
5	39.12	268.66	16.93	147.43	3.10	47.07	49.52	266.00	43.01	1.82
6	43.05	297.10	21.46	206.68	2.10	66.57	25.84	266.00	43.01	1.44
T	41.86	286.60	21.46	206.68	1.91	67.47	19.79	237.50	38.40	1.39

NOTE: Max Pile Reaction under Primary Loads is 67 Kips. It is 50 kips when longitudinal live load is added.

STABILITY ANALYSIS (SERVICE LOADS) - WITHOUT VERTICAL COMPONENT OF LL SURCHARGE

AASHTO GROUP	BAL. SUM V	BAL. MOMENT	O.T. SUM H	O.T. MOMENT	FRONT ROW TO RESULT	PILE LOAD FRONT	PILE LOAD BACK	PILE LOAD ALLOW	LAT. RESIST	F.S. O.T.
1	39.12	268.66	21.46	206.68	1.58	67.43	10.02	190.00	30.72	1.30
2	39.12	268.66	16.93	147.43	3.10	47.07	49.52	237.50	38.40	1.82
3	39.12	268.66	21.46	206.68	1.58	67.43	10.02	237.50	38.40	1.30
4	39.12	268.66	21.46	206.68	1.58	67.43	10.02	237.50	38.40	1.30
5	39.12	268.66	16.93	147.43	3.10	47.07	49.52	266.00	43.01	1.82
6	39.12	268.66	21.46	206.68	1.58	67.43	10.02	266.00	43.01	1.30
T	37.93	258.15	21.46	206.68	1.36	68.33	3.97	237.50	38.40	1.25

FOOTING ANALYSIS (SERVICE LOADS) - WITH VERTICAL COMPONENT OF LL SURCHARGE

AASHTO GROUP	TOE MOMENT	TOE SHEAR @	HEEL MOMENT	HEEL SHEAR @
1	29.93	0.00 D	76.34	21.40 F
2	20.47	0.00 D	51.34	14.60 F
3	29.93	0.00 D	76.34	21.40 F
4	29.93	0.00 D	76.34	21.40 F
5	20.47	0.00 D	51.34	14.60 F
6	29.93	0.00 D	76.34	21.40 F
T	30.36	0.00 D	80.24	22.41 F

FOOTING ANALYSIS (SERVICE LOADS) - WITHOUT VERTICAL COMPONENT OF LL SURCHARGE

AASHTO GROUP	TOE MOMENT	TOE SHEAR @	HEEL MOMENT	HEEL SHEAR @
1	30.34	0.00 D	76.86	21.18 F
2	20.47	0.00 D	51.34	14.60 F
3	30.34	0.00 D	76.86	21.18 F
4	30.34	0.00 D	76.86	21.18 F
5	20.47	0.00 D	51.34	14.60 F
6	30.34	0.00 D	76.86	21.18 F
T	30.78	0.00 D	80.77	22.19 F

FOOTING DESIGN - WITH AND WITHOUT VERTICAL COMPONENT OF LL SURCHARGE

FOOTING WIDTH	FOOTING THICKNESS	TOE PROJ	HEEL PROJ	EFFECTIVE DEPTH TOE	EFFECTIVE DEPTH HEEL
13.42	3.50	3.00	6.79	3.250	3.333

SHEAR STR TOE	ALLOWABLE SHEAR TOE	SHEAR STR HEEL	ALLOWABLE SHEAR HEEL	TRANS REINFORCEMENT TOE-BOT	TRANS REINFORCEMENT HEEL-TOP	LONGITUDINAL REINFORCEMENT
0.000	0.060	0.047	0.060	0.53	1.01	0.12

LONGITUDINAL MOMENT CALCULATIONS - WITH VERTICAL COMPONENT OF LL SURCHARGE
GROUP 1 GOVERNS

MAX UNIFORM LOAD (w) = 3.209 K/FT
MAX UNIFORM SERVICE LOAD (w) = 3.209 K/FT
MAX PILE SPACING = 6.00 FEET
MAX LONGITUDINAL MOMENT = 5.78 K-FT
MAX LONG. MOMENT (SERVICE) = 5.78 K-FT

PILE PATTERN

ROW	BATTER	DISTANCE	SPACING	% BATTERED
1	3.0 ON 12	1.50	3.00	100.0
2	3.0 ON 12	4.50	3.00	100.0
3	VERT	10.50	6.00	0.0

C.G. OF PILES 4.50 FT. FROM TOE I OF PILES PER FOOT 9.000
PILE DENSITY 0.8333 PILES PER FOOT

STEM ANALYSIS

DESIGN OF STEM SECTION AT 5.12 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	T
MOMENT	7.74	3.18	7.74	7.74	3.18	7.74	3.42
AXIAL FORCE	1.26	1.26	1.26	1.26	1.26	1.26	1.26
SHEAR	2.56	1.30	2.56	2.56	1.30	2.56	1.54

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	BACK FACE REINF
1.701	0.012	0.060	0.255

DESIGN OF STEM SECTION AT 10.25 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	T
MOMENT	29.10	15.80	29.10	29.10	15.80	29.10	18.17
AXIAL FORCE	3.43	3.43	3.43	3.43	3.43	3.43	3.40
SHEAR	5.95	3.80	5.95	5.95	3.80	5.95	4.38

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	BACK FACE REINF
2.341	0.019	0.060	0.650

DESIGN OF STEM SECTION AT 15.38 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	T
MOMENT	71.24	44.65	71.24	71.24	44.65	71.24	50.94
AXIAL FORCE	6.48	6.48	6.48	6.48	6.48	6.48	6.43
SHEAR	10.63	7.60	10.63	10.63	7.60	10.63	8.53

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	BACK FACE REINF
2.982	0.026	0.060	0.996

DESIGN OF STEM SECTION AT 20.50 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	T
MOMENT	140.87	96.43	140.87	140.87	96.43	140.87	108.45
AXIAL FORCE	10.44	10.44	10.44	10.44	10.44	10.44	10.35
SHEAR	16.62	12.70	16.62	16.62	12.70	16.62	13.98

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	BACK FACE REINF
3.623	0.033	0.060	1.626

SUMMARY OF STEEL DESIGN

FOOTING DESIGN

Top longitudinal reinforcement:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
3	5	1.240
6	5	0.620
8	5	0.465
9	5	0.413
12	5	0.310

Bottom longitudinal reinforcement:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
3	5	1.240
6	5	0.620
8	5	0.465
9	5	0.413
12	5	0.310

Top transverse reinforcement:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
3	5	1.240
6	7	1.200
8	8	1.185
9	8	1.053
12	9	1.000 *

Bottom transverse reinforcement:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
3	5	1.240
6	5	0.620
8	6	0.660
9	6	0.587
12	7	0.600

* - INDICATES THAT AREA SELECTED IS LESS THAN REQUIRED BUT WITHIN TOLERANCE

STEM DESIGN

Vertical reinforcement on back face:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
At Section #3, 5.12 feet from the top:		
3	4	0.800
6	4	0.400
8	4	0.300
9	4	0.267
12	5	0.310
At Section #4, 10.25 feet from the top:		
3	4	0.800
6	6	0.880
8	6	0.660
9	7	0.800
12	8	0.790
At Section #5, 15.38 feet from the top:		
3	5	1.240
6	7	1.200
8	8	1.185
9	8	1.053
12	9	1.000
At Section #6, 20.50 feet from the top:		
3	6	1.760
6	9	2.000
8	10	1.905
9	10	1.693
12 ** NO AVAILABLE DESIGN FOR THIS SPACING		

OPTIMUM REBAR DESIGN

FOOTING:

```
Top longitudinal reinforcement:
     Spacing       Rebar

       12.             5
        9.             5
        6.             5
        8.             5
        3.             5
Bottom longitudinal reinforcement:
     Spacing       Rebar

       12.             5
        9.             5
        6.             5
        8.             5
        3.             5
  Top transverse reinforcement:
     Spacing       Rebar

       12.             9
        9.             8
        6.             7
        8.             8
        3.             5
 Bottom transverse reinforcement:
     Spacing       Rebar

       12.             7
        9.             6
        6.             5
        8.             6
        3.             5
```

Proposed Footing Reinforcing

Top Mat:
#5's @ 12" Longitudinally
#8's @ 9" Laterally
Bottom Mat:
#5's @ 12" Longitudinally
#6's @ 9" Laterally

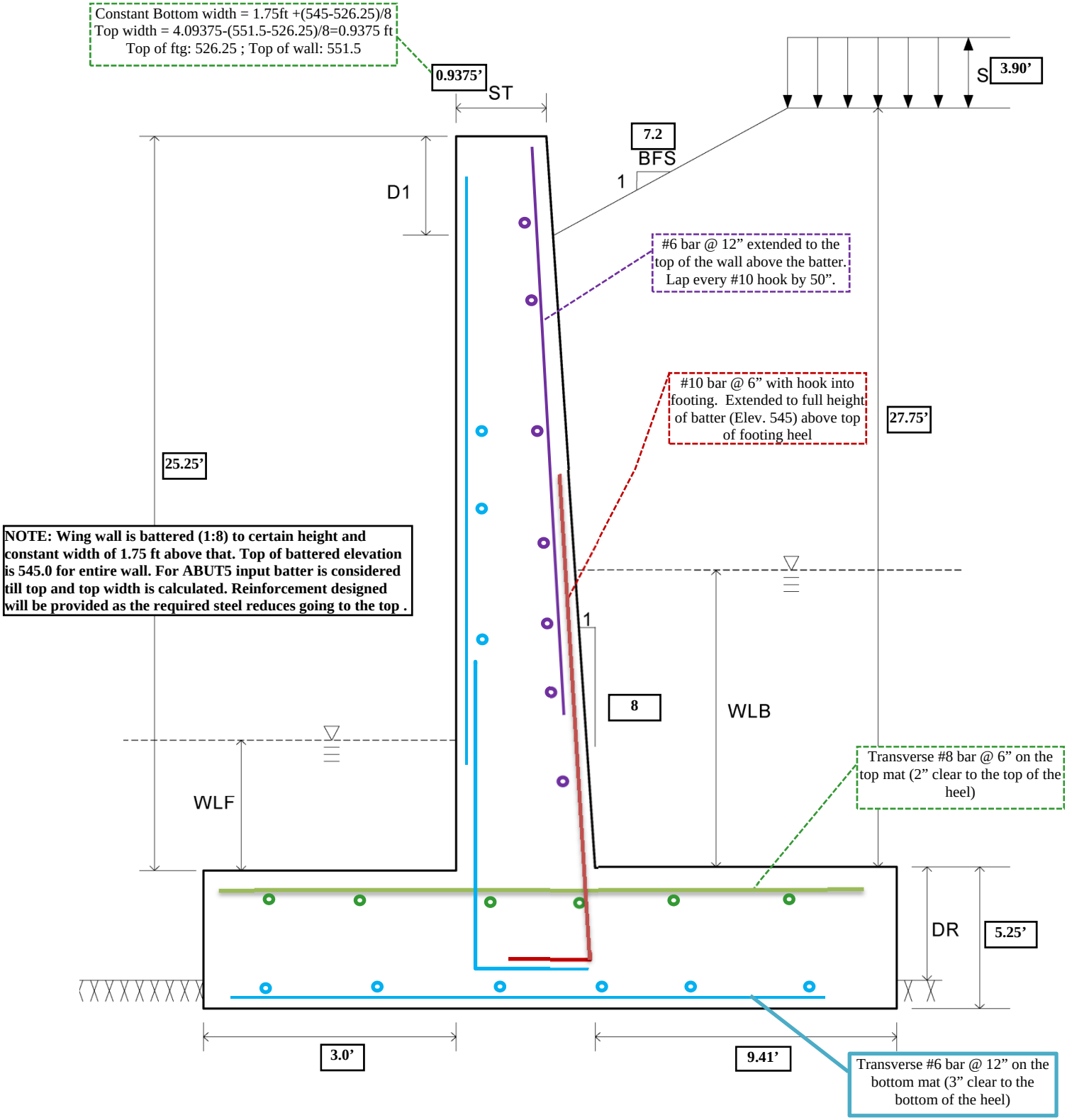
STEM:

```
Design #  1:
============
The theoretical cutoff for area of steel = 2.000
sq.in.
using a linear interpolation between design
sections
is  5.09 feet from the top of the footing.

Reqd. extension of reinf. per AASHTO 8.24.1.2.1 :
  1.  15 bar diameters  = 16.920 inches
  2.  effective depth   = 41.470 inches
  3.  1 ft. minimum

      USE               = 41.470 inches


Use # 9 bars at  6. in. up to a height of  8.55
ft.
Above this height use # 9 bars at 12. in.



Design #  2:
============
The theoretical cutoff for area of steel = 1.760
sq.in.
using a linear interpolation between design
sections
is  6.84 feet from the top of the footing.

Reqd. extension of reinf. per AASHTO 8.24.1.2.1 :
  1.  15 bar diameters  = 11.250 inches
  2.  effective depth   = 41.470 inches
  3.  1 ft. minimum

      USE               = 41.470 inches


Use # 6 bars at  3. in. up to a height of 10.29
ft.
Above this height use # 6 bars at  6. in.


STEEL DESIGNS LISTED FROM MOST DESIRABLE TO LEAST
OPTIMAL
```

Proposed Primary Stem Reinforcing

Rear Mat Vertical Bars:
#9's @ 6" hooked into foundation and extending to the top of batter (elevation 545) above top of footing.

#6's @ 12" extend to the top of the wall above the batter lapping the #9 bars. Lap should be 50 in . These reinforcing steel are used to match the wingwall section designed close to abutment (File WW3) for detailing and construction ease.



NOTE: Heights "H" are increased (approximately 6") in this analysis to ensure conservative design will still apply subject to any potential minor elevation changes during final detailing.

Reinforcing Note: All other bars shall be #5's at 12". 3" Clear to bottom of footing. 2" clear to all other surfaces. Vertical stem rebar should be outside of horizontal rebar. Transverse footing rebars should be on the top/bottom of longitudinal rebar.

```
LFD Abutment and Retaining Wall                    333067

PROGRAM P4354040                                          06/03/2014 09:12
VERSION 5.4.0.0           LAST UPDATED 05/05/2005        DOCUMENTATION 05/2005

INPUT: WW_SE3.dat

**PROJECT: ODOT/EMH&T: HAM-75-7.85, GF JOB # 054682
**NSRR OVER PROSSER: SE WINGWALL AT LAST PILE ROW BEFORE STEP;
STATION 158+33.38 OFFSET -17.13

                              ANALYSIS OF
                             RETAINING WALL

                        TYPE OF FOOTING : ON PILES

                                                                  EQUIV FLUID
DESIGN                    FTG    PILE            PILE              PRESSURE
METHOD   A OR D   TYPE   TYPE     D   EMBEDDED  ROWS   R OR S     DRY   SAT
  SL       A       3      2     12.0     Y       4               46.4

                        ALLOW SOIL   WATER  TOP FTG   WATER    TOP WALL   LIVE
COEFF OF  BACKFILL  PRESS OR AXIAL LEVEL  TO TOP    LEVEL     TO TOP    LOAD
FRICTION    SLOPE     PILE CAPAC    BACK  EMBANK    FRONT    BACKFILL   SURCH
             7.2        190.00             27.75                        3.90

TOP FTG  F'C                                   LATERAL                     PILE
TO ROCK  BACK   F'C    F'C  REBAR   PILE  PILE  PILE      REBAR  OVR  80%   ROW
SURFACE  WALL   STEM   FTG  GRADE  BATTER  OPT  CAPAC  Kv  DES   STR  RULE  OPT
        4000.  4000.  4000.   60.                0.1   6.4  Y     Y

  TOP OF
(BACK)WALL                                          MAX
TO TOP OF     TOP      PROJECTION          MAX    FTG
 FOOTING   THICKNESS   TOE   HEEL  T OR H  PROJ  WIDTH   H1     H2    H3    H4
  25.25      0.94      3.00  9.41

                             FRONT           HEIGHT
                             FACE   BRIDGE     OF
                             TO DL   SEAT   BACKWALL  BACK   FOOTING  PILE  FTG
 W1   W2   W3   BW1   BW2   REACT   WIDTH   BATTER  BATTER THICKNESS COST  COST
                                                        8.    5.25

     TRIAL
 REBAR SPACING  BAR   BW    STEM    FTG    MIN
 1  2  3  4  5  LOC  EXPOS  EXPOS  EXPOS   As
 3  6  8  9 12   Y     S      S      S

     STEM REBAR COVER           FOOTING REBAR COVER
 VERT  HORIZ   VERT  HORIZ    LONG  TRANS   LONG  TRANS
 2.00   3.00   2.00   3.00    3.00   2.00   4.00   3.00

                                          LONG
                      WIND   WIND         FORCE
 DL     LL    WIND    ON     ON   UPWARD  FROM   CENTR  TEMP
REACT  REACT  ON LL  SUPER   SUB   WIND    LL    FORCE  FORCE


                              BACKWALL            ALLOW
    PARAPET OR EXTERNAL       LIVE LOAD  SEISMIC   PILE
 HORZ   DIST   VERT   DIST   VERT   HORZ   LOAD   UPLIFT
                                                  19.00
             DISTANCE           PERCENT
ROW   PILE   BETWEEN    PILE      ROW
NO   BATTER    ROWS    SPACING  BATTERED
 1     3.0      1.50     3.00    100.0
 2     3.0      3.00     3.00    100.0
 3              3.00     3.00
 4              6.00     6.00
```

```
                    REINFORCEMENT SUMMARY
                    ---------------------
                     REBAR SPACINGS (in):
                              3.
                              6.
                              8.
                              9.
                             12.

          REBAR COVERS (in):   (C.G. OF BAR TO OUTER FACE)
                  STEM                      FOOTING
          BACK          FRONT          TOP         BOTTOM
       VERT  HORIZ   VERT  HORIZ  LONG   TRANS  LONG   TRANS
       2.00   3.00   2.00   3.00  3.00    2.00  4.00    3.00

                     BACK WALL -- SEVERE
       EXPOSURES:    STEM ------- SEVERE
                     FOOTING ---- SEVERE

       MINIMUM AREA OF STEEL PER FOOT: 0.125 SQUARE INCHES


***  NOTE  ***
 - THE REBARS IN THE BOTTOM OF THE FOOTING ARE BELOW THE TOP OF THE PILES.


                        PILE FOOTING ANALYSIS
                        ---------------------

***  NOTE  ***
 - FOR THE TEMPORARY CONSTRUCTION LOADING CONDITION (GROUP T) - FILL HEIGHT
   TO ABUTMENT SEAT, LIVE LOAD SURCHARGE, BUOYANCY AND WIND ON SUBSTRUCTURE
   (IF APPLICABLE):
    > PILE CAPACITY HAS BEEN INCREASED BY A FACTOR OF 1.25.

 - FOR PILE ROWS WITH BOTH VERTICAL AND BATTERED PILES, PILE LOADS ARE
   FOR THE MAXIMUM CONDITION (AXIAL LOAD IN BATTERED PILES).
```

HP12x53 Piles with ODOT allowable 95T per pile
HP piles are chosen for their higher capacity and lower displacement. Lower displacement is desirable due to the proximity of underground utilities and residential foundations.

STABILITY ANALYSIS (SERVICE LOADS) - WITH VERTICAL COMPONENT OF LL SURCHARGE

AASHTO GROUP	SUM V	BAL. MOMENT	SUM H	O.T. MOMENT	FRONT ROW TO RESULT	PILE LOAD FRONT	PILE LOAD BACK	PILE LOAD ALLOW	LAT. RESIST	F.S. O.T.
1	**66.33**	**546.13**	**29.96**	**353.33**	**2.91**	**81.59**	**16.70**	**190.00**	**30.72**	**1.55**
2	**60.45**	**494.87**	**24.12**	**259.26**	**3.90**	**59.30**	**41.52**	**237.50**	**38.40**	**1.91**
3	66.33	546.13	29.96	353.33	2.91	81.59	16.70	237.50	38.40	1.55
4	66.33	546.13	29.96	353.33	2.91	81.59	16.70	237.50	38.40	1.55
5	60.45	494.87	24.12	259.26	3.90	59.30	41.52	266.00	43.01	1.91
6	66.33	546.13	29.96	353.33	2.91	81.59	16.70	266.00	43.01	1.55
T	65.01	531.91	29.96	353.33	2.75	82.59	11.81	237.50	38.40	1.51

NOTE: Max Pile Reaction under Primary Loads is 82 Kips. It is 60 kips when longitudinal live load is added.

STABILITY ANALYSIS (SERVICE LOADS) - WITHOUT VERTICAL COMPONENT OF LL SURCHARGE

AASHTO GROUP	SUM V	BAL. MOMENT	SUM H	O.T. MOMENT	FRONT ROW TO RESULT	PILE LOAD FRONT	PILE LOAD BACK	PILE LOAD ALLOW	LAT. RESIST	F.S. O.T.
1	60.45	494.87	29.96	353.33	2.34	82.95	0.22	190.00	30.72	1.40
2	60.45	494.87	24.12	259.26	3.90	59.30	41.52	237.50	38.40	1.91
3	60.45	494.87	29.96	353.33	2.34	82.95	0.22	237.50	38.40	1.40
4	60.45	494.87	29.96	353.33	2.34	82.95	0.22	237.50	38.40	1.40
5	60.45	494.87	24.12	259.26	3.90	59.30	41.52	266.00	43.01	1.91
6	60.45	494.87	29.96	353.33	2.34	82.95	0.22	266.00	43.01	1.40
T	59.13	480.66	29.96	353.33	2.15	83.95	-4.68	237.50	38.40	1.36

FOOTING ANALYSIS (SERVICE LOADS) - WITH VERTICAL COMPONENT OF LL SURCHARGE

AASHTO GROUP	TOE MOMENT	TOE SHEAR @	HEEL MOMENT	HEEL SHEAR @
1	36.03	0.00 D	184.26	25.60 F
2	25.22	0.00 D	136.86	16.58 F
3	36.03	0.00 D	184.26	25.60 F
4	36.03	0.00 D	184.26	25.60 F
5	25.22	0.00 D	136.86	16.58 F
6	36.03	0.00 D	184.26	25.60 F
T	36.52	0.00 D	189.75	27.00 F

FOOTING ANALYSIS (SERVICE LOADS) - WITHOUT VERTICAL COMPONENT OF LL SURCHARGE

AASHTO GROUP	TOE MOMENT	TOE SHEAR @	HEEL MOMENT	HEEL SHEAR @
1	36.69	0.00 D	182.18	26.23 F
2	25.22	0.00 D	136.86	16.58 F
3	36.69	0.00 D	182.18	26.23 F
4	36.69	0.00 D	182.18	26.23 F
5	25.22	0.00 D	136.86	16.58 F
6	36.69	0.00 D	182.18	26.23 F
T	37.18	0.00 D	187.66	27.63 F

FOOTING DESIGN - WITH AND WITHOUT VERTICAL COMPONENT OF LL SURCHARGE

FOOTING WIDTH	FOOTING THICKNESS	TOE PROJ	HEEL PROJ	EFFECTIVE DEPTH TOE	EFFECTIVE DEPTH HEEL
16.50	5.25	3.00	9.40	5.000	5.083

SHEAR STR TOE	ALLOWABLE SHEAR TOE	SHEAR STR HEEL	ALLOWABLE SHEAR HEEL	TRANS REINFORCEMENT TOE-BOT	TRANS REINFORCEMENT HEEL-TOP	LONGITUDINAL REINFORCEMENT
0.000	0.060	0.037	0.060	0.41	1.60	0.12

LONGITUDINAL MOMENT CALCULATIONS - WITH VERTICAL COMPONENT OF LL SURCHARGE
GROUP 1 GOVERNS

MAX UNIFORM LOAD (w) = 4.020 K/FT
MAX UNIFORM SERVICE LOAD (w) = 4.020 K/FT
MAX PILE SPACING = 6.00 FEET
MAX LONGITUDINAL MOMENT = 7.24 K-FT
MAX LONG. MOMENT (SERVICE) = 7.24 K-FT

PILE PATTERN

ROW	BATTER	DISTANCE	SPACING	% BATTERED
1	3.0 ON 12	1.50	3.00	100.0
2	3.0 ON 12	4.50	3.00	100.0
3	VERT	7.50	3.00	0.0
4	VERT	13.50	6.00	0.0

C.G. OF PILES 5.79 FT. FROM TOE I OF PILES PER FOOT 17.571
PILE DENSITY 1.1667 PILES PER FOOT

STEM ANALYSIS

DESIGN OF STEM SECTION AT 6.31 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	T
MOMENT	9.96	4.09	9.96	9.96	4.09	9.96	5.60
AXIAL FORCE	1.57	1.57	1.57	1.57	1.57	1.57	1.56
SHEAR	2.96	1.51	2.96	2.96	1.51	2.96	2.07

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	BACK FACE REINF
1.729	0.013	0.060	0.325

DESIGN OF STEM SECTION AT 12.62 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	T
MOMENT	41.83	23.15	41.83	41.83	23.15	41.83	30.20
AXIAL FORCE	4.49	4.49	4.49	4.49	4.49	4.49	4.47
SHEAR	7.39	4.79	7.39	7.39	4.79	7.39	5.98

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	BACK FACE REINF
2.518	0.022	0.060	0.695

DESIGN OF STEM SECTION AT 18.94 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	T
MOMENT	107.66	68.95	107.66	107.66	68.95	107.66	85.55
AXIAL FORCE	8.77	8.77	8.77	8.77	8.77	8.77	8.72
SHEAR	13.67	9.92	13.67	13.67	9.92	13.67	11.75

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	BACK FACE REINF
3.307	0.030	0.060	1.362

DESIGN OF STEM SECTION AT 25.25 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	T
MOMENT	219.16	153.23	219.16	219.16	153.23	219.16	183.37
AXIAL FORCE	14.40	14.40	14.40	14.40	14.40	14.40	14.32
SHEAR	21.79	16.91	21.79	21.79	16.91	21.79	19.36

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	BACK FACE REINF
4.096	0.039	0.060	2.245

SUMMARY OF STEEL DESIGN

FOOTING DESIGN

Top longitudinal reinforcement:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
3	5	1.240
6	5	0.620
8	5	0.465
9	5	0.413
12	5	0.310

Bottom longitudinal reinforcement:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
3	5	1.240
6	5	0.620
8	5	0.465
9	5	0.413
12	5	0.310

Top transverse reinforcement:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
3	6	1.760
6	8	1.580 *
8	10	1.905
9	10	1.693
12 ** NO AVAILABLE DESIGN FOR THIS SPACING		

Bottom transverse reinforcement:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
3	5	1.240
6	5	0.620
8	5	0.465
9	5	0.413
12	6	0.440

STEM DESIGN

Vertical reinforcement on back face:

Vertical reinforcement on back face:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
At Section #3, 6.31 feet from the top:		
3	4	0.800
6	4	0.400
8	5	0.465
9	5	0.413
12	6	0.440
At Section #4, 12.62 feet from the top:		
3	4	0.800
6	6	0.880
8	7	0.900
9	7	0.800
12	8	0.790
At Section #5, 18.94 feet from the top:		
3	6	1.760
6	8	1.580
8	9	1.500
9	10	1.693
12	11	1.560
At Section #6, 25.25 feet from the top:		
3	7	2.400
6	10	2.540
8	11	2.340
9 ** NO AVAILABLE DESIGN FOR THIS SPACING		
12 ** NO AVAILABLE DESIGN FOR THIS SPACING		

OPTIMUM REBAR DESIGN

FOOTING:

```
Top longitudinal reinforcement:
     Spacing       Rebar

       12.             5
        9.             5
        6.             5
        8.             5
        3.             5
Bottom longitudinal reinforcement:
     Spacing       Rebar

       12.             5
        9.             5
        6.             5
        8.             5
        3.             5
  Top transverse reinforcement:
     Spacing       Rebar

        6.             8
        9.            10
        3.             6
        8.            10
 Bottom transverse reinforcement:
     Spacing       Rebar

       12.             6
        9.             5
        6.             5
        8.             5
        3.             5
```

Proposed Footing Reinforcing

Top Mat:
#5's @ 12" Longitudinally
#8's @ 6" Laterally
Bottom Mat:
#5's @ 12" Longitudinally
#6's @ 12" Laterally

STEM:

```
Design #  1:
============
The theoretical cutoff for area of steel = 2.540
sq.in.
using a linear interpolation between design
sections
is  7.18 feet from the top of the footing.

Reqd. extension of reinf. per AASHTO 8.24.1.2.1 :
  1.  15 bar diameters  = 19.050 inches
  2.  effective depth   = 47.155 inches
  3.  1 ft. minimum

      USE               = 47.155 inches


Use #10 bars at  6. in. up to a height of 11.11
ft.
Above this height use #10 bars at 12. in.



Design #  2:
============
The theoretical cutoff for area of steel = 2.400
sq.in.
using a linear interpolation between design
sections
is  7.84 feet from the top of the footing.

Reqd. extension of reinf. per AASHTO 8.24.1.2.1 :
  1.  15 bar diameters  = 13.125 inches
  2.  effective depth   = 47.155 inches
  3.  1 ft. minimum

      USE               = 47.155 inches


Use # 7 bars at  3. in. up to a height of 11.77
ft.
Above this height use # 7 bars at  6. in.


STEEL DESIGNS LISTED FROM MOST DESIRABLE TO LEAST
OPTIMAL
```

Proposed Primary Stem Reinforcing

Rear Mat Vertical Bars:
#10's @ 6" hooked into foundation and extending full height of batter above top of footing.

#6's @ 12" extend to the top of the wall above the batter.

DESIGN CALCULATION COVER SHEET

Gannett Fleming
Engineers and Architects, P.C.
Excellence Delivered ***As Promised***

ISO 9001:2008 CERTIFIED

PROJECT: **ODOT/EMH&T: HAM-75-7.85**
BRIDGE NO: **HAM-75-PROS**
SUBJECT: ***Analysis***
Southwest Wingwall- Permanent Caisson wall
FILE NAME(s): *HAM075_PROS_08P.xmcd*

ODOT PID NO: *77889*
EMH&T JOB NO:
GF JOB NO: 54682
CALC NO.: **HAM075_PROS_08P**
REV NO.: **01**
SHEET: 1 OF 8

SUPERSEDES CALC NO: 0 (Stage 1)

CALCULATION DESCRIPTION:
This will analyze the proposed permanent southwest caisson wingwall.

DESIGN BASIS OR REFERENCES:
NS Underpass Design Criteria
AREMA (2008) Chapter 8, Section 5
Input Load Calculation (HAM075_PROS_06)

MISCELLANEOUS NOTES:

RECORD OF ISSUES:

REV	DESCRIPTION	BY	DATE	VERIF'D	DATE
00	Stage 1 Calculation	EFD	01/23/12		
01	Stage 2 Calculation	VDT	06/04/14	CTM	06/16/14

RECORD OF CHECK OF CURRENT CALCULATION VERSION

BY	DATE	CHKD	DATE	UPD.	DATE	VERIF'D	DATE
VDT	06/04/14	CTM	06/12/14	VDT	06/16/14	CTM	06/16/14

Caisson Wall Design
(AREMA- ALLOWABLE STRESS DESIGN)

Design notes

- Variables with a box around them require review/user input.
- Code references are made using the following abbreviations:

  - AREMA - American Railway Engineering and Maintenance-of-Way Association
  - SEF - Structural Engineering Formulas (Ilya Mikhelson)
  - AISC - AISC Steel Construction Manual, Thirteenth Edition
  - GB6 - ODOT Office of Geotechnical Engineering: Bulletin 6
  - CERM - Civil Engineering Reference Manual

- General Design data and assumptions:
  1. Concrete caisson wall with reinforcing steel
  2. Calculation based on SEF 11.2 & Boussinesq Equation Spreadsheet for Live Load Surcharge Calculation

1) GENERAL INPUTS

1.1) Dimensions and Elevations

Retained Soil Elevation: $TopSoil := 548 \cdot ft$

Bottom of excavation elevation: $BotExc := 536 \cdot ft$

Bottom of Caisson Elevation: $Emb := 501 \cdot ft$

Retained height of soil: $H_{Ret} := TopSoil - BotExc$

$H_{Ret} = 12\,ft$

Effective Embedment below Excav.: $D_{emb} := BotExc - Emb - 6in$

$D_{emb} = 34.5\,ft$

Spacing: Center-Center of Caissons: $Cais_{Spa} := 3.5ft$

Caisson Diameter: $Cais_{Dia} := 42in$

Concrete Strength: $f_c := 4000psi$

Reinforcing Steel Used: $A_s := 16.0in^2$

16 - #9s

1.2) Soil Parameter inputs

$\beta := \operatorname{atan}\left(\frac{1}{2.12}\right)$ (Input - 2:1 backslope taken perpendicular to shoring)

Fill Angle: $\beta = 25.25 \cdot deg$

Wall Angle: $\theta := 90deg$

Earth Surcharge Load factor (ES): $ES := 1.0$

AREMA Allowable Stress, Passive resistance is reduced in subsequent pages

Taken from Retaining Wall Loads spreadsheet

Passive Resistance Safety Factor: $\Phi_P := 0.66$

AREMA 8-28.5.1.2 Reduction

Table 8-5-1. Types of Backfill for Retaining Walls

Backfill Type	Backfill Description
1	Coarse-grained soil without admixture of fine soil particles, very free-draining (sand, gravel or broken stone).
2	Coarse-grained soil of low permeability due to admixture of particles of silt size.
3	Fine silty sand; granular materials with conspicuous clay content; or residual soil with stones.
4	Soft or very soft clay, organic silt; or soft silty clay.
5	Medium or stiff clay that may be placed in such a way that a negligible amount of water will enter the spaces between the chunks during floods or heavy rains.

c. Types 4 and 5 backfill shall be used only with the permission of the Engineer. In all cases the wall design shall be based on the type of backfill used.

Table 8-5-2. Properties of Backfill Materials

Type of Backfill	Unit Weight Lb. Per Cu. Ft.	Cohesion "c"	Angle of Internal Friction
1	105	0	33° 42′ (38° for broken stone)
2	110	0	30°
3	125	0	28°
4	100	0	0°
5	120	240	0°

Specify for Backfill Type 1, but use the average of Type 1 and Type 2 to be conservative

	Backfill	Embedment
Unit weight of soil:	$\gamma_{bk} := 110pcf$	$\gamma_{emb} := 125pcf$
Effective Angle of Internal Friction:	$\phi_{bk} := 31deg$	$\phi_{emb} := 31deg$
Friction Angle Between Fill & Wall:	$\delta_{bk} := 15.5deg$	$\delta_{emb} := 15.5deg$

assumed to be 1/2 of internal friction angle

Embedded soil parameters are from Geotechnical Report

1.3) Coefficient of active earth pressure: Coulomb Approach is utilized, AREMA 8-20.3.1

Backfill

$$\Gamma_{abk} := \left[1 + \left(\frac{\sin(\phi_{bk} + \delta_{bk})\cdot\sin(\phi_{bk} - \beta)}{\sin(\theta - \delta_{bk})\sin(\theta + \beta)}\right)^{0.5}\right]^2$$

$$\Gamma_{abk} = 1.661$$

$$K_{a.bk} := \frac{\sin(\theta + \phi_{bk})^2}{\Gamma_{abk}\cdot\left(\sin(\theta)^2\cdot\sin(\theta - \delta_{bk})\right)}$$

$$K_{a.bk} = 0.459$$

Embedment

$$\Gamma_{emb} := \left[1 + \left(\frac{\sin(\phi_{emb} + \delta_{emb})\cdot\sin(\phi_{emb})}{\sin(\theta - \delta_{emb})\sin(\theta)}\right)^{0.5}\right]^2$$

$$\Gamma_{emb} = 2.633$$

$$K_{a.emb} := \frac{\sin(\theta + \phi_{emb})^2}{\Gamma_{emb}\cdot\left(\sin(\theta)^2\cdot\sin(\theta - \delta_{emb})\right)}$$

$$K_{a.emb} = 0.290$$

1.4) Coefficient of passive earth pressure: Coulomb Approach is utilized, AREMA 8-20.3.4

Backfill

$$\Gamma_{pbk} := \left[1 - \left(\frac{\sin(\phi_{bk} + \delta_{bk})\cdot\sin(\phi_{bk} + \beta)}{\sin(\theta + \delta_{bk})\sin(\theta + \beta)}\right)^{0.5}\right]^2$$

$$\Gamma_{pbk} = 0.028$$

$$K_{p.bk} := \frac{\sin(\theta - \phi_{bk})^2\cdot\Phi_P}{\Gamma_{pbk}\cdot\left(\sin(\theta)^2\cdot\sin(\theta + \delta_{bk})\right)}$$

$$K_{p.bk} = 17.808$$

Embedment

$$\Gamma_{embp} := \left[1 - \left(\frac{\sin(\phi_{emb} + \delta_{emb})\cdot\sin(\phi_{emb})}{\sin(\theta + \delta_{emb})\sin(\theta)}\right)^{0.5}\right]^2$$

$$\Gamma_{embp} = 0.142$$

$$K_{p.emb} := \frac{\sin(\theta - \phi_{emb})^2\cdot\Phi_P}{\Gamma_{embp}\cdot\left(\sin(\theta)^2\cdot\sin(\theta + \delta_{emb})\right)}$$

$$K_{p.emb} = 3.534$$

1.5) Concrete Properties:

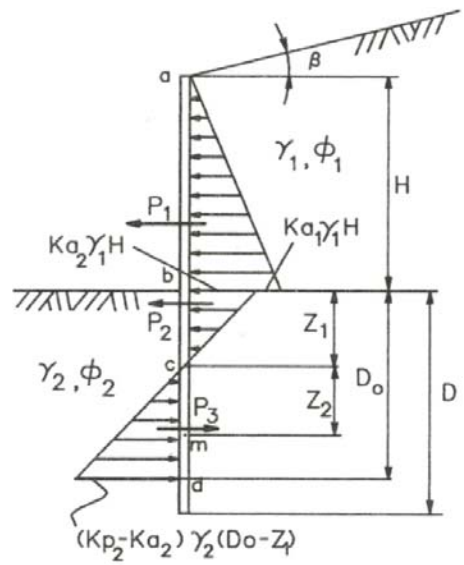
$$A_{cg} := \pi\left(\frac{Cais_{Dia}}{2}\right)^2 = 1385.44\cdot in^2 \qquad A_{ce} := \pi\left(\frac{Cais_{Dia}}{2} - 3in\right)^2 = 1017.88\cdot in^2$$

$$I_c := \frac{\pi}{4}\left(\frac{Cais_{Dia}}{2}\right)^4 = 152745.02\cdot in^4 \qquad S_x := \frac{\pi}{4}\left(\frac{Cais_{Dia}}{2}\right)^3 = 7273.57\cdot in^3$$

$$E_c := 57000\cdot\sqrt{f_c\cdot psi} = 3605\cdot ksi$$

1.6) Analysis Reference: SEF Figure 11.2 (2) - Cantilever Sheet Piling

2



Earth pressure:

$$P_1 = 0.5K_a\gamma H^2\,, \qquad z_1 = \frac{K_a\gamma_1 H}{(K_{p_2}-K_{a_2})\gamma_2}$$

$$P_2 = 0.5K_{a_2}\gamma_1 H\cdot z_1\,, \qquad z_2 = \sqrt{\frac{P_1+P_2}{0.5(K_{p_2}-K_{a_2})\gamma_2}}$$

$$P_3 = 0.5(K_{p_2}-K_{a_2})\gamma_2(D_0-z_1)^2$$

Equation to determine D_0: $\sum M_d = 0$

$$P_1\left(\frac{H}{3}+D_0\right)+P_2\left(D_0-\frac{z_1}{3}\right)-P_3\frac{1}{3}(D_0-z_1)=0$$

$D = (1.2 \text{ to } 1.4)D_0$ for factor of safety at 1.5 to 2.0

m = point of zero shear and maximum bending moment

Maximum bending moment

$$M_{max} = P_1\left(\frac{H}{3}+z_1+z_2\right)+P_2\left(\frac{2}{3}z_1+z_2\right) -0.5(K_p-K_a)\gamma z_2^2\cdot\left(\frac{z_2}{3}\right)$$

2) Caisson Wall Embedment Check

Earth pressure due to Active soil

$$P_1 := \left[0.5\left(\gamma_{bk}\cdot K_{a.bk}\cdot H_{Ret}^2\right)\right]\cdot Cais_{Spa}$$

$P_1 = 12.73\cdot kip$

Values form Ret. Wall Spreadsheet

Live load surcharge

$P_{LLS1} := 5.02\cdot klf\cdot Cais_{Spa}$ $\qquad$ $P_{LLS2} := 4.93\cdot klf$

$H_{LLS1} := 6.64\cdot ft$ $\qquad$ $H_{LLS2} := -7.15\cdot ft$

from bottom of exposed wall

Earth pressure due to Passive soil

z1 depth uses load and resistance factors; conservative for deflection (service) calculations

$$z_1 := \frac{\left[ES\cdot\left(K_{a.bk}\cdot\gamma_{bk}\cdot H_{Ret}\right)+\frac{P_{LLS1}}{H_{Ret}\cdot Cais_{Spa}}\right]\cdot Cais_{Spa}}{\left(K_{p.emb}-ES\cdot K_{a.emb}\right)\gamma_{emb}\cdot Cais_{Dia}-\frac{P_{LLS2}}{D_{emb}}} = 2.81\cdot ft$$

$$P_2 := \left(0.5\cdot K_{a.emb}\cdot\gamma_{bk}\cdot H_{Ret}\cdot z_1\right)\cdot Cais_{Dia} = 1.88\cdot kip$$

Passive Resistance

$$P_3 := \left[0.5\cdot\left(K_{p.emb}-K_{a.emb}\right)\cdot\gamma_{emb}\cdot\left(D_{emb}-z_1\right)^2\right]\cdot Cais_{Dia} = 712.83\cdot kip$$

Negative Moment mean that the passive (P3) force will tend to rotate the wall towards the filled material.
Positive moments indicate that there is insufficient resisting force.

$$M_D := ES\cdot P_1\cdot\left(D_{emb}+\frac{H_{Ret}}{3}\right)+ES\cdot P_2\cdot\left(D_{emb}-\frac{z_1}{3}\right)+ES\cdot P_{LLS1}\cdot\left(D_{emb}+H_{LLS1}\right)\ \cdots$$
$$+\ ES\cdot P_{LLS2}\cdot\left(D_{emb}+H_{LLS2}\right)\cdot Cais_{Dia}-P_3\cdot\frac{\left(D_{emb}-z_1\right)}{3}$$

$M_D = -5782.43\ ft\cdot kip$

$EMBEDMENT_CHECK := if\left(M_D < 0, \text{"OK"}, \text{"INCREASE EMBEDMENT"}\right)$ $\quad$ $EMBEDMENT_CHECK = \text{"OK"}$

3) Caisson Wall Design (See graphic in section 1.6)

3.1) Required Section Modulus

Zero shear location = m = Point of Max Bending Moment:

$$z_2 := \left[\frac{ES\cdot\left(P_1 + P_2 + P_{LLS1}\right)}{0.5\left(K_{p.emb} - ES\cdot K_{a.emb}\right)\gamma_{emb}\cdot Cais_{Dia} - \frac{P_{LLS2}}{D_{emb}}}\right]^{0.5} \qquad z_2 = 7.534\,ft \qquad z_m := z_1 + z_2 \qquad z_m = 10.343\,ft$$

Moment at zero shear location

$$M_{m.fact} := \left[ES\cdot P_1\cdot\left(z_m + \frac{H_{Ret}}{3}\right) + ES\cdot P_2\cdot\left(\frac{2\cdot z_1}{3} + z_2\right)\right] \ldots$$
$$+ \left[ES\cdot P_{LLS1}\cdot\left(z_m + H_{LLS1}\right) \ldots + ES\cdot P_{LLS2}\cdot Cais_{Dia}\cdot\left(z_m + H_{LLS2}\right) - \left[0.5\left(K_{p.emb} - ES\cdot K_{a.emb}\right)\cdot\gamma_{emb}\cdot\left(z_2\right)^2\cdot\frac{z_2}{3}\right]\cdot Cais_{Dia}\right]$$

$$M_{m.fact} = 452.5\,ft\cdot kip$$

Required section modulus of steel pile section: $S_{x.req} := \frac{M_{m.fact}}{.40\cdot f_c}$ $\qquad S_{x.req} = 3393.79\cdot in^3$

$$S_x = 7273.57\cdot in^3$$

$$SECTION_CHECK := if\left(S_x > S_{x.req}, \text{"OK"}, \text{"INCREASE SECTION"}\right) \qquad SECTION_CHECK = \text{"OK"}$$

3.2) Calculated Deflection (Unfactored)

Deflection will be calculated assuming the post is fixed at the zero shear (max moment) depth.

The deflections will be calculated by approximating the deflection by each equivalent point load (service) as follows:

Active Deflection
$$\Delta_{p.1} := \frac{\left[P_1\cdot\left(z_m + \frac{H_{Ret}}{3}\right)^2\right]}{\left(6\cdot E_c\cdot I_c\right)}\cdot\left[3\cdot\left(z_m + H_{Ret}\right) - \left(z_m + \frac{H_{Ret}}{3}\right)\right] = 0.07\cdot in$$

Surcharge Deflection
$$\Delta_{p.LLS} := \frac{\left[P_{LLS1}\cdot\left(z_m + H_{LLS1}\right)^2\right]}{\left(6\cdot E_c\cdot I_c\right)}\cdot\left[3\cdot\left(z_m + H_{Ret}\right) - \left(z_m + H_{LLS1}\right)\right] = 0.13\cdot in$$

Embedded Active Deflection
$$\Delta_{p.2} := \frac{\left[P_2\cdot\left(\frac{2\cdot z_1}{3} + z_2\right)^2\right]}{\left(6\cdot E_c\cdot I_c\right)}\cdot\left[3\cdot\left(z_m + H_{Ret}\right) - \left(\frac{2\cdot z_1}{3} + z_2\right)\right] = 0.01\cdot in$$

Note: P.LLS2 will not be included in the surcharge deflection calculation. The surcharge load on the embedded portion of the caisson will be counter-acted by the passive resistance.

Deflection at the top of the post: $\Delta_{Top} := \Delta_{p.1} + \Delta_{p.LLS} + \Delta_{p.2}$ $\qquad \Delta_{Top} = 0.21\cdot in$

Deflection Ratio $\Delta_{Ratio} := \frac{H_{Ret}}{\Delta_{Top}}$ $\qquad \Delta_{Ratio} = 686.41$

4) Reinforcing Steel Design

4.1) Calculate Minimum Area of Flexural Steel (Unfactored):

The following values are calculated to find the required percentage of steel per chart in Civil Engineering Reference Manual

$A_g := A_{cg} = 1385.44 \cdot in^2$ $h := Cais_{Dia} = 42 \cdot in$ $P_n := 0$ No axial load, only lateral forces

AREMA requires min. flexural steel provides a design moment of 1.2 x cracking moment:

$f_r := 7.5 \cdot \sqrt{f_c \cdot psi} = 474.34 \cdot psi$ AREMA Ch.8 2.26.1a

$M_{cr} := \dfrac{(f_r \cdot I_c)}{\left(\dfrac{Cais_{Dia}}{2}\right)} = 287.51\, ft \cdot kip$ AREMA Ch.8 2.23.7c

Design for greater of the two:

$M_{eff} := \max(M_{m.fact}, 1.2 \cdot M_{cr}) = 452.5\, ft \cdot kip$

$X_{axis} := \dfrac{M_{m.fact}}{A_g \cdot h} = 0.09\, ksi$ $Y_{axis} := \dfrac{P_n}{A_g} = 0$

Based on the chart, input value for ρ steel:

$\boxed{\rho_s := .01}$ $As_{min} := A_g \cdot \rho_s = 13.85 \cdot in^2$ CERM - Appendix 52.A, p. A-95

STEEL_CHK := if(As_min < A_s, "OK" , "MORE STEEL REQ'D")

STEEL_CHK = "OK"

4.2) Calculate Shear Capacity (Unfactored):

The maximum shear will be at the ground line, or at the base of the exposed face of the wall

$V_{max} := P_1 + P_{LLS1} = 30.3 \cdot kip$

AREMA has a method to convert circular sections to square, use the less of their method and effective area calculated p.3

$b_w := Cais_{Dia}$ $d := .8 \cdot Cais_{Dia}$ $A_{arema} := b_w \cdot d = 1411.2 \cdot in^2$ AREMA - Ch.8 - 2.29.1a

$A_{eff} := \min(A_{arema}, A_{ce}) = 1017.88 \cdot in^2$

$M_v := P_1 \cdot \left(\dfrac{H_{Ret}}{3}\right) + P_{LLS1} \cdot H_{LLS1} = 167.57\, ft \cdot kip$

Max Shear assumed to be at the bottom of the exposed face of the wall, this calculates the Moment at that location

$v_{c_all} := .9 \cdot \sqrt{f_c \cdot psi} + 1100\left(\dfrac{A_s}{A_{cg}}\right) \cdot \dfrac{(V_{max} \cdot 0.8 \cdot Cais_{Dia}) \cdot psi}{M_v} = 0.06\, ksi$ AREMA - Ch.8 - 2.29.2b EQ 2-15

$V_{c_all} := v_{c_all} \cdot A_{ce} = 64.48 \cdot kip$

SHEAR_CHK := if(V_max < V_c_all, "OK" , "VERIFY SHEAR REINFORCING IS ADEQUATE") SHEAR_CHK = "OK"

4.3) Calculate Shear Reinforcing (Unfactored):

The concrete is capable resisting all of the shear force, but since the applied force is more than half of the concrete allowable shear (close), minimum shear steel is required per AREMA, Chapter 8, Section 2.10.1 a

Minimum Required Shear Area: $A_{vmin} := \frac{\left[60 \cdot \left(Cais_{Dia}\right) \cdot 4in\right]}{60000} = 0.17 \cdot in^2$ **AREMA - Ch.8 - 2.10.1b EQ 2-6**

Assumed 4" spacing, Grade 60 reinforcing

Provide Minimum Steel (#4 spirals @ 4" pitch): $A_v := .2in^2$ *Area of #4 bars*

$$SHEAR_STEEL_CHK := if\left(A_{vmin} < A_v, \text{"OK"}, \text{"NEED MORE STEEL"}\right) = \text{"OK"}$$

DESIGN CALCULATION COVER SHEET

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ISO 9001:2008 CERTIFIED

PROJECT: **ODOT/EMH&T: HAM-75-7.85**
BRIDGE NO: **HAM-75-PROS**
SUBJECT: ***Analysis***
SW Tangent Caisson Wall
MATHCAD Program
FILE NAME(s): *HAM075_PROS_08T.xmcd*

ODOT PID NO: *77889*
EMH&T JOB NO:
GF JOB NO: 54682
CALC NO.: **HAM075_PROS_08T**
REV NO.: **00**
SHEET: 1 OF 8

SUPERSEDES CALC NO: (N/A)

CALCULATION DESCRIPTION:

This will analyze the Tangent Caisson Wall used to replace the SW Wingwall. The reason for making this a tangent wall was due to the existing track's train surcharge, and because deflection during construction has to be minimized under 1/2" deflection because it is supporting the existing track.

DESIGN BASIS OR REFERENCES:

NS Underpass Design Criteria
AREMA (2008) Chapter 8, Section 5
CERM-Civil Engineering Reference Manual

Stage 1 Plans and Comments

MISCELLANEOUS NOTES:

A Coulomb approach was taken when calculating lateral soil pressures behind and in front of the wall.

RECORD OF ISSUES:

REV	DESCRIPTION	BY	DATE	VERIF'D	DATE
00	Stage 1 Calculation	VDT	06/03/14	CTM	06/16/14

RECORD OF CHECK OF CURRENT CALCULATION VERSION

BY	DATE	CHKD	DATE	UPD.	DATE	VERIF'D	DATE
VDT	05/19/14	CTM	06/12/14	VDT	06/16/14	CTM	06/16/14

Caisson Wall Design
(AREMA- ALLOWABLE STRESS DESIGN)

Design notes

- Variables with a box around them require review/user input.
- Code references are made using the following abbreviations:

  - AREMA - American Railway Engineering and Maintenance-of-Way Association
  - SEF - Structural Engineering Formulas (Ilya Mikhelson)
  - AISC - AISC Steel Construction Manual, Thirteenth Edition
  - GB6 - ODOT Office of Geotechnical Engineering: Bulletin 6
  - CERM - Civil Engineering Reference Manual

- General Design data and assumptions:
  1. Concrete caisson wall with reinforcing steel
  2. Calculation based on SEF 11.2 & Boussinesq Equation Spreadsheet for Live Load Surcharge Calculation

1) GENERAL INPUTS

1.1) Dimensions and Elevations

Retained Soil Elevation: $TopSoil := 537 \cdot ft$

Bottom of excavation elevation: $BotExc := 522 \cdot ft$

Bottom of Caisson Elevation: $Emb := 496 \cdot ft$

Retained height of soil: $H_{Ret} := TopSoil - BotExc$

$H_{Ret} = 15\,ft$

Effective Embedment below Excav.: $D_{emb} := BotExc - Emb - 1.5 \cdot Cais_{Dia}$

$D_{emb} = 20.75\,ft$ *Additional reduction term per AREMA 8-28.5.3.2 a*

Spacing: Center-Center of Caissons: $Cais_{Spa} := 3.5ft$

Caisson Diameter: $Cais_{Dia} := 42in$

Concrete Strength: $f_c := 4000psi$

Reinforcing Steel Used: $A_s := 16.0in^2$

16 - #9s

1.2) Soil Parameter inputs

$\beta := \operatorname{atan}\left(\frac{1}{2.71}\right)$ (Input - 2:1 backslope taken perpendicular to shoring)

Fill Angle: $\beta = 20.25 \cdot deg$

Wall Angle: $\theta := 90deg$

Earth Surcharge Load factor (ES): $ES := 1.0$

AREMA Allowable Stress, Passive resistance is reduced in subsequent pages

Taken from Retaining Wall Loads spreadsheet

Passive Resistance Safety Factor: $\Phi_P := 0.66$

AREMA 8-28.5.1.2 Reduction

	Backfill	Embedment
Unit weight of soil:	$\gamma_{bk} := 125pcf$	$\gamma_{emb} := 125pcf$
Effective Angle of Internal Friction:	$\phi_{bk} := 31deg$	$\phi_{emb} := 31deg$
Friction Angle Between Fill & Wall:	$\delta_{bk} := 15.5deg$	$\delta_{emb} := 15.5deg$

assumed to be 1/2 of internal friction angle

Soil Parameters shown are from the Geotechnical Report for this project.

1.3) Coefficient of active earth pressure: Coulomb Approach is utilized, AREMA 8-20.3.1

Backfill

$$\Gamma_{abk} := \left[1 + \left(\frac{\sin(\phi_{bk} + \delta_{bk})\cdot\sin(\phi_{bk} - \beta)}{\sin(\theta - \delta_{bk})\sin(\theta + \beta)}\right)^{0.5}\right]^2$$

$$\Gamma_{abk} = 1.923$$

$$K_{a.bk} := \frac{\sin(\theta + \phi_{bk})^2}{\Gamma_{abk}\cdot\left(\sin(\theta)^2\cdot\sin(\theta - \delta_{bk})\right)}$$

$$K_{a.bk} = 0.396$$

Embedment

$$\Gamma_{emb} := \left[1 + \left(\frac{\sin(\phi_{emb} + \delta_{emb})\cdot\sin(\phi_{emb})}{\sin(\theta - \delta_{emb})\sin(\theta)}\right)^{0.5}\right]^2$$

$$\Gamma_{emb} = 2.633$$

$$K_{a.emb} := \frac{\sin(\theta + \phi_{emb})^2}{\Gamma_{emb}\cdot\left(\sin(\theta)^2\cdot\sin(\theta - \delta_{emb})\right)}$$

$$K_{a.emb} = 0.290$$

1.4) Coefficient of passive earth pressure: Coulomb Approach is utilized, AREMA 8-20.3.4

Backfill

$$\Gamma_{pbk} := \left[1 - \left(\frac{\sin(\phi_{bk} + \delta_{bk})\cdot\sin(\phi_{bk} + \beta)}{\sin(\theta + \delta_{bk})\sin(\theta + \beta)}\right)^{0.5}\right]^2$$

$$\Gamma_{pbk} = 0.044$$

$$K_{p.bk} := \frac{\sin(\theta - \phi_{bk})^2\cdot\Phi_P}{\Gamma_{pbk}\cdot\left(\sin(\theta)^2\cdot\sin(\theta + \delta_{bk})\right)}$$

$$K_{p.bk} = 11.528$$

Embedment

$$\Gamma_{embp} := \left[1 - \left(\frac{\sin(\phi_{emb} + \delta_{emb})\cdot\sin(\phi_{emb})}{\sin(\theta + \delta_{emb})\sin(\theta)}\right)^{0.5}\right]^2$$

$$\Gamma_{embp} = 0.142$$

$$K_{p.emb} := \frac{\sin(\theta - \phi_{emb})^2\cdot\Phi_P}{\Gamma_{embp}\cdot\left(\sin(\theta)^2\cdot\sin(\theta + \delta_{emb})\right)}$$

$$K_{p.emb} = 3.534$$

1.5) Concrete Properties:

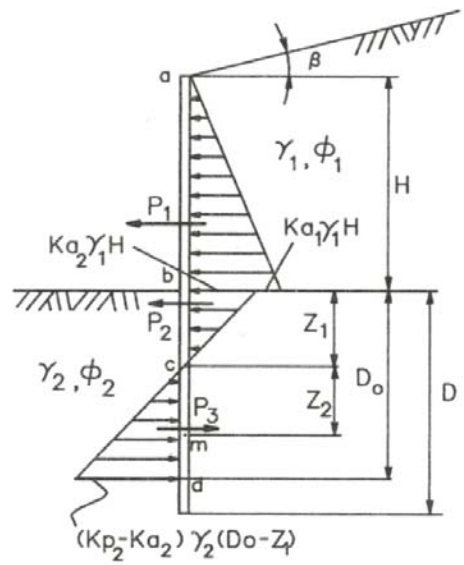
$$A_{cg} := \pi\left(\frac{Cais_{Dia}}{2}\right)^2 = 1385.44\cdot in^2 \qquad A_{ce} := \pi\left(\frac{Cais_{Dia}}{2} - 3in\right)^2 = 1017.88\cdot in^2$$

$$I_c := \frac{\pi}{4}\left(\frac{Cais_{Dia}}{2}\right)^4 = 152745.02\cdot in^4 \qquad S_x := \frac{\pi}{4}\left(\frac{Cais_{Dia}}{2}\right)^3 = 7273.57\cdot in^3$$

$$E_c := 57000\cdot\sqrt{f_c\cdot psi} = 3605\cdot ksi$$

1.6) Analysis Reference: SEF Figure 11.2 (2) - Cantilever Sheet Piling

2



Earth pressure:

$$P_1 = 0.5K_a\gamma H^2 , \qquad z_1 = \frac{K_a\gamma_1 H}{(K_{p_2} - K_{a_2})\gamma_2}$$

$$P_2 = 0.5K_{a_2}\gamma_1 H \cdot z_1 , \qquad z_2 = \sqrt{\frac{P_1 + P_2}{0.5(K_{p_2} - K_{a_2})\gamma_2}}$$

$$P_3 = 0.5(K_{p_2} - K_{a_2})\gamma_2 (D_0 - z_1)^2$$

Equation to determine D_0: $\sum M_d = 0$

$$P_1\left(\frac{H}{3} + D_0\right) + P_2\left(D_0 - \frac{z_1}{3}\right) - P_3\frac{1}{3}(D_0 - z_1) = 0$$

$D = (1.2 \text{ to } 1.4)D_0$ for factor of safety at 1.5 to 2.0

m = point of zero shear and maximum bending moment

Maximum bending moment

$$M_{max} = P_1\left(\frac{H}{3} + z_1 + z_2\right) + P_2\left(\frac{2}{3}z_1 + z_2\right) - 0.5(K_p - K_a)\gamma z_2^2 \cdot \left(\frac{z_2}{3}\right)$$

2) Caisson Wall Embedment Check

Earth pressure due to Active soil

$$P_1 := \left[0.5\left(\gamma_{bk} \cdot K_{a.bk} \cdot H_{Ret}^2\right)\right] \cdot Cais_{Spa}$$

$$P_1 = 19.51 \cdot kip$$

Live load surcharge

Values form Ret. Wall Spreadsheet

$P_{LLS1} := 4.97 \cdot klf \cdot Cais_{Spa}$ $\qquad$ $P_{LLS2} := 3.04klf$

$H_{LLS1} := 9.05 \cdot ft$ $\qquad$ $H_{LLS2} := -5.78 \cdot ft$

from bottom of exposed wall

Negative Moment mean that the passive (P3) force will tend to rotate the wall towards the filled material.
Positive moments indicate that there is insufficient resisting force.

Earth pressure due to Passive soil

z1 depth uses load and resistance factors; conservative for deflection (service) calculations

$$z_1 := \frac{\left[ES \cdot \left(K_{a.bk} \cdot \gamma_{bk} \cdot H_{Ret}\right) + \frac{P_{LLS1}}{H_{Ret} \cdot Cais_{Spa}}\right] \cdot Cais_{Spa}}{\left(K_{p.emb} - ES \cdot K_{a.emb}\right)\gamma_{emb} \cdot Cais_{Dia} - \frac{P_{LLS2}}{D_{emb}}} = 2.95 \cdot ft$$

$$P_2 := \left(0.5 \cdot K_{a.emb} \cdot \gamma_{bk} \cdot H_{Ret} \cdot z_1\right) \cdot Cais_{Dia} = 2.81 \cdot kip$$

Passive Resistance

$$P_3 := \left[0.5 \cdot \left(K_{p.emb} - K_{a.emb}\right) \cdot \gamma_{emb} \cdot \left(D_{emb} - z_1\right)^2\right] \cdot Cais_{Dia} = 224.75 \cdot kip$$

$$M_D := ES \cdot P_1 \cdot \left(D_{emb} + \frac{H_{Ret}}{3}\right) + ES \cdot P_2 \cdot \left(D_{emb} - \frac{z_1}{3}\right) + ES \cdot P_{LLS1} \cdot \left(D_{emb} + H_{LLS1}\right) \ldots$$
$$+ ES \cdot P_{LLS2} \cdot \left(D_{emb} + H_{LLS2}\right) \cdot Cais_{Dia} - P_3 \cdot \frac{\left(D_{emb} - z_1\right)}{3}$$

$M_D = -97.55 \text{ ft} \cdot \text{kip}$

$EMBEDMENT_CHECK := if\left(M_D < 0, \text{"OK"}, \text{"INCREASE EMBEDMENT"}\right)$ $\qquad$ $EMBEDMENT_CHECK = \text{"OK"}$

3) Caisson Wall Design (See graphic in section 1.6)

3.1) Required Section Modulus

Zero shear location = m = Point of Max Bending Moment:

$$z_2 := \left[\frac{ES\cdot\left(P_1 + P_2 + P_{LLS1}\right)}{0.5\left(K_{p.emb} - ES\cdot K_{a.emb}\right)\gamma_{emb}\cdot Cais_{Dia} - \frac{P_{LLS2}}{D_{emb}}}\right]^{0.5} \qquad z_2 = 8.397\,ft \qquad z_m := z_1 + z_2 \qquad z_m = 11.352\,ft$$

Moment at zero shear location

$$M_{m.fact} := \left[ES\cdot P_1\cdot\left(z_m + \frac{H_{Ret}}{3}\right) + ES\cdot P_2\cdot\left(\frac{2\cdot z_1}{3} + z_2\right)\right] \ldots + \left[\begin{array}{l} ES\cdot P_{LLS1}\cdot\left(z_m + H_{LLS1}\right) \ldots \\ + ES\cdot P_{LLS2}\cdot Cais_{Dia}\cdot\left(z_m + H_{LLS2}\right) - \left[0.5\left(K_{p.emb} - ES\cdot K_{a.emb}\right)\cdot\gamma_{emb}\cdot\left(z_2\right)^2\cdot\frac{z_2}{3}\right]\cdot Cais_{Dia} \end{array}\right]$$

$$M_{m.fact} = 622.29\,ft\cdot kip$$

Required section modulus of steel pile section: $S_{x.req} := \frac{M_{m.fact}}{.40\cdot f_c}$ $\qquad S_{x.req} = 4667.17\cdot in^3$

$$S_x = 7273.57\cdot in^3$$

$$SECTION_CHECK := if\left(S_x > S_{x.req}, \text{"OK"}, \text{"INCREASE SECTION"}\right) \qquad SECTION_CHECK = \text{"OK"}$$

3.2) Calculated Deflection (Unfactored)

Deflection will be calculated assuming the post is fixed at the zero shear (max moment) depth.

The deflections will be calculated by approximating the deflection by each equivalent point load (service) as follows:

Active Deflection

$$\Delta_{p.1} := \frac{\left[P_1\cdot\left(z_m + \frac{H_{Ret}}{3}\right)^2\right]}{\left(6\cdot E_c\cdot I_c\right)}\cdot\left[3\cdot\left(z_m + H_{Ret}\right) - \left(z_m + \frac{H_{Ret}}{3}\right)\right] = 0.17\cdot in$$

Surcharge Deflection

$$\Delta_{p.LLS} := \frac{\left[P_{LLS1}\cdot\left(z_m + H_{LLS1}\right)^2\right]}{\left(6\cdot E_c\cdot I_c\right)}\cdot\left[3\cdot\left(z_m + H_{Ret}\right) - \left(z_m + H_{LLS1}\right)\right] = 0.22\cdot in$$

Embedded Active Deflection

$$\Delta_{p.2} := \frac{\left[P_2\cdot\left(\frac{2\cdot z_1}{3} + z_2\right)^2\right]}{\left(6\cdot E_c\cdot I_c\right)}\cdot\left[3\cdot\left(z_m + H_{Ret}\right) - \left(\frac{2\cdot z_1}{3} + z_2\right)\right] = 0.01\cdot in$$

Note: P.LLS2 will not be included in the surcharge deflection calculation. The surcharge load on the embedded portion of the caisson will be counter-acted by the passive resistance.

Deflection at the top of the post: $\Delta_{Top} := \Delta_{p.1} + \Delta_{p.LLS} + \Delta_{p.2}$ $\qquad \Delta_{Top} = 0.4\cdot in$

Deflection Ratio $\Delta_{Ratio} := \frac{H_{Ret}}{\Delta_{Top}}$ $\qquad \Delta_{Ratio} = 445.45$

4) Reinforcing Steel Design

4.1) Calculate Minimum Area of Steel (Unfactored):

The following values are calculated to find the required percentage of steel per chart in Civil Engineering Reference Manual

$A_g := A_{cg} = 1385.44 \cdot in^2$ $h := Cais_{Dia} = 42 \cdot in$ $P_n := 0$ No axial load, only lateral forces

AREMA requires min. flexural steel provides a design moment of 1.2 x cracking moment:

$f_r := 7.5 \cdot \sqrt{f_c \cdot psi} = 474.34 \cdot psi$ AREMA Ch.8 2.26.1a

$M_{cr} := \dfrac{(f_r \cdot I_c)}{\left(\dfrac{Cais_{Dia}}{2}\right)} = 287.51\, ft \cdot kip$ AREMA Ch.8 2.23.7c

Design for greater of the two:

$M_{eff} := \max(M_{m.fact}, 1.2 \cdot M_{cr}) = 622.29\, ft \cdot kip$

$X_{axis} := \dfrac{M_{m.fact}}{A_g \cdot h} = 0.13\, ksi$ $Y_{axis} := \dfrac{P_n}{A_g} = 0$

Based on the chart, input value for ρ steel:

$\boxed{\rho_s := .01}$ $As_{min} := A_g \cdot \rho_s = 13.85 \cdot in^2$ CERM - Appendix 52.A, p. A-95

$STEEL_CHK := if(As_{min} < A_s, \text{"OK"}, \text{"MORE STEEL REQ'D"})$

STEEL_CHK = "OK"

4.2) Calculate Shear Capacity (Unfactored):

The maximum shear will be at the ground line, or at the base of the exposed face of the wall

$V_{max} := P_1 + P_{LLS1} = 36.91 \cdot kip$

AREMA has a method to convert circular sections to square, use the less of their method and effective area calculated p.3

$b_w := Cais_{Dia}$ $d := .8 \cdot Cais_{Dia}$ $A_{arema} := b_w \cdot d = 1411.2 \cdot in^2$ AREMA - Ch.8 - 2.29.1a

$A_{eff} := \min(A_{arema}, A_{ce}) = 1017.88 \cdot in^2$

$V_{all} := 0.95 \sqrt{f_c \cdot psi} \cdot A_{eff} = 61.16 \cdot kip$ AREMA - Ch.8 - 2.26.1b

$SHEAR_CHK := if(V_{max} < V_{all}, \text{"OK"}, \text{"NEED LARGER SECTION"})$

SHEAR_CHK = "OK"

Although there will be reinforcing in the Caisson Walls, checking the capacity of just the concrete is done to be conservative

4.3) Calculate Shear Reinforcing (Unfactored):

The concrete is capable resisting all of the shear force, but since the applied force is more than half of the concrete allowable shear, minimum shear steel is required per AREMA, Chapter 8, Section 2.10.1 a

Minimum Required Shear Area: $A_{vmin} := \frac{\left[60 \cdot \left(Cais_{Dia}\right) \cdot 4in\right]}{60000} = 0.17 \cdot in^2$

AREMA - Ch.8 - 2.10.1b EQ 2-6

Assumed 4" spacing, Grade 60 reinforcing

Provide Minimum Steel (#4 spirals @ 4" pitch): $\boxed{A_v := .2in^2}$ *Area of a #4 bar*

$SHEAR_STEEL_CHK := if\left(A_{vmin} < A_v, \text{"OK"}, \text{"NEED MORE STEEL"}\right)$

$SHEAR_STEEL_CHK = \text{"OK"}$

DESIGN CALCULATION COVER SHEET

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ISO 9001:2008 CERTIFIED

PROJECT:	**ODOT/EMH&T: HAM-75-7.85**	*ODOT PID NO:*	*77889*
BRIDGE NO:	**HAM-075-PROS**	*EMH&T JOB NO:*	
SUBJECT:	***Analysis***	**GF JOB NO:**	**54682**
	Sheet Piling	CALC NO.:	**HAM075_PROS_09B**
	Prosheet	REV NO.:	**00**
FILE NAME(s):	*PROS_FWD.bak*	**SHEET:**	**1 OF 25**

SUPERSEDES CALC NO: N/A

CALCULATION DESCRIPTION:

This will analyze the braced sheeting located at the east end of the caisson wall.

DESIGN BASIS OR REFERENCES:

NS Underpass Design Criteria
AREMA (2014) Chapter 15
AISC Steel Construction Manual, Thirteenth Edition
BDM - ODOT Bridge Design Manual (2004) - Current Edition

MISCELLANEOUS NOTES:

RECORD OF ISSUES:

REV	DESCRIPTION	BY	DATE	VERIF'D	DATE
00	Stage 2 Calculation	SNH	08/18/14	VDT	11/19/14

RECORD OF CHECK OF CURRENT CALCULATION VERSION

BY	DATE	CHKD	DATE	UPD.	DATE	VERIF'D	DATE
SNH	08/18/14	CTM	09/10/14	CTM	09/12/14	VDT	11/19/14

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Calc:	SNH	Date:	8/18/2014
Check:	CTM	Date:	9/10/2014
Update:	CTM	Date:	9/12/2014
Verified:	VDT	Date:	11/19/2014

Lateral Pressure from Parallel Strip Loading

Lateral Pressure from Parallel Strip Loading

Calculations as per Norfolk Southern Design Standards

Min. Depth of Resulting Table (Hs_{min}):	1.00 ft	Centerline Track Distance 1, CLT_1:	31.8 ft
Max Depth of Resulting Table (Hs_{max}):	56.00 ft	Centerline Track Distance 2, CLT_2:	31.8 ft
Spacing of Max Axle Load:	5.0 ft	Centerline Track Distance 3, CLT_3:	31.8 ft
Max Axle Load:	80.0 kip	Centerline Track Distance 4, CLT_4:	31.8 ft
Width of Tie (TL):	8.5 ft	Applied Pressure (q):	1.88 ksf

Hs	**β** *(Radians)*				**α** *(Radians)*				*Active Pressure,* **Ps** *(psf)*			
	β_1	β_2	β_3	β_4	α_1	α_2	α_3	α_4	Ps_1	Ps_2	Ps_3	Ps_4
0.00 ft	0.0000	0.0000	0.0000	0.0000	1.5708	1.5708	1.5708	1.5708	0.00	0.00	0.00	0.00
1.00 ft	0.0086	0.0086	0.0086	0.0086	1.5387	1.5387	1.5387	1.5387	20.54	20.54	20.54	20.54
3.75 ft	0.0317	0.0317	0.0317	0.0317	1.4511	1.4511	1.4511	1.4511	74.97	74.97	74.97	74.97
6.50 ft	0.0535	0.0535	0.0535	0.0535	1.3654	1.3654	1.3654	1.3654	122.80	122.80	122.80	122.80
9.25 ft	0.0730	0.0730	0.0730	0.0730	1.2828	1.2828	1.2828	1.2828	160.72	160.72	160.72	160.72
12.00 ft	0.0897	0.0897	0.0897	0.0897	1.2042	1.2042	1.2042	1.2042	187.27	187.27	187.27	187.27
14.75 ft	0.1035	0.1035	0.1035	0.1035	1.1302	1.1302	1.1302	1.1302	202.71	202.71	202.71	202.71
17.50 ft	0.1143	0.1143	0.1143	0.1143	1.0612	1.0612	1.0612	1.0612	208.51	208.51	208.51	208.51
20.25 ft	0.1223	0.1223	0.1223	0.1223	0.9972	0.9972	0.9972	0.9972	206.71	206.71	206.71	206.71
23.00 ft	0.1280	0.1280	0.1280	0.1280	0.9383	0.9383	0.9383	0.9383	199.43	199.43	199.43	199.43
25.75 ft	0.1316	0.1316	0.1316	0.1316	0.8841	0.8841	0.8841	0.8841	188.59	188.59	188.59	188.59
28.50 ft	0.1336	0.1336	0.1336	0.1336	0.8344	0.8344	0.8344	0.8344	175.72	175.72	175.72	175.72
31.25 ft	0.1343	0.1343	0.1343	0.1343	0.7888	0.7888	0.7888	0.7888	161.97	161.97	161.97	161.97
34.00 ft	0.1339	0.1339	0.1339	0.1339	0.7470	0.7470	0.7470	0.7470	148.15	148.15	148.15	148.15
36.75 ft	0.1327	0.1327	0.1327	0.1327	0.7088	0.7088	0.7088	0.7088	134.79	134.79	134.79	134.79
39.50 ft	0.1309	0.1309	0.1309	0.1309	0.6736	0.6736	0.6736	0.6736	122.19	122.19	122.19	122.19
42.25 ft	0.1287	0.1287	0.1287	0.1287	0.6413	0.6413	0.6413	0.6413	110.52	110.52	110.52	110.52
45.00 ft	0.1262	0.1262	0.1262	0.1262	0.6116	0.6116	0.6116	0.6116	99.86	99.86	99.86	99.86
47.75 ft	0.1235	0.1235	0.1235	0.1235	0.5843	0.5843	0.5843	0.5843	90.19	90.19	90.19	90.19
50.50 ft	0.1207	0.1207	0.1207	0.1207	0.5590	0.5590	0.5590	0.5590	81.48	81.48	81.48	81.48
53.25 ft	0.1178	0.1178	0.1178	0.1178	0.5356	0.5356	0.5356	0.5356	73.67	73.67	73.67	73.67
56.00 ft	0.1149	0.1149	0.1149	0.1149	0.5139	0.5139	0.5139	0.5139	66.68	66.68	66.68	66.68

Note: The following two pages contain numerical and graphical representations of these loads over a selected area.

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Update:	CTM	Date:	9/12/2014
Verified:	VDT	Date:	11/19/2014

Numerical Summary of Forces

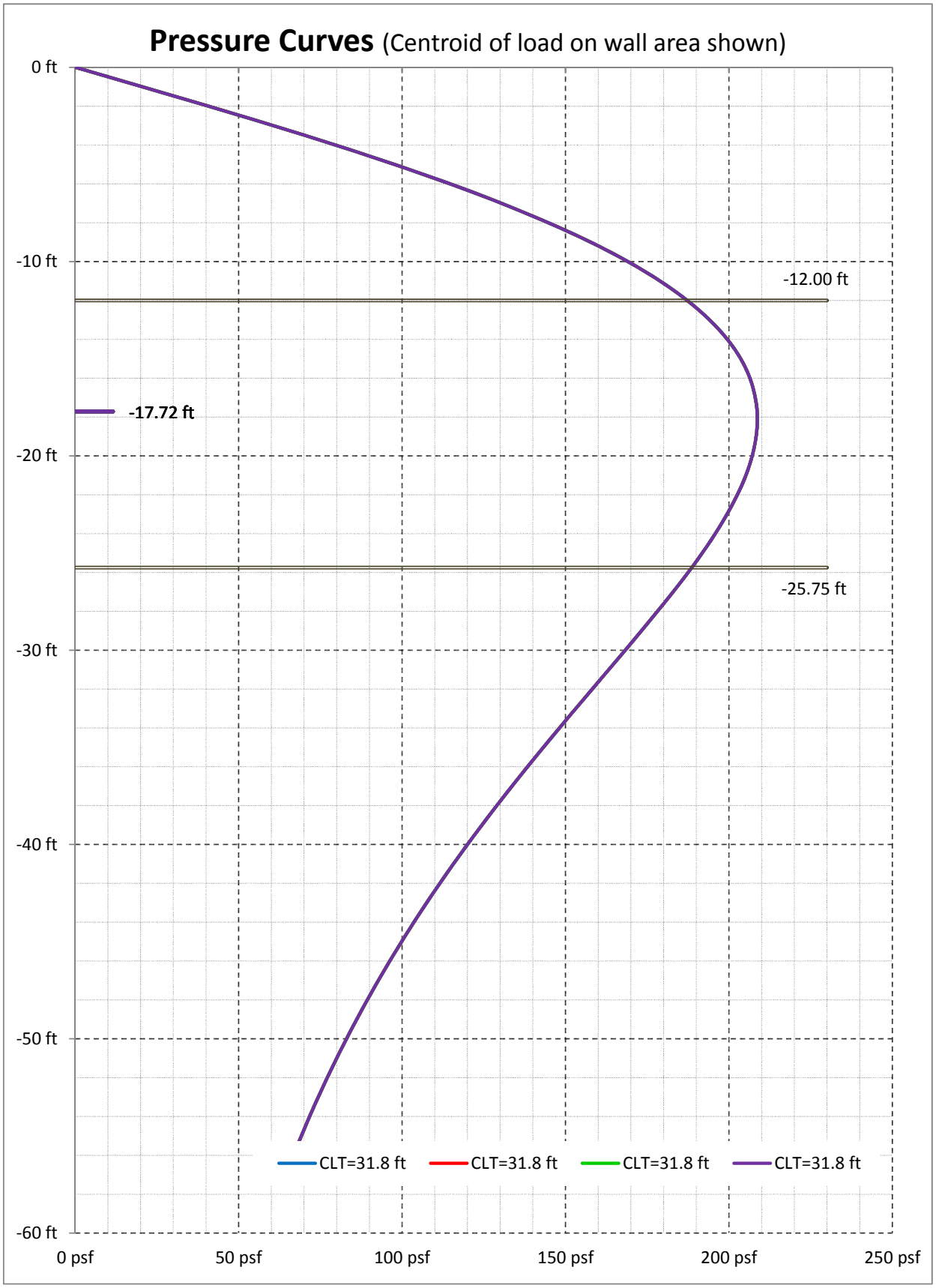
To gain total forces, the area under the graph is required. The area under the graph for the considered depth is divided by the total considered depth to arrive at a uniformly applied lateral pressure on any surface in question. The total lateral force is the total area under the graph. Simpsons Rule is used to arrive at the total area under the graph (median value force over a height is used)

	Values at CLT Distances			
Data Below is intended to give summary values that can be used in the design of walls and surfaces affected by the surcharge. A graphical output is on the next page	CLT=31.8 ft	CLT=31.8 ft	CLT=31.8 ft	CLT=31.8 ft
Top of Wall (Min. Depth Considered): 12.00 ft	187.3 psf	187.3 psf	187.3 psf	187.3 psf
Bottom of Wall (Max. Depth Considered): 25.75 ft	188.6 psf	188.6 psf	188.6 psf	188.6 psf

The bottom of the exposed wall face is 25.75 feet below the bottom of the tie. The top of the fill behind the wall (applying lateral pressure) is 12.00 feet below the bottom of the tie. The height of exposed wall face with surcharge pressure applied is 13.75 feet.

Maximum Applied Lateral Pressure on limits of the wall				
Maximum Applied Lateral Pressure	208.5 psf	208.5 psf	208.5 psf	208.5 psf
Total Applied Lateral Pressure on limits of the wall *(as an equivilant point load)*				
Total Lateral Load (Per foot)	3.38 klf	3.38 klf	3.38 klf	3.38 klf
Centroid of Total Lateral Load (Distance down from zero)	17.72 ft	17.72 ft	17.72 ft	17.72 ft
Total Applied Moment due to Surcharge *(assumes fixed at max depth)*	27.1 k-ft	27.1 k-ft	27.1 k-ft	27.1 k-ft
Uniformly Applied Lateral Pressure on limits of the wall (For Lateral Stability) *(Total Applied Lateral Pressure **Thrust Controls;** Total lateral moment is not correct)*				
Uniformly Applied Lateral Load	245.7 psf	245.7 psf	245.7 psf	245.7 psf
Moment due to Uniform Load *(assumes fixed at max depth)*	23.2 k-ft	23.2 k-ft	23.2 k-ft	23.2 k-ft
Difference in moment from that of the Total Point Load	-14%	-14%	-14%	-14%
Uniformly Applied Lateral Pressure on limits of the wall (For Moments/Deflectoins) *(Total Applied Lateral Pressure **Moment Controls;** Total lateral thrust is not correct)*				
Uniformly Applied Lateral Load	287.1 psf	287.1 psf	287.1 psf	287.1 psf
Moment due to Uniform Load *(assumes fixed at max depth)*	27.1 k-ft	27.1 k-ft	27.1 k-ft	27.1 k-ft
Difference in moment from that of the Total Point Load	0%	0%	0%	0%
Total of Uniformly Applied Lateral Load	3.95 klf	3.95 klf	3.95 klf	3.95 klf
Difference in Load from that of the Total Point Load	17%	17%	17%	17%

Total Uniform Load to Approximate Dual Tracks on Wall	**574.2 psf**


Pressure Curves (Centroid of load on wall area shown)
0 ft
-10 ft
-20 ft
-30 ft
-40 ft
-50 ft
-60 ft
-12.00 ft
-17.72 ft
-25.75 ft
CLT=31.8 ft
CLT=31.8 ft
CLT=31.8 ft
CLT=31.8 ft
0 psf
50 psf
100 psf
150 psf
200 psf
250 psf

Calc:	SNH	Date:	8/18/2014
Check:	CTM	Date:	9/10/2014
Update:	CTM	Date:	9/12/2014
Verified:	VDT	Date:	11/19/2014

L-Pile Loads on a 30" Wide Shaft should be entered as follows (depth is inches from pile head)

Shaft Width: 12.0 in
1.000 ft

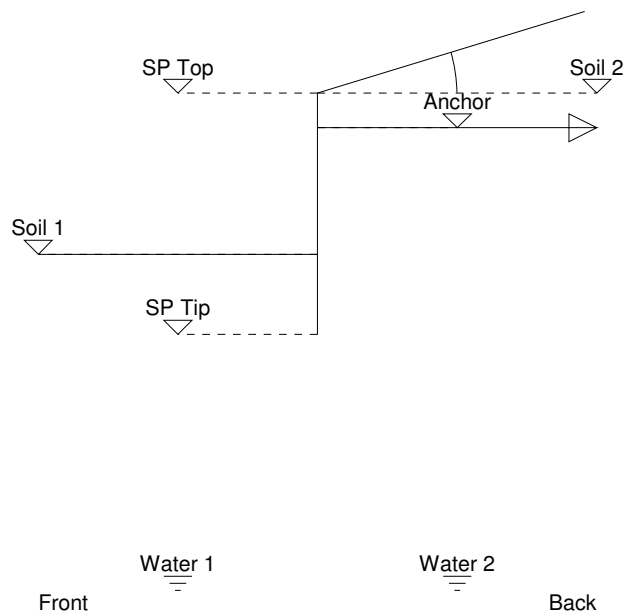
Depth	Depth	Last CLT column Lateral Load
12.00 ft	0.000 ft	0.187 ksf
14.75 ft	2.750 ft	0.203 ksf
17.50 ft	5.500 ft	0.209 ksf
20.25 ft	8.250 ft	0.207 ksf
25.75 ft	13.750 ft	0.189 ksf
31.25 ft	19.250 ft	0.162 ksf
36.75 ft	24.750 ft	0.135 ksf
42.25 ft	30.250 ft	0.111 ksf
47.75 ft	35.750 ft	0.090 ksf
56.00 ft	44.000 ft	0.067 ksf

Sheet Pile Design According to Blum-Method

Project Name:	HAM075-PROSC
Date:	9/16/2014
Author:	SNH
Company:	
Comment:	Waler Braced Section at SW Wingwall

Geodata

	Unit
Sheet Pile Top Level [ft]	0.000
Sheet Pile Tip Level [ft]	20.956
Soil Level in Front [ft]	14.000
Soil Level behind [ft]	0.000
Anchorlevel [ft]	3.000
Water Level in Front [ft]	42.000
Water Level behind [ft]	42.000
Soil Surface Inclination in Front [Deg]	0.000
Soil Surface Inclination behind [Deg]	17.000
Caquot Surcharge in Front [kip/ft2]	0.000
Caquot Surcharge behind [kip/ft2]	0.000
Anchor Inclination [Deg]	0.000
Earth Support	Free



Soil Layers

Layers in Front

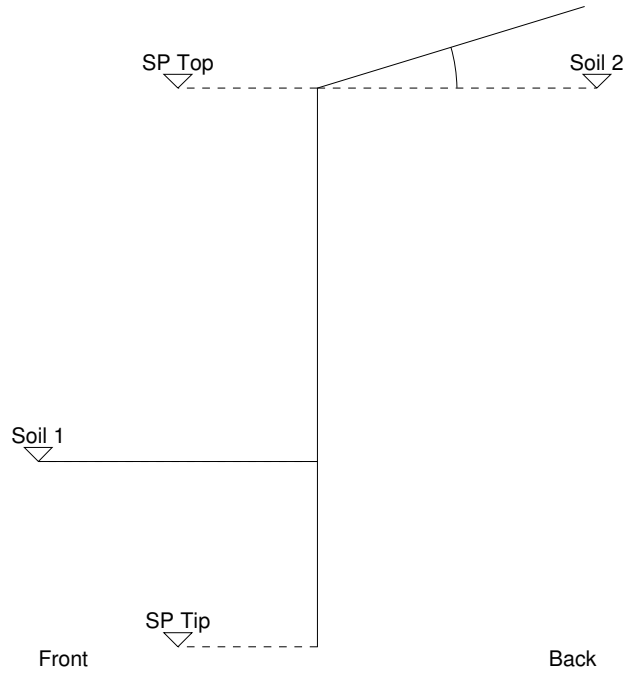
	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kph	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	100.000	0.125	0.063	3.128	31.000	0.000	0.000

Layers behind

	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kph	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	100.000	0.125	0.063	0.397	31.000	0.000	0.000

Boussinesq

	Distance Wall [ft]	Width Surcharge [ft]	Depth Surcharge [ft]	Surcharge [kip/ft2]

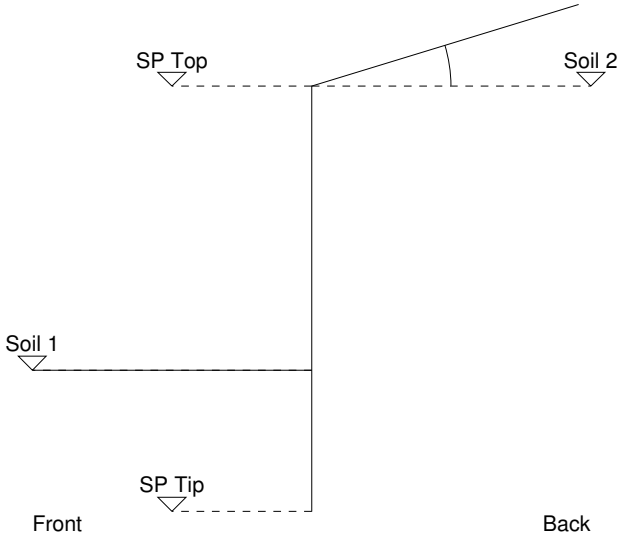


Userdefined Pressures

	Pressure Top [kip/ft2]	Pressure Tip [kip/ft2]	Depth Top [ft]	Depth Tip [ft]
Strip 1	0.187	0.203	0.000	2.750
Strip 2	0.203	0.209	2.750	5.500
Strip 3	0.209	0.207	5.500	8.250
Strip 4	0.207	0.189	8.250	13.750
Strip 5	0.189	0.162	13.750	19.250
Strip 6	0.162	0.135	19.250	24.750
Strip 7	0.135	0.111	24.750	30.250
Strip 8	0.111	0.090	30.250	35.750
Strip 9	0.090	0.067	35.750	44.000

Concentrated Forces

	Horiz. Component [kip/ft]	Vert. Component [kip/ft]	Depth Horiz. Comp. [ft]



Pile Section

Name	AZ 12-700
Inertia [in4/ft]	138.254
Modulus [in3/ft]	22.413
Area [in2/ft]	5.821
Mass [lbs/ft2]	19.806
Steel Grade [lb/in2]	39000.000
Requested Safety	1.500

All Values

Depth [ft]	Deflection [ft]	Rotation [Rad]	Cross Force [kip/ft]	Moment [kipft/ft]	Total Pressure [kip/ft2]	Earth Pressure in Front [kip/ft2]	behind [kip/ft2]	Water Pressure [kip/ft2]	Userdefined Pressure [kip/ft2]
0.000	0.012	-0.004	0.000	0.000	0.187	0.000	0.000	0.000	0.187
0.820	0.008	-0.004	0.172	0.068	0.232	0.000	0.041	0.000	0.192
0.820	0.008	-0.004	0.172	0.068	0.232	0.000	0.041	0.000	0.192
1.640	0.005	-0.004	0.381	0.292	0.278	0.000	0.081	0.000	0.197
1.640	0.005	-0.004	0.381	0.292	0.278	0.000	0.081	0.000	0.197
2.461	0.002	-0.004	0.628	0.704	0.323	0.000	0.122	0.000	0.201
2.461	0.002	-0.004	0.628	0.704	0.323	0.000	0.122	0.000	0.201
2.750	0.001	-0.004	0.724	0.899	0.339	0.000	0.136	0.000	0.203
2.750	0.001	-0.004	0.724	0.899	0.339	0.000	0.136	0.000	0.203
3.000	0.000	-0.004	0.810	1.091	0.352	0.000	0.149	0.000	0.204
3.000	0.000	-0.004	-4.630	1.091	0.352	0.000	0.149	0.000	0.204
3.820	-0.003	-0.004	-4.324	-2.584	0.395	0.000	0.189	0.000	0.205
3.820	-0.003	-0.004	-4.324	-2.584	0.395	0.000	0.189	0.000	0.205
4.640	-0.006	-0.004	-3.982	-5.992	0.437	0.000	0.230	0.000	0.207
4.640	-0.006	-0.004	-3.982	-5.992	0.437	0.000	0.230	0.000	0.207
5.461	-0.009	-0.003	-3.606	-9.107	0.480	0.000	0.271	0.000	0.209
5.461	-0.009	-0.003	-3.606	-9.107	0.480	0.000	0.271	0.000	0.209
5.500	-0.009	-0.003	-3.588	-9.249	0.482	0.000	0.273	0.000	0.209
5.500	-0.009	-0.003	-3.588	-9.249	0.482	0.000	0.273	0.000	0.209
6.320	-0.012	-0.003	-3.176	-12.025	0.522	0.000	0.313	0.000	0.208
6.320	-0.012	-0.003	-3.176	-12.025	0.522	0.000	0.313	0.000	0.208
7.140	-0.015	-0.003	-2.732	-14.450	0.562	0.000	0.354	0.000	0.208
7.140	-0.015	-0.003	-2.732	-14.450	0.562	0.000	0.354	0.000	0.208
7.961	-0.017	-0.002	-2.254	-16.497	0.602	0.000	0.395	0.000	0.207
7.961	-0.017	-0.002	-2.254	-16.497	0.602	0.000	0.395	0.000	0.207
8.250	-0.017	-0.002	-2.078	-17.123	0.616	0.000	0.409	0.000	0.207
8.250	-0.017	-0.002	-2.078	-17.123	0.616	0.000	0.409	0.000	0.207
9.070	-0.019	-0.002	-1.557	-18.616	0.654	0.000	0.450	0.000	0.204
9.070	-0.019	-0.002	-1.557	-18.616	0.654	0.000	0.450	0.000	0.204
9.890	-0.020	-0.001	-1.005	-19.669	0.692	0.000	0.490	0.000	0.202
9.890	-0.020	-0.001	-1.005	-19.669	0.692	0.000	0.490	0.000	0.202
10.711	-0.021	-0.001	-0.422	-20.257	0.730	0.000	0.531	0.000	0.199
10.711	-0.021	-0.001	-0.422	-20.257	0.730	0.000	0.531	0.000	0.199
11.531	-0.021	0.000	0.192	-20.353	0.768	0.000	0.572	0.000	0.196
11.531	-0.021	0.000	0.192	-20.353	0.768	0.000	0.572	0.000	0.196
12.351	-0.021	0.001	0.838	-19.933	0.806	0.000	0.612	0.000	0.194
12.351	-0.021	0.001	0.838	-19.933	0.806	0.000	0.612	0.000	0.194
13.171	-0.020	0.001	1.515	-18.970	0.844	0.000	0.653	0.000	0.191
13.171	-0.020	0.001	1.515	-18.970	0.844	0.000	0.653	0.000	0.191
13.750	-0.020	0.001	2.011	-17.951	0.871	0.000	0.682	0.000	0.189
13.750	-0.020	0.001	2.011	-17.951	0.871	0.000	0.682	0.000	0.189
14.000	-0.019	0.002	2.230	-17.421	0.882	0.000	0.694	0.000	0.188
14.000	-0.019	0.002	2.230	-17.421	0.882	0.000	0.694	0.000	0.188
14.820	-0.018	0.002	2.837	-15.327	0.598	-0.321	0.735	0.000	0.184
14.820	-0.018	0.002	2.837	-15.327	0.598	-0.321	0.735	0.000	0.184
15.640	-0.016	0.002	3.211	-12.831	0.314	-0.641	0.776	0.000	0.180
15.640	-0.016	0.002	3.211	-12.831	0.314	-0.641	0.776	0.000	0.180
16.461	-0.014	0.003	3.352	-10.124	0.030	-0.962	0.816	0.000	0.176
16.461	-0.014	0.003	3.352	-10.124	0.030	-0.962	0.816	0.000	0.176
17.281	-0.011	0.003	3.259	-7.397	-0.254	-1.283	0.857	0.000	0.172
17.281	-0.011	0.003	3.259	-7.397	-0.254	-1.283	0.857	0.000	0.172
18.101	-0.009	0.003	2.934	-4.841	-0.538	-1.604	0.898	0.000	0.168
18.101	-0.009	0.003	2.934	-4.841	-0.538	-1.604	0.898	0.000	0.168
18.921	-0.006	0.003	2.376	-2.647	-0.822	-1.924	0.938	0.000	0.164
18.921	-0.006	0.003	2.376	-2.647	-0.822	-1.924	0.938	0.000	0.164
19.250	-0.005	0.003	2.087	-1.912	-0.936	-2.053	0.955	0.000	0.162
19.250	-0.005	0.003	2.087	-1.912	-0.936	-2.053	0.955	0.000	0.162
20.070	-0.002	0.003	1.203	-0.547	-1.220	-2.373	0.995	0.000	0.158
20.070	-0.002	0.003	1.203	-0.547	-1.220	-2.373	0.995	0.000	0.158
20.890	0.000	0.003	0.086	-0.002	-1.504	-2.694	1.036	0.000	0.154
20.890	0.000	0.003	0.086	-0.002	-1.504	-2.694	1.036	0.000	0.154
20.956	0.001	0.003	-0.014	0.000	-1.527	-2.720	1.039	0.000	0.154

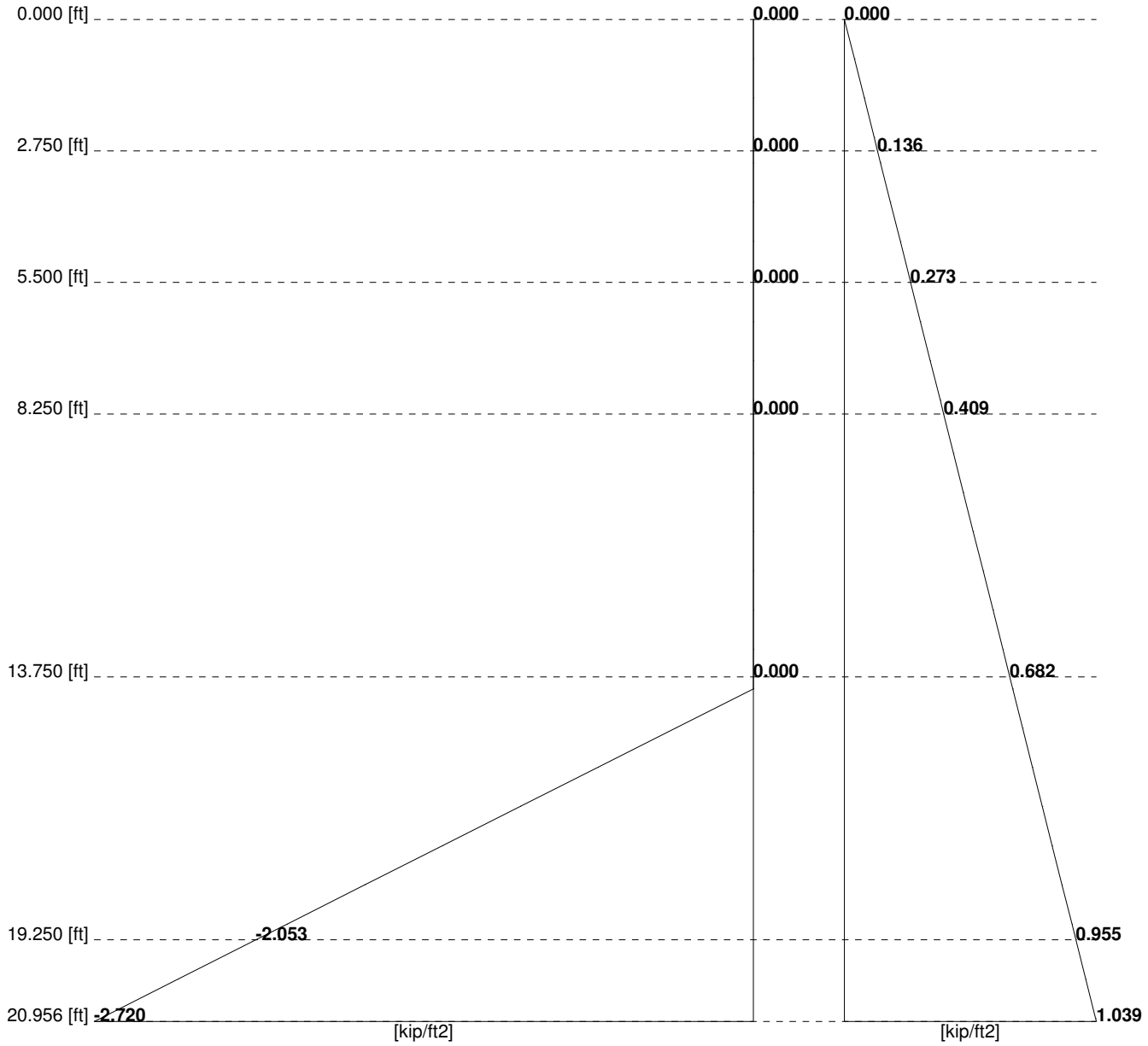
Extremal Values

	z Min [ft]	Min	z Max [ft]	Max
Deflection [ft]	11.531	-0.021	0.000	0.012
Cross Force [kip/ft]	3.000	-4.630	16.461	3.352
Moment [kipft/ft]	11.301	-20.377	3.000	1.091

Pile Check

		Depth [ft]
Name	AZ 12-700	
Inertia [in4/ft]	138.254	
Modulus [in3/ft]	22.413	
Area [in2/ft]	5.821	
Mass [lbs/ft2]	19.806	
Steel Grade [lb/in2]	39000.000	
Minimal Moment [kipft/ft]	-20.377	11.301
Maxmimal Moment [kipft/ft]	1.091	3.000
Normal Forces at Max. Moment [kip/ft]	0.000	11.301
Normal Forces at Min. Moment [kip/ft]	0.000	3.000
Deflection at Min. Moment [ft]	-0.021	11.301
Deflection at Max. Moment [ft]	0.000	3.000
Min. Stress at Min. Moment [lb/in2]	-10909.592	11.301
Max. Stress at Min. Moment [lb/in2]	10909.592	11.301
Min. Stress at Max. Moment [lb/in2]	-583.991	3.000
Max. Stress at Max. Moment [lb/in2]	583.991	3.000
Safety > Req. Safety = 1.500	3.575	
Sheet Pile Top Level [ft]	0.000	
Sheet Pile Tip Level [ft]	20.956	
Sheet Pile Length [ft]	20.956	
Included OverLength [ft]	0.000	
Vertical Equilibrium [kip/ft]	0.000	
Anchor Force (horiz.) [kip/ft]	-5.440	

Earth Pressure Diagram

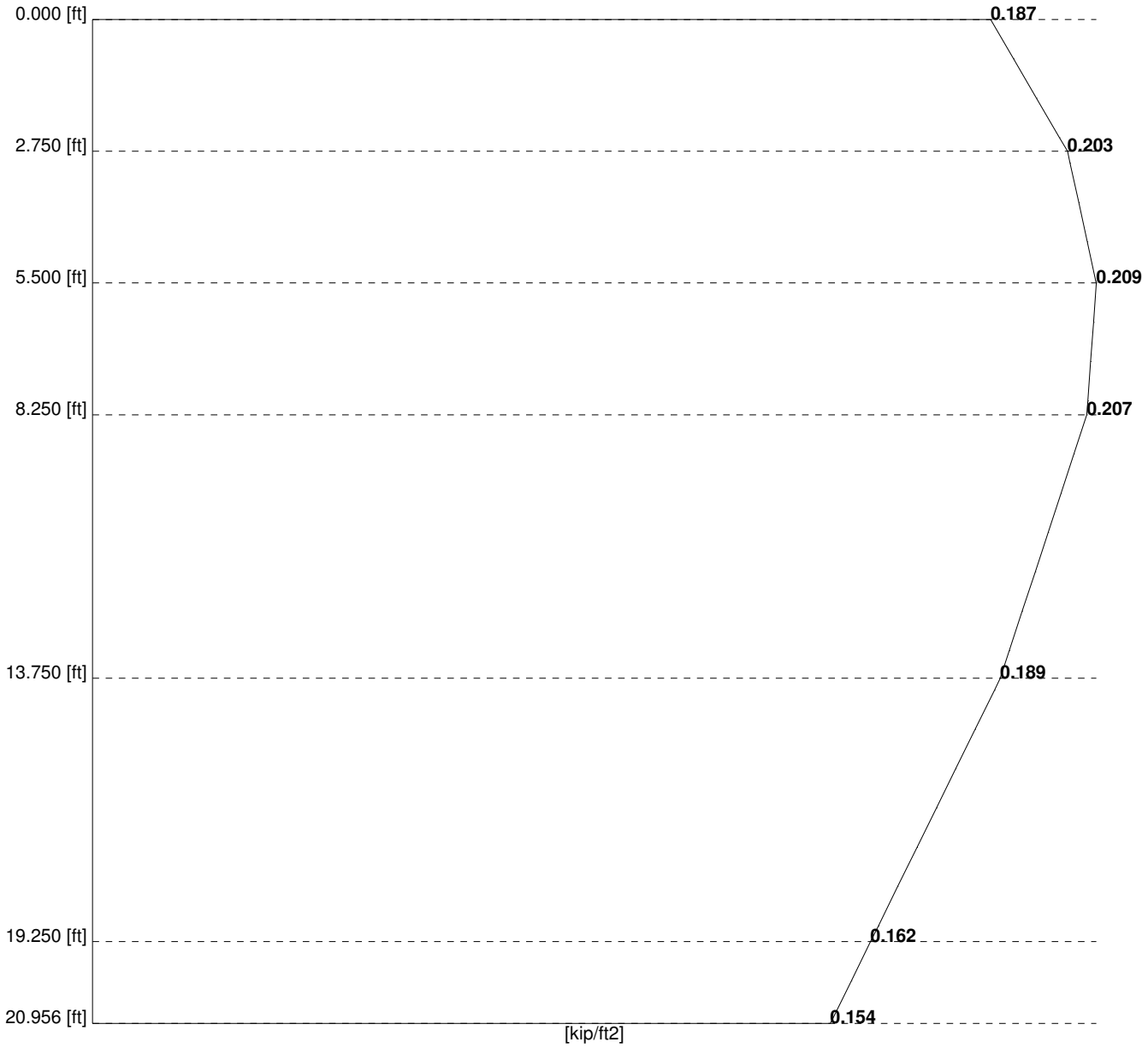


Water Pressure Diagram

0.000 [ft]	**0.000**
2.750 [ft]	**0.000**
5.500 [ft]	**0.000**
8.250 [ft]	**0.000**
13.750 [ft]	**0.000**
19.250 [ft]	**0.000**
20.956 [ft]	**0.000**

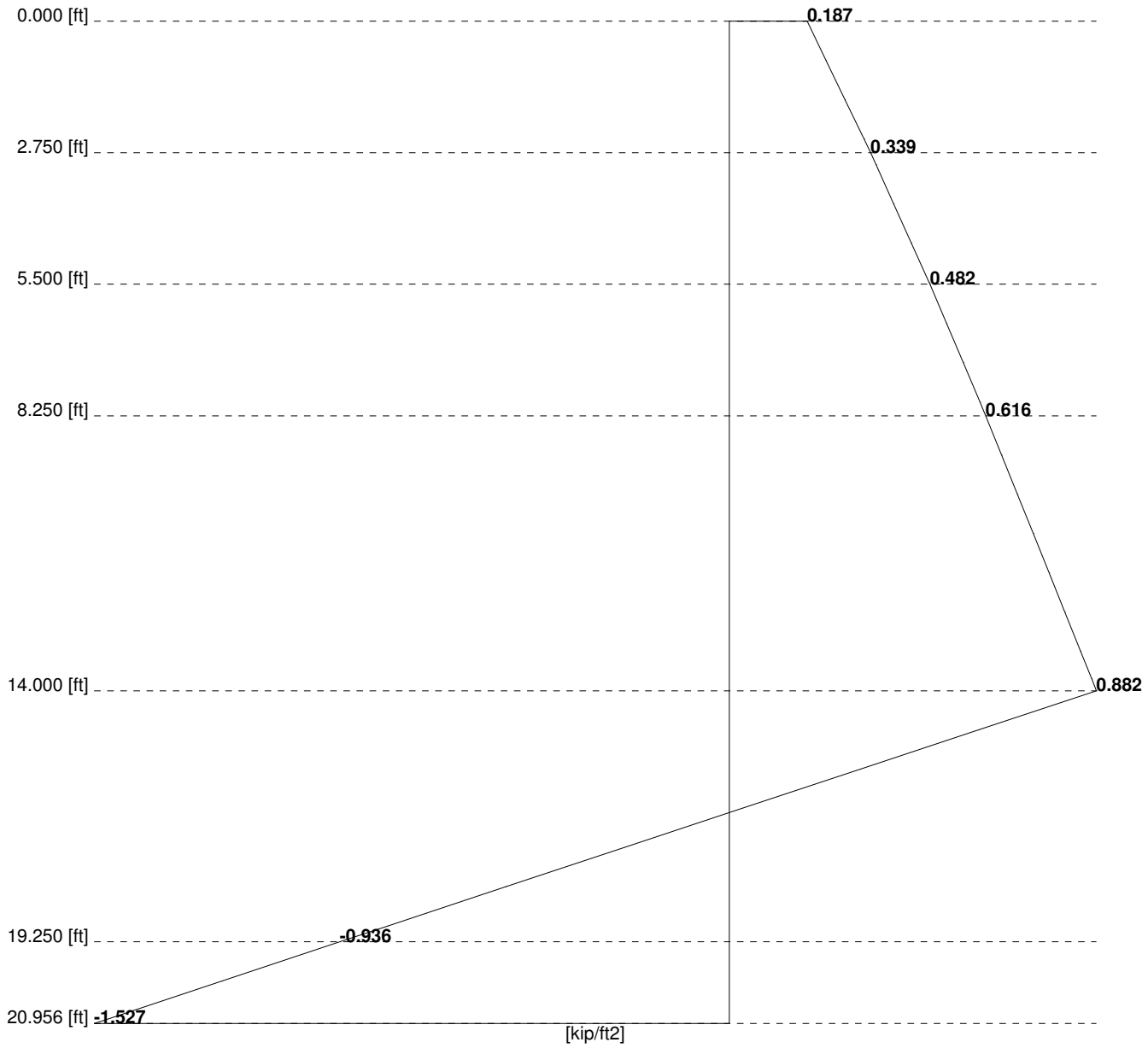
[kip/ft2]

Userdefined Pressure Diagram

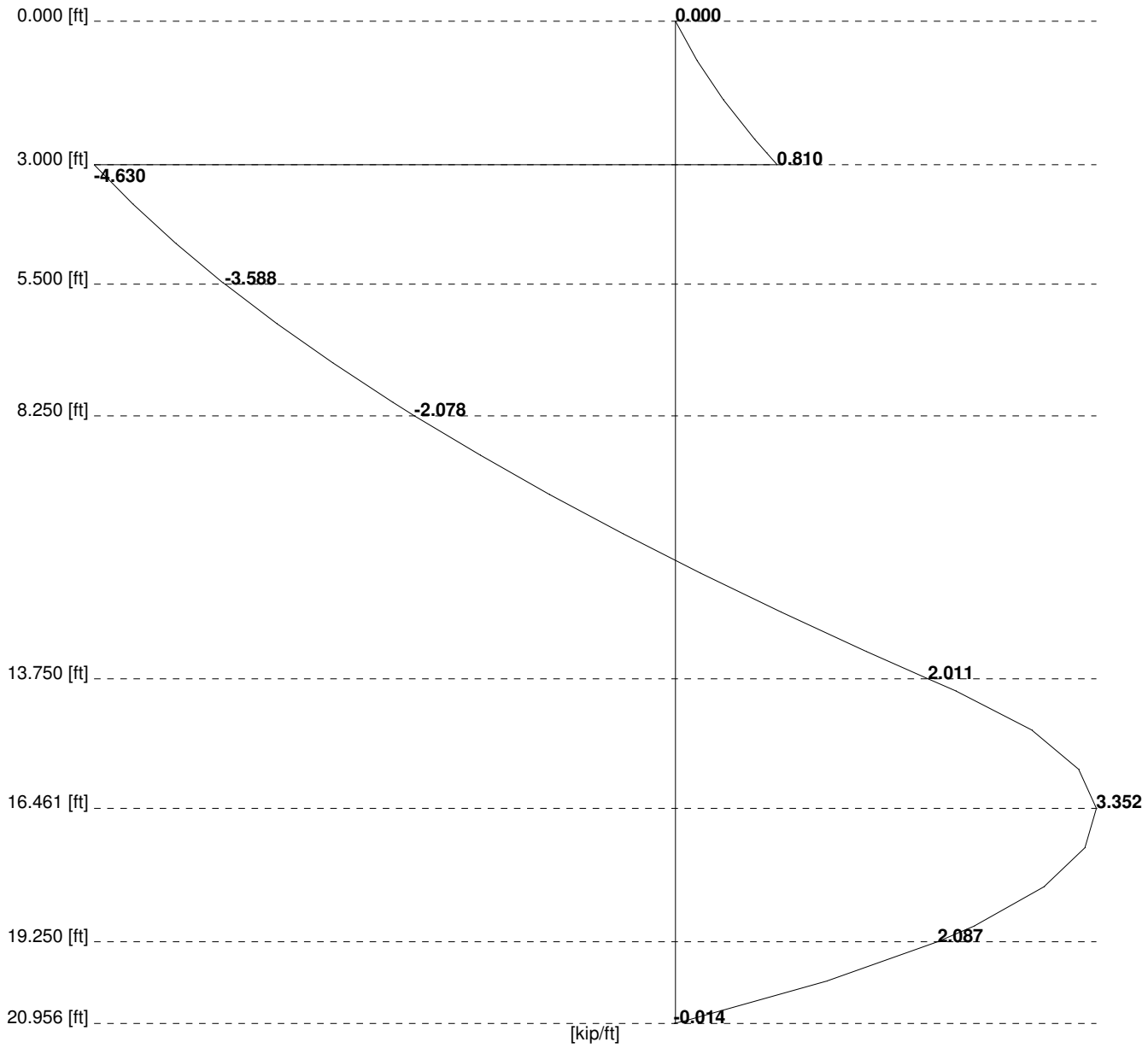


Boussinesq Diagram

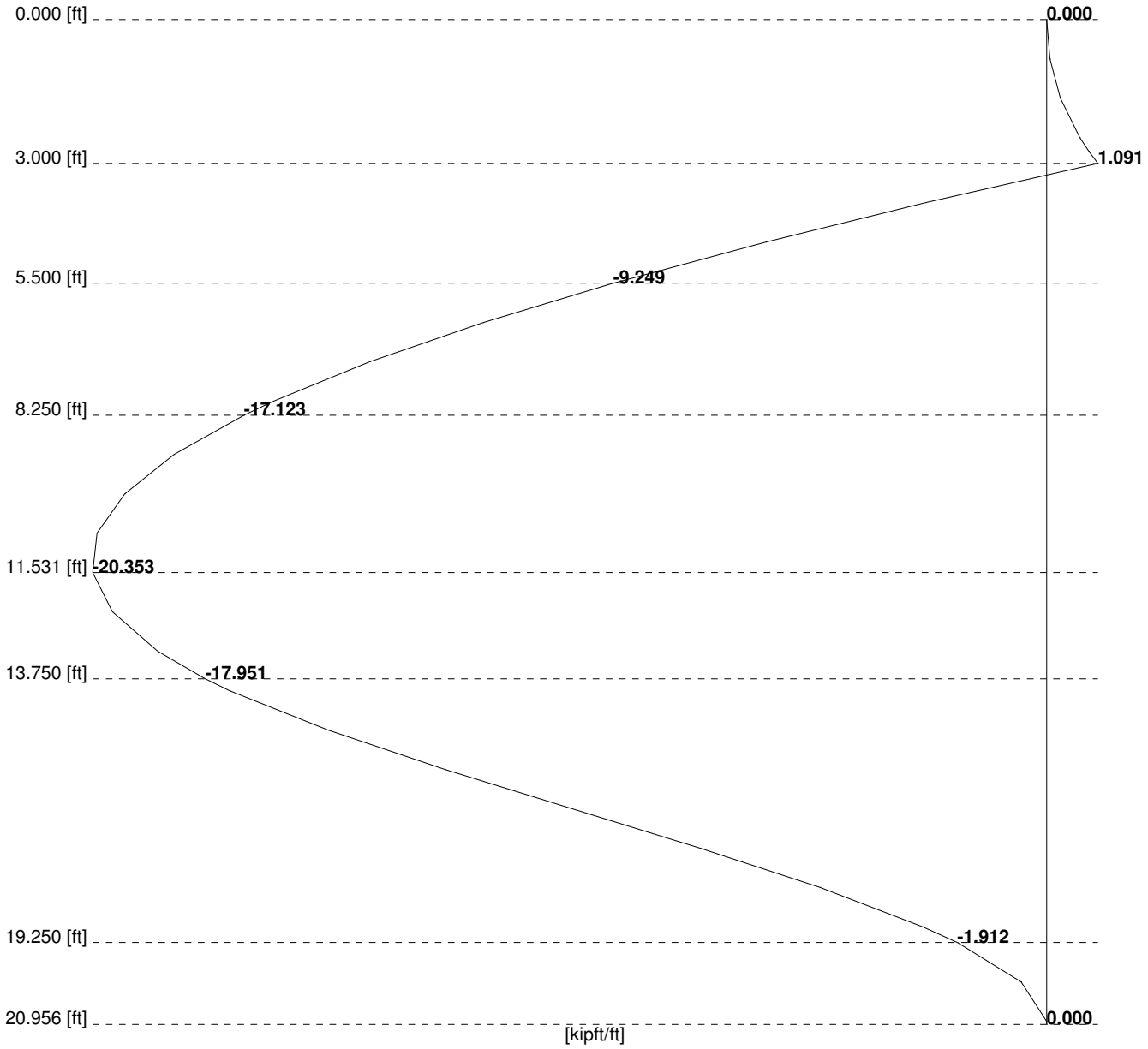
Total Pressure Diagram



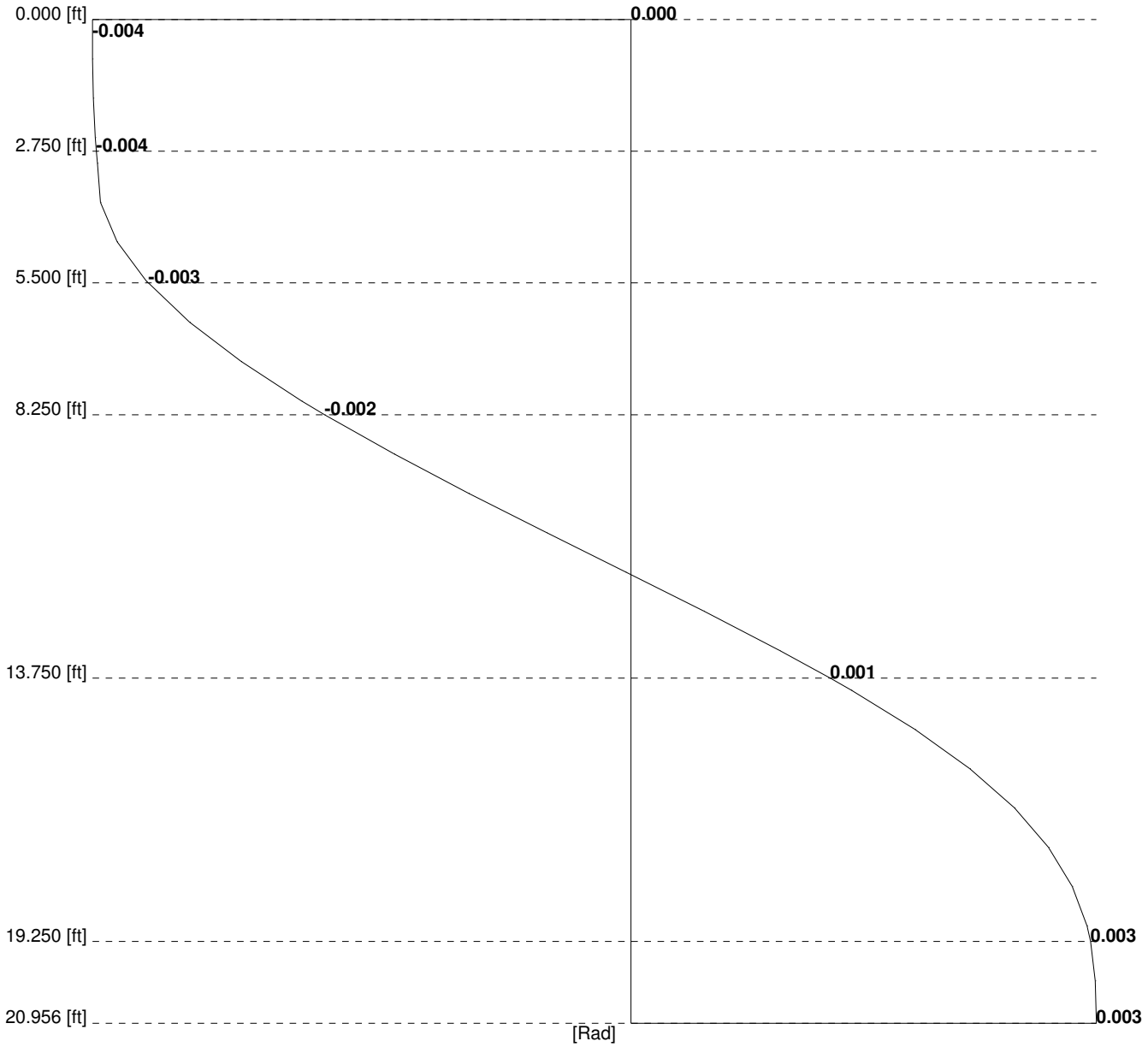
Cross Force Diagram



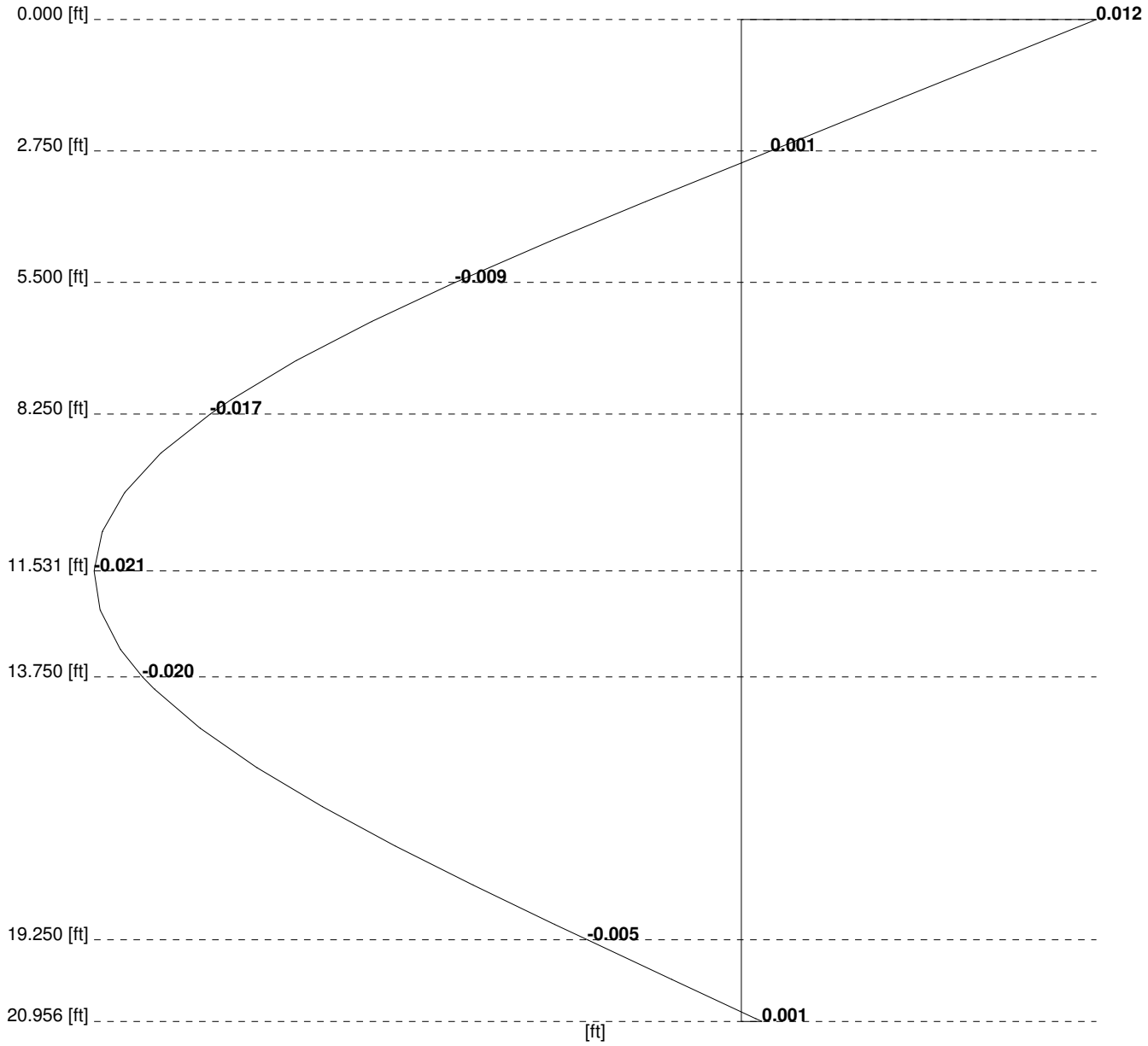
Moment Diagram



Rotation Diagram

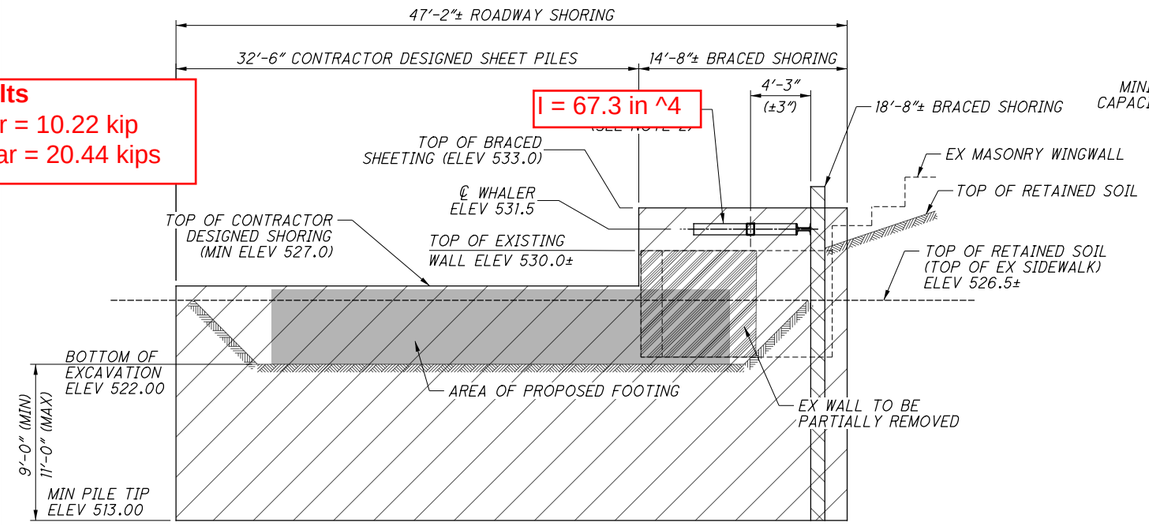


Deflection Diagram



7/8" dia bolts
single shear = 10.22 kip
double shear = 20.44 kips

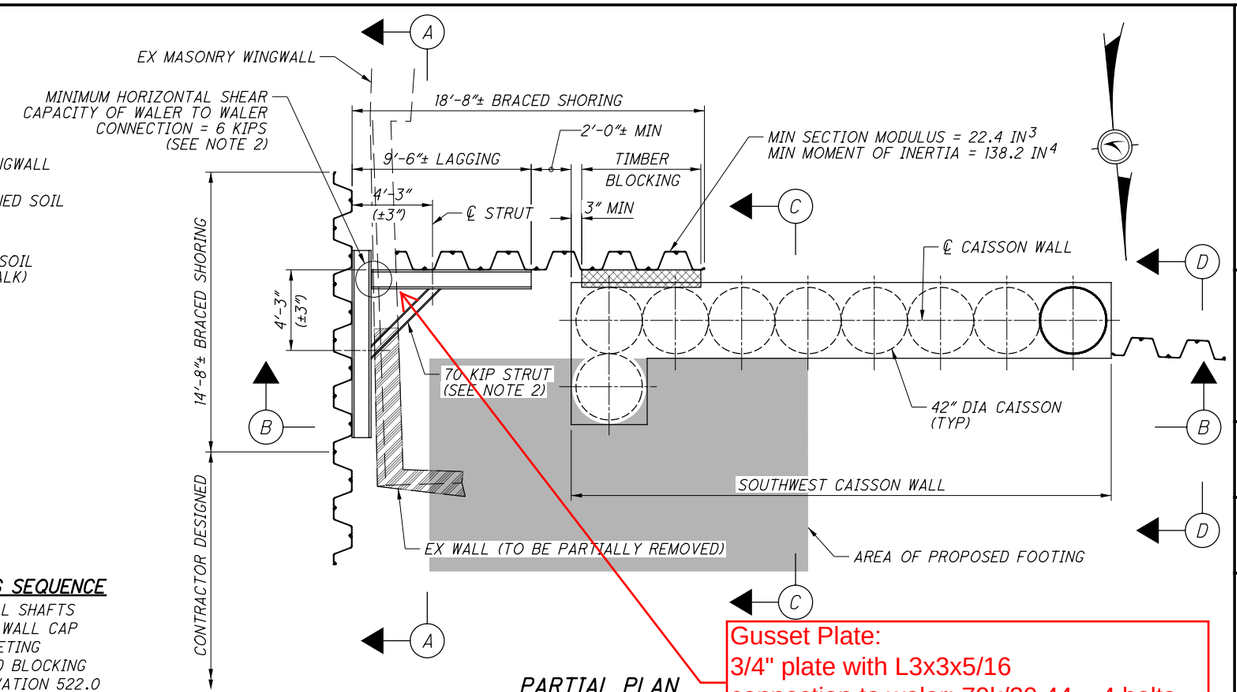
I = 67.3 in ^4



(A) ELEVATION ALONG ROADWAY
FORWARD ABUTMENT, LOOKING DOWNSTATION

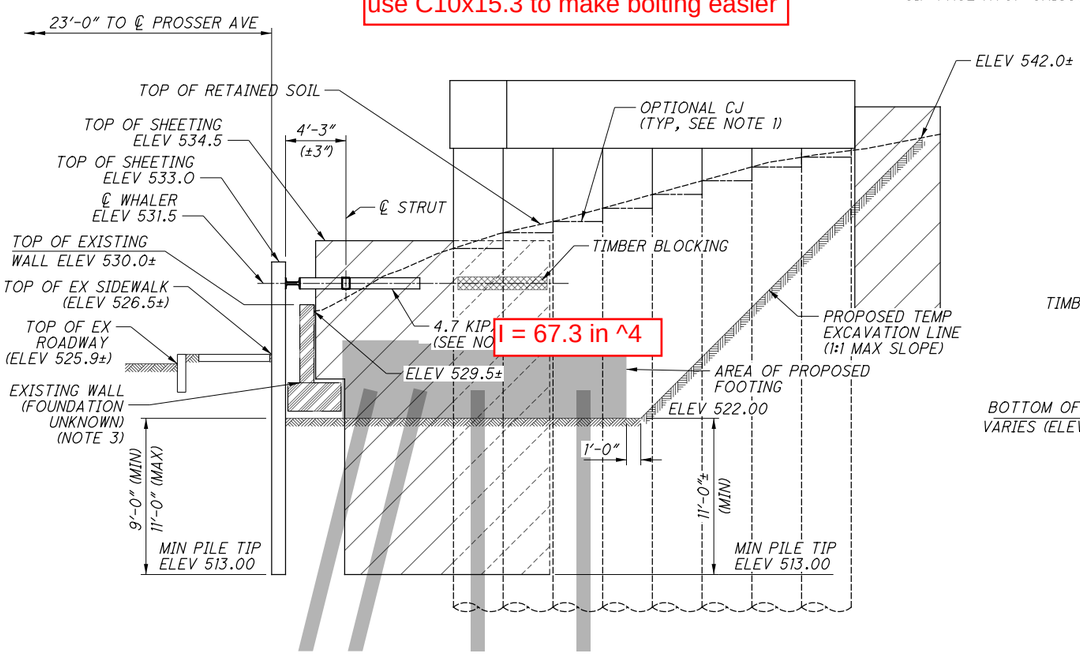
waler:
5.4 k/ft
unsupported length ~ 3 ft
AISC Table 3-8
use C10x15.3 to make bolting easier

SUGGESTED SHORING SEQUENCE
1) INSTALL CAISSON WALL SHAFTS
2) CONSTRUCT CAISSON WALL CAP
3) INSTALL BRACED SHEETING
4) INSTALL BRACING AND BLOCKING
5) EXCAVATION TO ELEVATION 522.0
6) PARTIALLY REMOVE EX WALL
7) CONSTRUCT PROPOSED STRUCTURE
8) REMOVE BRACED AND CONTRACTOR DESIGNED SHORING
9) GRADE TO FINAL ELEVATIONS AND CAST CIP FACE ATOP CAISSON WALL



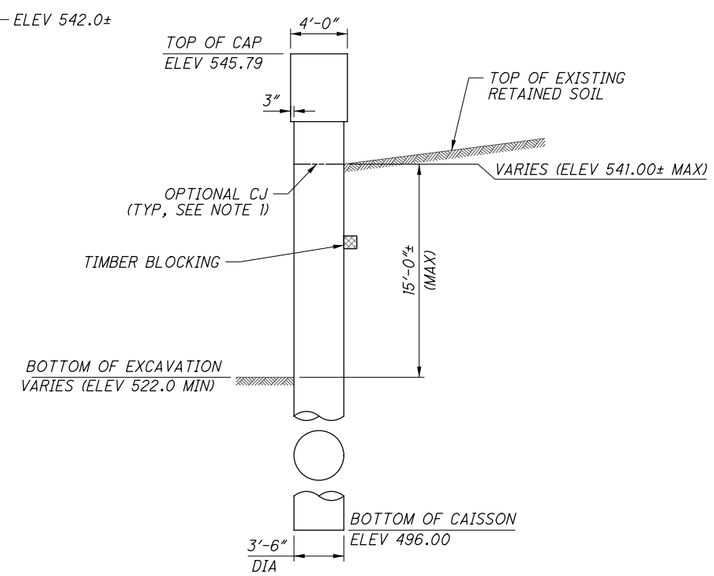
Gusset Plate:
3/4" plate with L3x3x5/16
connection to waler: 70k/20.44 = 4 bolts

PARTIAL PLAN
(AT SOUTHWEST WINGWALL)
UPSTATION



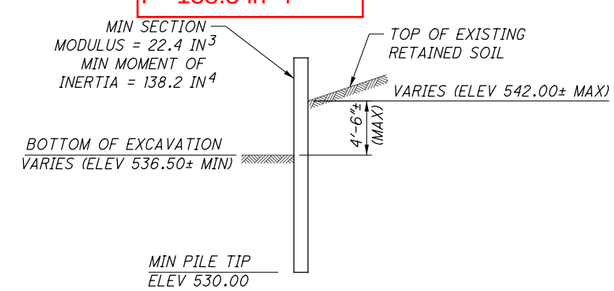
I = 67.3 in ^4

(B) ELEVATION AT CAISSON WALL
FORWARD ABUTMENT, LOOKING SOUTH



(C) SECTION OF CAISSON WALL
FORWARD ABUTMENT, TEMPORARY CONDITION

I = 138.3 in^4



(D) SECTION OF SHEETING
FORWARD ABUTMENT, TEMPORARY CONDITION

TOTAL DEFLECTION
Sheeting: .021 ft = .252 in
Waler: .015 in (from 562, conservative)
TOTAL = .267 in < .5 in

OF EACH CAISSON. THE CAISSON WALL IS USED FOR TEMPORARY SHORING AND INCORPORATED AS A FINAL WINGWALL. FOR FURTHER CAISSON DETAILS, INCLUDING REINFORCING, SEE SHEET 20/32.

2. CONTRACTOR SHALL PROBE TO ESTABLISH THE LIMITS OF THE EXISTING RETAINING WALL FOOTING. IF SHORING DEVIATES FROM WHAT IS SHOWN ON THESE PLANS, NEW SHORING PLANS SHALL BE SUBMITTED TO ODOT AND NSRR FOR REVIEW AND APPROVAL.

DESIGN AGENCY
Gannett Fleming
ENGINEERS & ARCHITECTS, P.C.
4151 EXECUTIVE PARKWAY SUITE 350
WESTERVILLE, OHIO 43081

DESIGNED	DRAWN	REVIEWED	DATE
SNH	CJG/EFD	PLC	11/2014
CHECKED	REVISED	ODOT SFN: XXXXXXX	
CTM		NSRR BR#: BR0018444	

RAILROAD SHORING PLAN & ELEVATIONS
BRIDGE NO. HAM-75-PROSSER (NSRR CT-0.89: CINCINNATI, OH)
NORFOLK SOUTHERN RAILROAD OVER PROSSER AVENUE

HAM-75-7.85
PID No. 77889

pw:\\gfpwing02.corporate.gannettfleming.com:GFPW02\Documents\Projects\54682\77889\structures\HAM_PROSSER AVE\sheets\07 5/28/2015 3:03:47 PM sharris

DESIGN CALCULATION COVER SHEET

Gannett Fleming
Engineers and Architects, P.C.
Excellence Delivered ***As Promised***

ISO 9001:2008 CERTIFIED

PROJECT: **ODOT/EMH&T: HAM-75-7.85**
ODOT PID NO: 77889

BRIDGE NO: **HAM-075-PROS**
EMH&T JOB NO:

SUBJECT: ***Analysis***
Sheet Piling
Prosheet
GF JOB NO: 54682

CALC NO.: **HAM075_PROS_09U**

REV NO.: **01**

FILE NAME(s): *HAM-PROSS_Unbraced Section at SW Wingwall.bak*

SHEET: 1 OF 44

SUPERSEDES CALC NO: 00 (Stage 2 Calculation)

CALCULATION DESCRIPTION:

The first section will analyze the unbraced sheeting located at the west end of the caisson wall. The second section is the design of the unbraced sheeting at the southeast wingwall.

DESIGN BASIS OR REFERENCES:

NS Underpass Design Criteria
AREMA (2014) Chapter 15
AISC Steel Construction Manual, Thirteenth Edition
BDM - ODOT Bridge Design Manual (2004) - Current Edition

MISCELLANEOUS NOTES:

Stage 3 Update: Added new section of sheeting, no RR surcharge.

RECORD OF ISSUES:

REV	DESCRIPTION	BY	DATE	VERIF'D	DATE
00	Stage 2 Calculation	CTM	10/03/14	VDT	11/19/14
01	Stage 3 Update	VDT	01/20/16	CTM	04/25/16

RECORD OF CHECK OF CURRENT CALCULATION VERSION

BY	DATE	CHKD	DATE	UPD.	DATE	VERIF'D	DATE
CTM	10/03/14	VDT	11/07/14	VDT	01/20/16	CTM	04/25/16

Gannett Fleming Engineers and Architects, P.C.
Excellence Delivered As Promised

Calc:	CTM	Date:	10/3/2014
Check:	VDT	Date:	11/7/2014
Update:	SNH	Date:	11/19/2014
Verified:	VDT	Date:	11/19/2014

Lateral Pressure from Parallel Strip Loading

Lateral Pressure from Parallel Strip Loading

Calculations as per Norfolk Southern Design Standards

Min. Depth of Resulting Table (Hs_{min}):	1.00 ft	Centerline Track Distance 1, CLT_1:	19.60 ft
Max Depth of Resulting Table (Hs_{max}):	41.00 ft	Centerline Track Distance 2, CLT_2:	19.60 ft
Spacing of Max Axle Load:	5.0 ft	Centerline Track Distance 3, CLT_3:	19.60 ft
Max Axle Load:	80.0 kip	Centerline Track Distance 4, CLT_4:	19.60 ft
Width of Tie (TL):	8.5 ft	Applied Pressure (q):	1.88 ksf

Hs	**β** *(Radians)*				**α** *(Radians)*				*Active Pressure,* **Ps** *(psf)*			
	β_1	β_2	β_3	β_4	α_1	α_2	α_3	α_4	Ps_1	Ps_2	Ps_3	Ps_4
0.00 ft	0.0000	0.0000	0.0000	0.0000	1.5708	1.5708	1.5708	1.5708	0.00	0.00	0.00	0.00
1.00 ft	0.0232	0.0232	0.0232	0.0232	1.5173	1.5173	1.5173	1.5173	55.32	55.32	55.32	55.32
3.00 ft	0.0679	0.0679	0.0679	0.0679	1.4117	1.4117	1.4117	1.4117	158.54	158.54	158.54	158.54
5.00 ft	0.1082	0.1082	0.1082	0.1082	1.3100	1.3100	1.3100	1.3100	241.96	241.96	241.96	241.96
7.00 ft	0.1424	0.1424	0.1424	0.1424	1.2141	1.2141	1.2141	1.2141	299.18	299.18	299.18	299.18
9.00 ft	0.1695	0.1695	0.1695	0.1695	1.1252	1.1252	1.1252	1.1252	330.10	330.10	330.10	330.10
11.00 ft	0.1896	0.1896	0.1896	0.1896	1.0438	1.0438	1.0438	1.0438	338.89	338.89	338.89	338.89
13.00 ft	0.2036	0.2036	0.2036	0.2036	0.9699	0.9699	0.9699	0.9699	331.46	331.46	331.46	331.46
15.00 ft	0.2124	0.2124	0.2124	0.2124	0.9032	0.9032	0.9032	0.9032	313.54	313.54	313.54	313.54
17.00 ft	0.2171	0.2171	0.2171	0.2171	0.8430	0.8430	0.8430	0.8430	289.83	289.83	289.83	289.83
19.00 ft	0.2186	0.2186	0.2186	0.2186	0.7888	0.7888	0.7888	0.7888	263.70	263.70	263.70	263.70
21.00 ft	0.2177	0.2177	0.2177	0.2177	0.7400	0.7400	0.7400	0.7400	237.38	237.38	237.38	237.38
23.00 ft	0.2150	0.2150	0.2150	0.2150	0.6960	0.6960	0.6960	0.6960	212.22	212.22	212.22	212.22
25.00 ft	0.2112	0.2112	0.2112	0.2112	0.6563	0.6563	0.6563	0.6563	188.93	188.93	188.93	188.93
27.00 ft	0.2066	0.2066	0.2066	0.2066	0.6202	0.6202	0.6202	0.6202	167.83	167.83	167.83	167.83
29.00 ft	0.2014	0.2014	0.2014	0.2014	0.5875	0.5875	0.5875	0.5875	148.97	148.97	148.97	148.97
31.00 ft	0.1960	0.1960	0.1960	0.1960	0.5578	0.5578	0.5578	0.5578	132.27	132.27	132.27	132.27
33.00 ft	0.1904	0.1904	0.1904	0.1904	0.5306	0.5306	0.5306	0.5306	117.55	117.55	117.55	117.55
35.00 ft	0.1848	0.1848	0.1848	0.1848	0.5057	0.5057	0.5057	0.5057	104.64	104.64	104.64	104.64
37.00 ft	0.1793	0.1793	0.1793	0.1793	0.4829	0.4829	0.4829	0.4829	93.32	93.32	93.32	93.32
39.00 ft	0.1739	0.1739	0.1739	0.1739	0.4619	0.4619	0.4619	0.4619	83.41	83.41	83.41	83.41
41.00 ft	0.1686	0.1686	0.1686	0.1686	0.4425	0.4425	0.4425	0.4425	74.72	74.72	74.72	74.72

Note: The following two pages contain numerical and graphical representations of these loads over a selected area.

Gannett Fleming Engineers and Architects, P.C.
Excellence Delivered As Promised

Calc:	CTM	Date:	10/3/2014
Check:	VDT	Date:	11/7/2014
Update:	SNH	Date:	11/19/2014
Verified:	VDT	Date:	11/19/2014

Numerical Summary of Forces

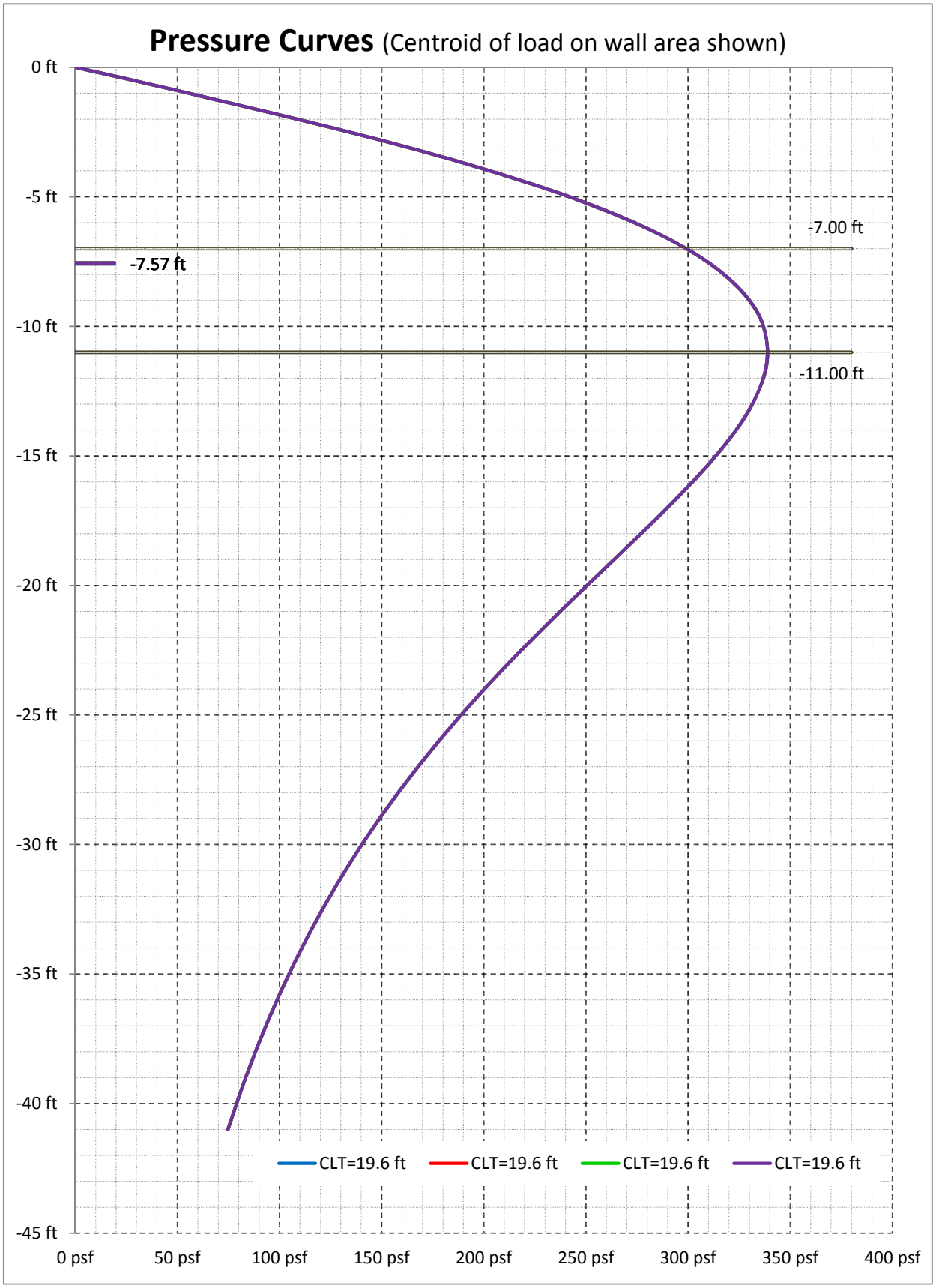
To gain total forces, the area under the graph is required. The area under the graph for the considered depth is divided by the total considered depth to arrive at a uniformly applied lateral pressure on any surface in question. The total lateral force is the total area under the graph. Simpsons Rule is used to arrive at the total area under the graph (median value force over a height is used)

	Values at CLT Distances			
Data Below is intended to give summary values that can be used in the design of walls and surfaces affected by the surcharge. A graphical output is on the next page	CLT=19.6 ft	CLT=19.6 ft	CLT=19.6 ft	CLT=19.6 ft
Top of Wall (Min. Depth Considered): 7.00 ft	299.2 psf	299.2 psf	299.2 psf	299.2 psf
Bottom of Wall (Max. Depth Considered): 11.00 ft	338.9 psf	338.9 psf	338.9 psf	338.9 psf

The bottom of the exposed wall face is 11.00 feet below the bottom of the tie. The top of the fill behind the wall (applying lateral pressure) is 7.00 feet below the bottom of the tie. The height of exposed wall face with surcharge pressure applied is 4.00 feet.

Maximum Applied Lateral Pressure on limits of the wall				
Maximum Applied Lateral Pressure	338.9 psf	338.9 psf	338.9 psf	338.9 psf
Total Applied Lateral Pressure on limits of the wall *(as an equivilant point load)*				
Total Lateral Load (Per foot)	1.60 klf	1.60 klf	1.60 klf	1.60 klf
Centroid of Total Lateral Load (Distance down from zero)	7.57 ft	7.57 ft	7.57 ft	7.57 ft
Total Applied Moment due to Surcharge *(assumes fixed at max depth)*	5.5 k-ft	5.5 k-ft	5.5 k-ft	5.5 k-ft
Uniformly Applied Lateral Pressure on limits of the wall (For Lateral Stability) *(Total Applied Lateral Pressure **Thrust Controls;** Total lateral moment is not correct)*				
Uniformly Applied Lateral Load	399.4 psf	399.4 psf	399.4 psf	399.4 psf
Moment due to Uniform Load *(assumes fixed at max depth)*	3.2 k-ft	3.2 k-ft	3.2 k-ft	3.2 k-ft
Difference in moment from that of the Total Point Load	-42%	-42%	-42%	-42%
Uniformly Applied Lateral Pressure on limits of the wall (For Moments/Deflectoins) *(Total Applied Lateral Pressure **Moment Controls;** Total lateral thrust is not correct)*				
Uniformly Applied Lateral Load	685.0 psf	685.0 psf	685.0 psf	685.0 psf
Moment due to Uniform Load *(assumes fixed at max depth)*	5.5 k-ft	5.5 k-ft	5.5 k-ft	5.5 k-ft
Difference in moment from that of the Total Point Load	0%	0%	0%	0%
Total of Uniformly Applied Lateral Load	2.74 klf	2.74 klf	2.74 klf	2.74 klf
Difference in Load from that of the Total Point Load	72%	72%	72%	72%

Total Uniform Load to Approximate Dual Tracks on Wall	**1369.9 psf**


Pressure Curves (Centroid of load on wall area shown)
0 ft
-5 ft
-10 ft
-15 ft
-20 ft
-25 ft
-30 ft
-35 ft
-40 ft
-45 ft
-7.00 ft
-7.57 ft
-11.00 ft
CLT=19.6 ft
CLT=19.6 ft
CLT=19.6 ft
CLT=19.6 ft
0 psf
50 psf
100 psf
150 psf
200 psf
250 psf
300 psf
350 psf
400 psf

Calc:	CTM	Date:	10/3/2014
Check:	VDT	Date:	11/7/2014
Update:	SNH	Date:	11/19/2014
Verified:	VDT	Date:	11/19/2014

L-Pile Loads on a 30" Wide Shaft should be entered as follows (depth is inches from pile head)

Shaft Width: 12.0 in
1.000 ft

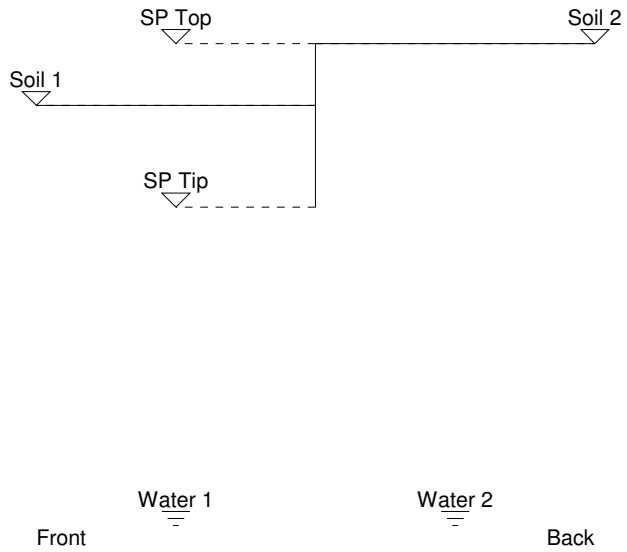
Depth	Depth	Last CLT column Lateral Load
0.00 ft	-7.000 ft	0.000 ksf
1.00 ft	-6.000 ft	0.055 ksf
3.00 ft	-4.000 ft	0.159 ksf
5.00 ft	-2.000 ft	0.242 ksf
7.00 ft	0.000 ft	0.299 ksf
9.00 ft	2.000 ft	0.330 ksf
11.00 ft	4.000 ft	0.339 ksf
15.00 ft	8.000 ft	0.314 ksf
31.00 ft	24.000 ft	0.132 ksf
41.00 ft	34.000 ft	0.075 ksf

Sheet Pile Design According to Blum-Method

Project Name:	HAM-PROSSER
Date:	11/19/2014
Author:	CTM
Company:	
Comment:	Unbraced section at SW Wingwall

Geodata

	Unit
Sheet Pile Top Level [ft]	0.000
Sheet Pile Tip Level [ft]	11.879
Soil Level in Front [ft]	4.500
Soil Level behind [ft]	0.000
Anchorlevel [ft]	0.000
Water Level in Front [ft]	34.000
Water Level behind [ft]	34.000
Soil Surface Inclination in Front [Deg]	0.000
Soil Surface Inclination behind [Deg]	0.000
Caquot Surcharge in Front [kip/ft2]	0.000
Caquot Surcharge behind [kip/ft2]	0.000
Anchor Inclination [Deg]	0.000
Earth Support	Cantilever



Soil Layers

Layers in Front

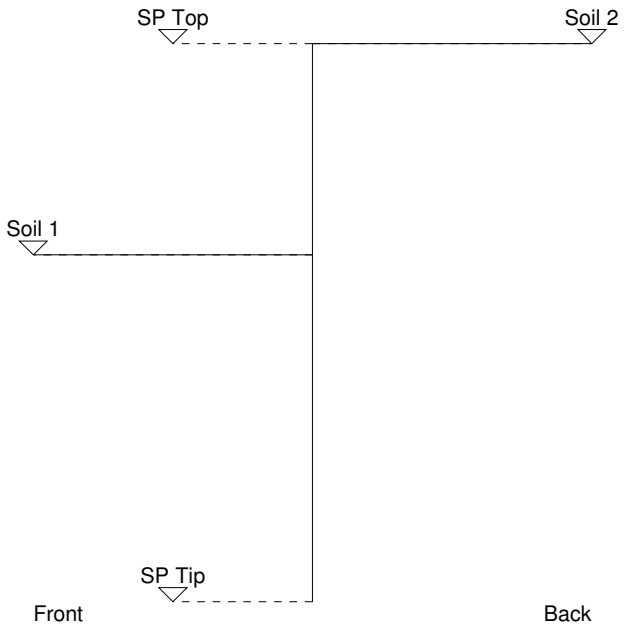
	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kph	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	100.000	0.125	0.063	3.120	31.000	0.000	0.000

Layers behind

	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kph	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	100.000	0.125	0.063	0.330	31.000	0.000	0.000

Boussinesq

	Distance Wall [ft]	Width Surcharge [ft]	Depth Surcharge [ft]	Surcharge [kip/ft2]

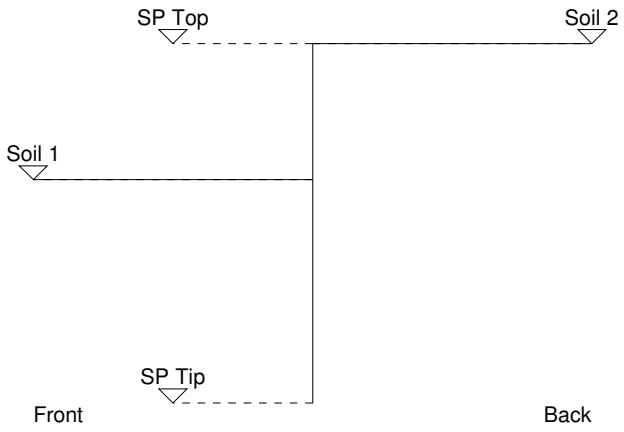


Userdefined Pressures

	Pressure Top [kip/ft2]	Pressure Tip [kip/ft2]	Depth Top [ft]	Depth Tip [ft]
Strip 1	0.000	0.055	0.000	1.000
Strip 2	0.055	0.159	1.000	3.000
Strip 3	0.159	0.242	3.000	5.000
Strip 4	0.242	0.299	5.000	7.000
Strip 5	0.299	0.330	7.000	9.000
Strip 6	0.330	0.339	9.000	11.000
Strip 7	0.339	0.313	11.000	15.000
Strip 8	0.313	0.132	15.000	31.000
Strip 9	0.132	0.074	31.000	41.000

Concentrated Forces

	Horiz. Component [kip/ft]	Vert. Component [kip/ft]	Depth Horiz. Comp. [ft]



Pile Section

Name	AZ 12-700
Inertia [in4/ft]	138.254
Modulus [in3/ft]	22.413
Area [in2/ft]	5.821
Mass [lbs/ft2]	19.806
Steel Grade [lb/in2]	39000.000
Requested Safety	1.500

All Values

Depth [ft]	Deflection [ft]	Rotation [Rad]	Cross Force [kip/ft]	Moment [kipft/ft]	Total Pressure [kip/ft2]	Earth Pressure in Front [kip/ft2]	behind [kip/ft2]	Water Pressure [kip/ft2]	Userdefined Pressure [kip/ft2]
0.000	-0.008	-0.001	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.820	-0.007	-0.001	0.032	0.009	0.079	0.000	0.034	0.000	0.045
0.820	-0.007	-0.001	0.032	0.009	0.079	0.000	0.034	0.000	0.045
1.000	-0.007	-0.001	0.048	0.016	0.096	0.000	0.041	0.000	0.055
1.000	-0.007	-0.001	0.048	0.016	0.096	0.000	0.041	0.000	0.055
1.820	-0.006	-0.001	0.158	0.096	0.173	0.000	0.075	0.000	0.098
1.820	-0.006	-0.001	0.158	0.096	0.173	0.000	0.075	0.000	0.098
2.640	-0.005	-0.001	0.331	0.293	0.249	0.000	0.109	0.000	0.140
2.640	-0.005	-0.001	0.331	0.293	0.249	0.000	0.109	0.000	0.140
3.000	-0.005	-0.001	0.427	0.429	0.283	0.000	0.124	0.000	0.159
3.000	-0.005	-0.001	0.427	0.429	0.283	0.000	0.124	0.000	0.159
3.820	-0.004	-0.001	0.687	0.882	0.351	0.000	0.158	0.000	0.193
3.820	-0.004	-0.001	0.687	0.882	0.351	0.000	0.158	0.000	0.193
4.500	-0.004	-0.001	0.944	1.434	0.407	0.000	0.186	0.000	0.221
4.500	-0.004	-0.001	0.944	1.434	0.407	0.000	0.186	0.000	0.221
5.000	-0.003	-0.001	1.109	1.951	0.253	-0.195	0.206	0.000	0.242
5.000	-0.003	-0.001	1.109	1.951	0.253	-0.195	0.206	0.000	0.242
5.820	-0.002	-0.001	1.209	2.917	-0.009	-0.515	0.240	0.000	0.265
5.820	-0.002	-0.001	1.209	2.917	-0.009	-0.515	0.240	0.000	0.265
6.640	-0.002	-0.001	1.094	3.876	-0.272	-0.835	0.274	0.000	0.289
6.640	-0.002	-0.001	1.094	3.876	-0.272	-0.835	0.274	0.000	0.289
7.000	-0.002	-0.001	0.975	4.249	-0.387	-0.975	0.289	0.000	0.299
7.000	-0.002	-0.001	0.975	4.249	-0.387	-0.975	0.289	0.000	0.299
7.820	-0.001	0.000	0.546	4.888	-0.661	-1.295	0.323	0.000	0.312
7.820	-0.001	0.000	0.546	4.888	-0.661	-1.295	0.323	0.000	0.312
8.640	-0.001	0.000	-0.108	5.083	-0.934	-1.615	0.356	0.000	0.324
8.640	-0.001	0.000	-0.108	5.083	-0.934	-1.615	0.356	0.000	0.324
9.000	0.000	0.000	-0.466	4.981	-1.054	-1.755	0.371	0.000	0.330
9.000	0.000	0.000	-0.466	4.981	-1.054	-1.755	0.371	0.000	0.330
9.820	0.000	0.000	-1.446	4.213	-1.336	-2.075	0.405	0.000	0.334
9.820	0.000	0.000	-1.446	4.213	-1.336	-2.075	0.405	0.000	0.334
10.640	0.000	0.000	-2.657	2.546	-1.618	-2.395	0.439	0.000	0.337
10.640	0.000	0.000	-2.657	2.546	-1.618	-2.395	0.439	0.000	0.337
11.000	0.000	0.000	-3.262	1.484	-1.742	-2.535	0.454	0.000	0.339
11.000	0.000	0.000	-3.262	1.484	-1.742	-2.535	0.454	0.000	0.339
11.427	0.000	0.000	-4.037	-0.071	-1.894	-2.701	0.471	0.000	0.336

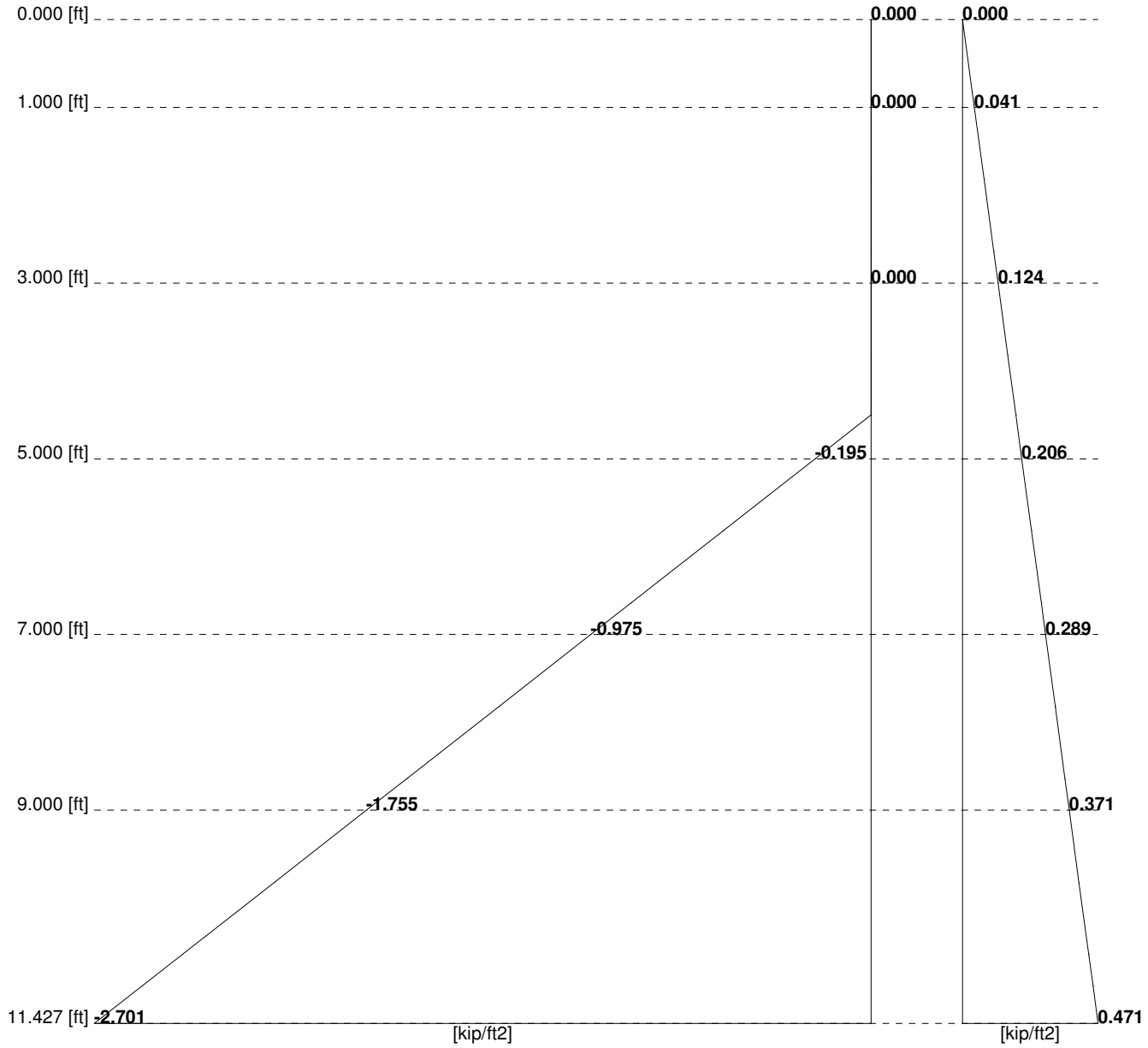
Extremal Values

	z Min [ft]	Min	z Max [ft]	Max
Deflection [ft]	0.000	-0.008	11.427	0.000
Cross Force [kip/ft]	11.427	-4.037	5.820	1.209
Moment [kipft/ft]	11.459	-0.204	8.542	5.089

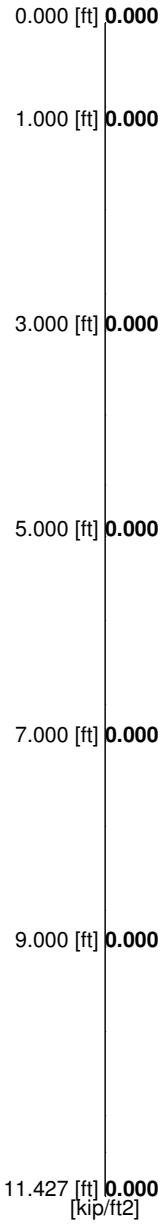
Pile Check

		Depth [ft]
Name	AZ 12-700	
Inertia [in4/ft]	138.254	
Modulus [in3/ft]	22.413	
Area [in2/ft]	5.821	
Mass [lbs/ft2]	19.806	
Steel Grade [lb/in2]	39000.000	
Minimal Moment [kipft/ft]	-0.204	11.459
Maxmimal Moment [kipft/ft]	5.089	8.542
Normal Forces at Max. Moment [kip/ft]	0.000	11.459
Normal Forces at Min. Moment [kip/ft]	0.000	8.542
Deflection at Min. Moment [ft]	0.000	11.459
Deflection at Max. Moment [ft]	-0.001	8.542
Min. Stress at Min. Moment [lb/in2]	-109.266	11.459
Max. Stress at Min. Moment [lb/in2]	109.266	11.459
Min. Stress at Max. Moment [lb/in2]	-2724.705	8.542
Max. Stress at Max. Moment [lb/in2]	2724.705	8.542
Safety > Req. Safety = 1.500	14.313	
Sheet Pile Top Level [ft]	0.000	
Sheet Pile Tip Level [ft]	11.879	
Sheet Pile Length [ft]	11.879	
Included OverLength [ft]	0.452	
Vertical Equilibrium [kip/ft]	0.000	
Anchor Force (horiz.) [kip/ft]	0.000	

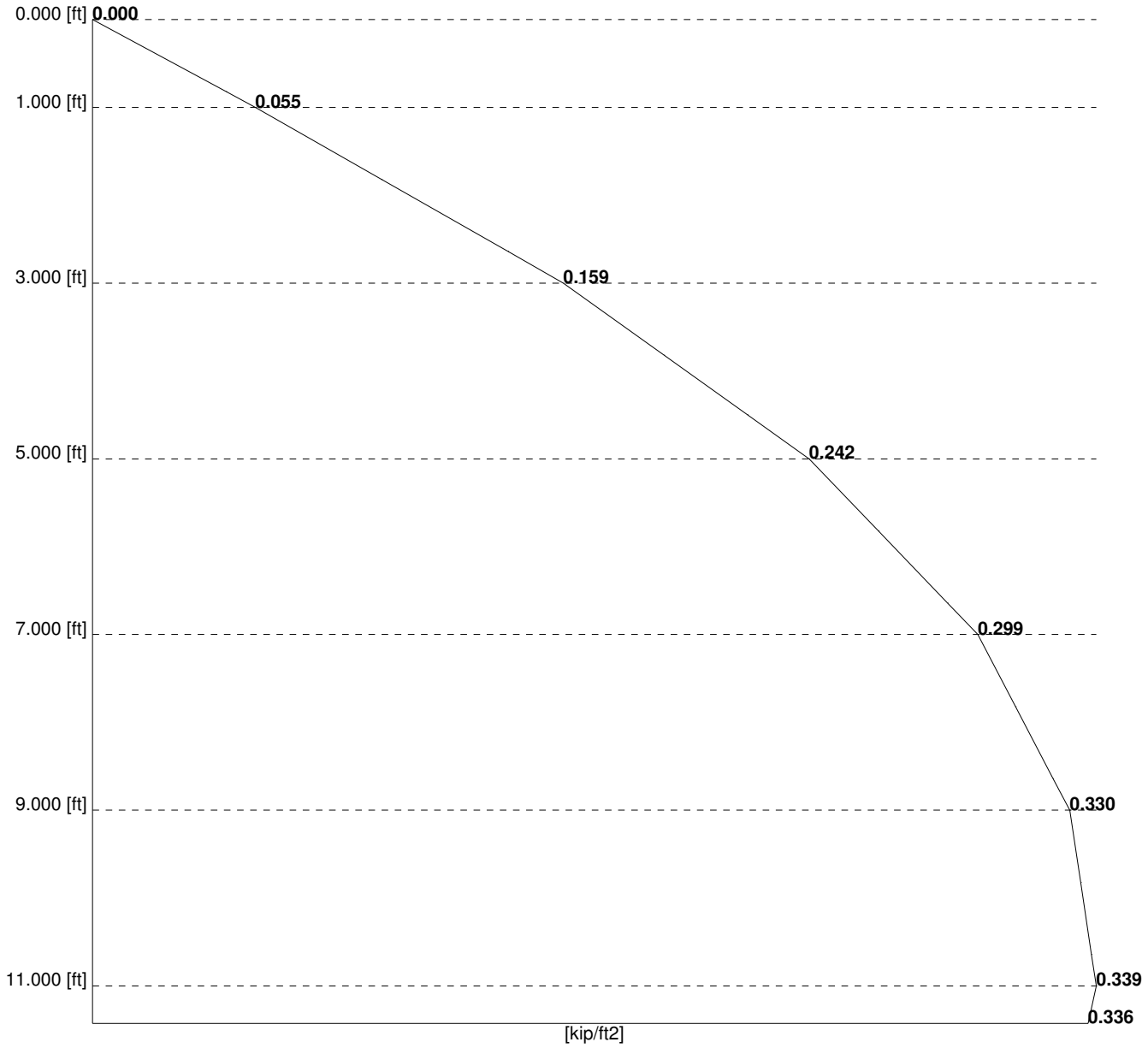
Earth Pressure Diagram



Water Pressure Diagram

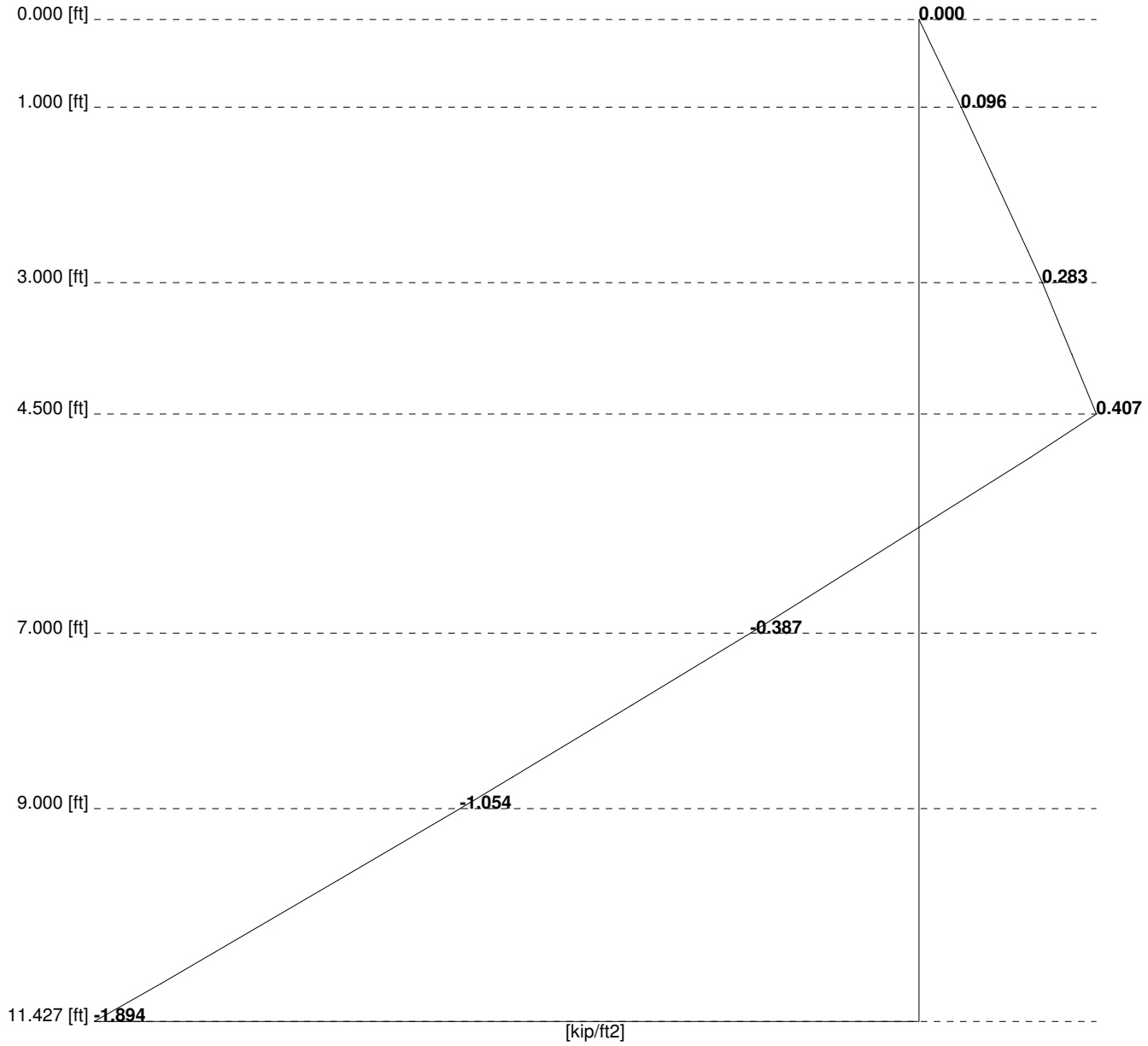


Userdefined Pressure Diagram

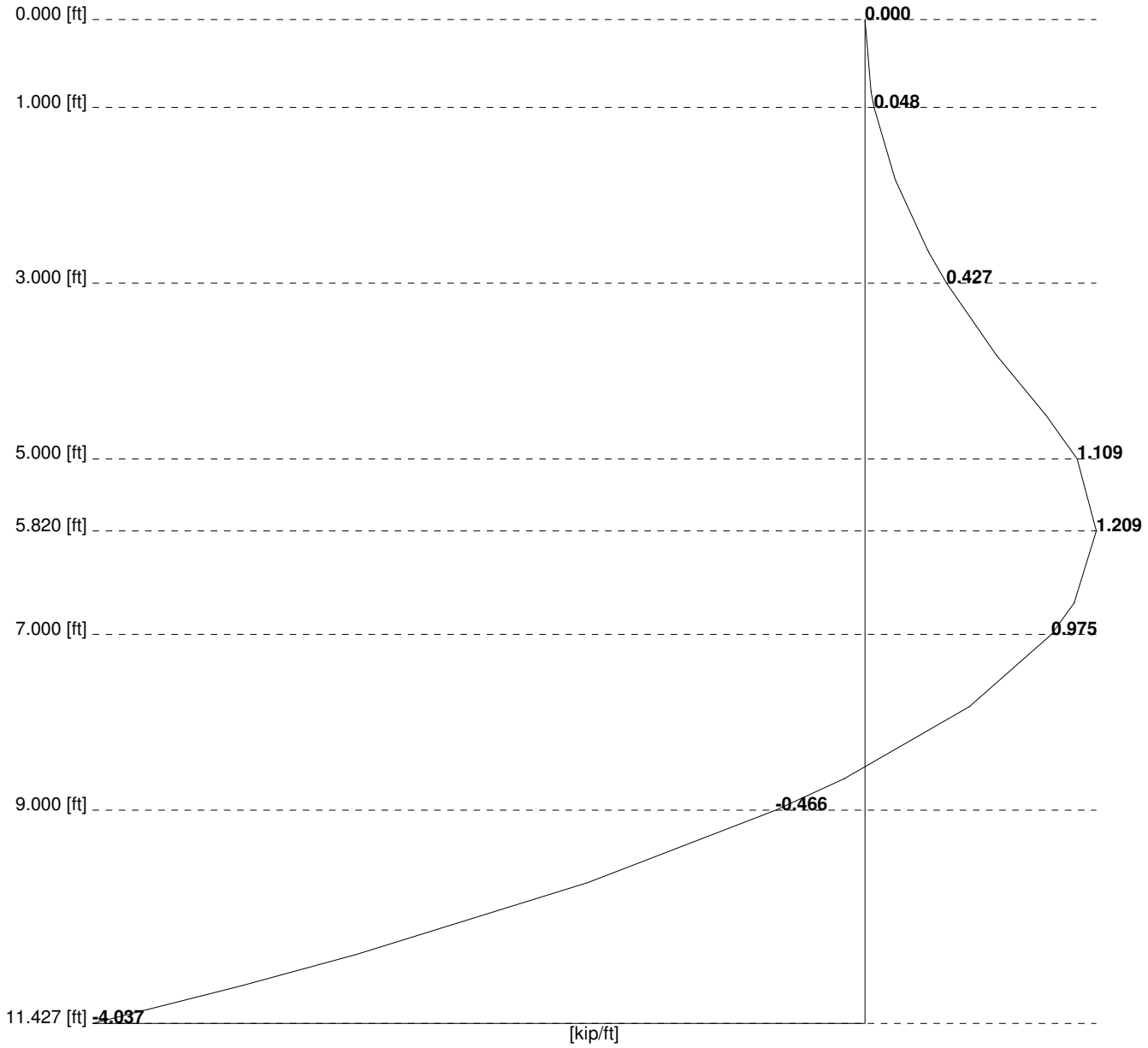


Boussinesq Diagram

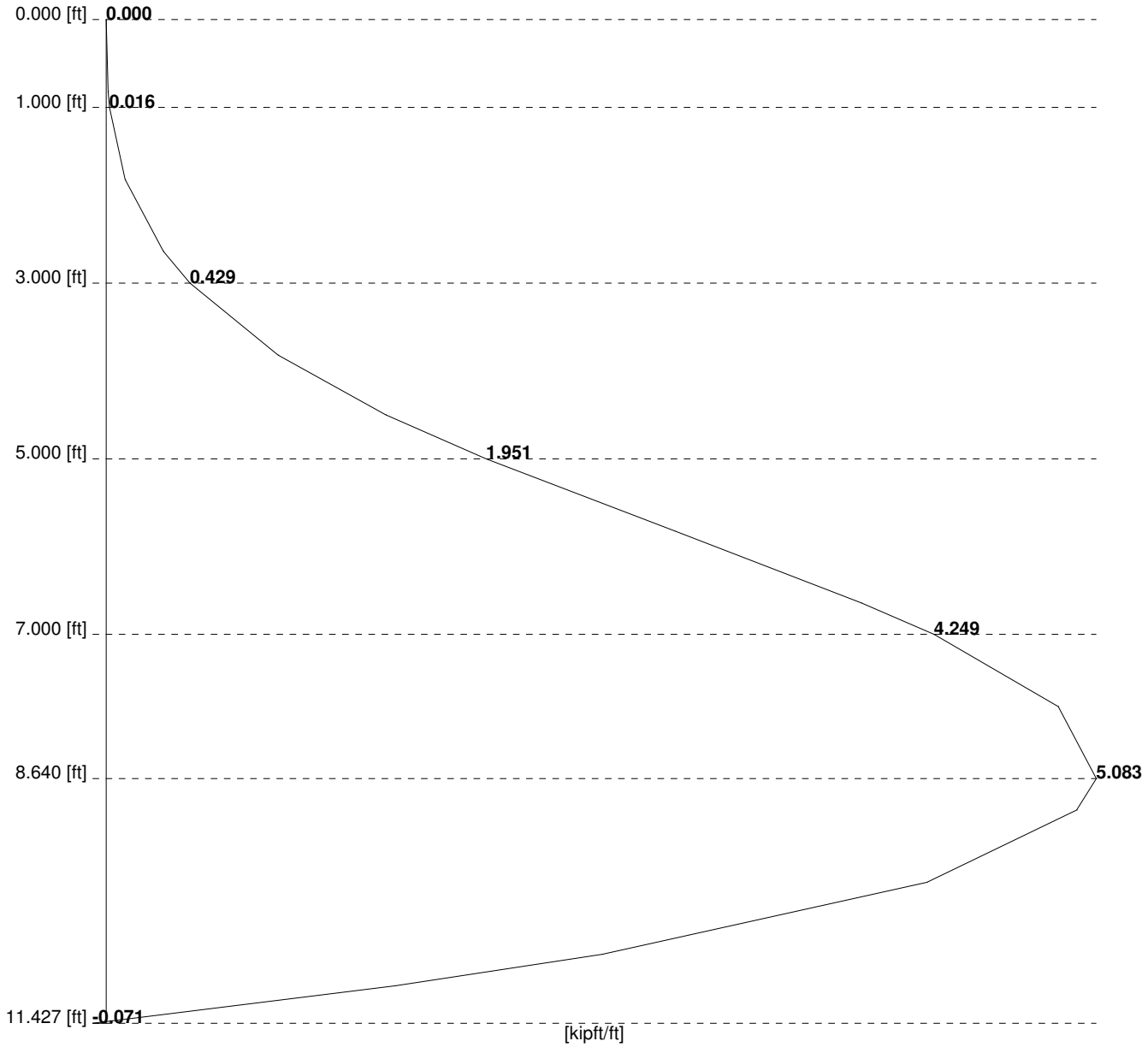
Total Pressure Diagram



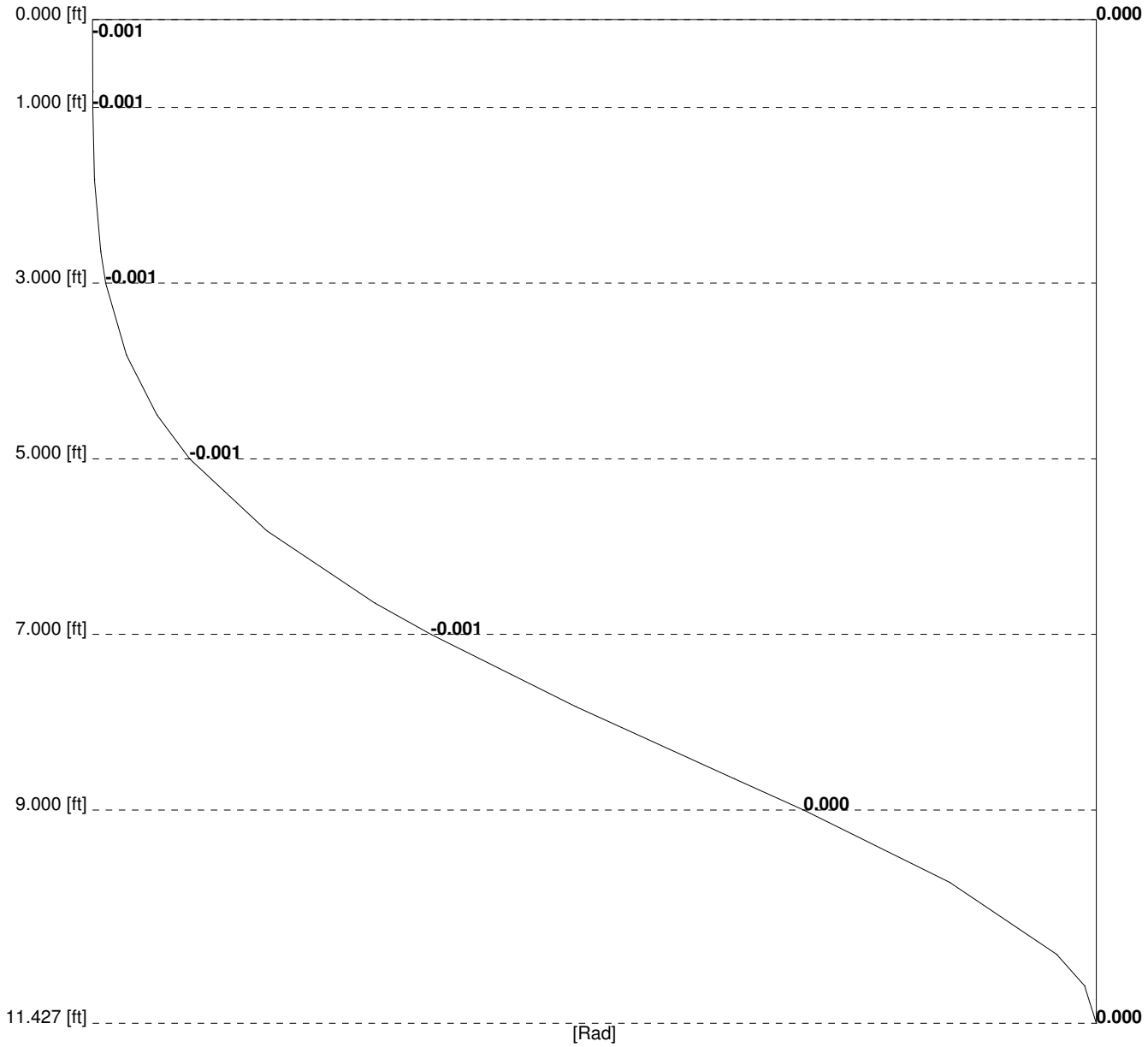
Cross Force Diagram



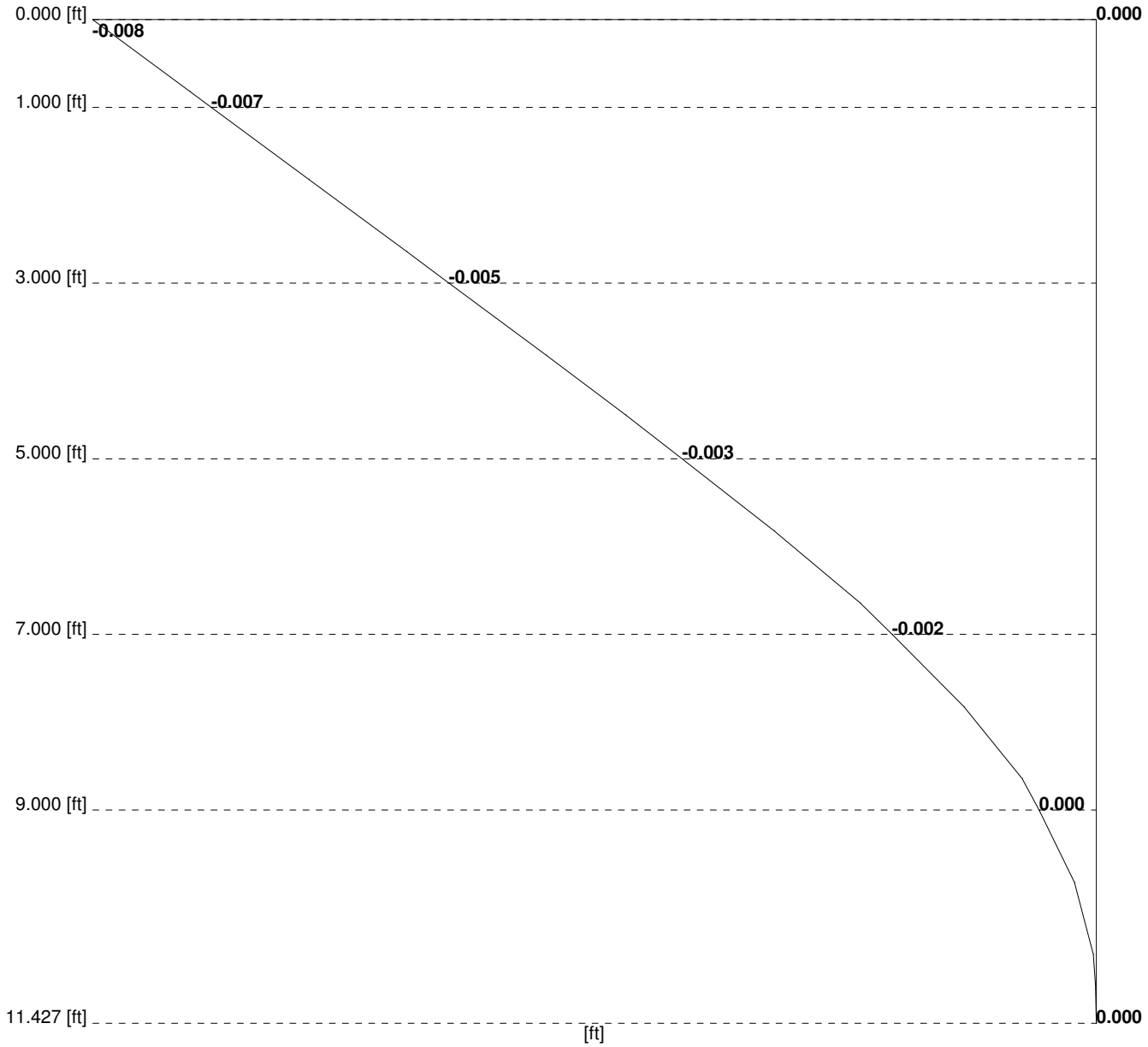
Moment Diagram



Rotation Diagram



Deflection Diagram



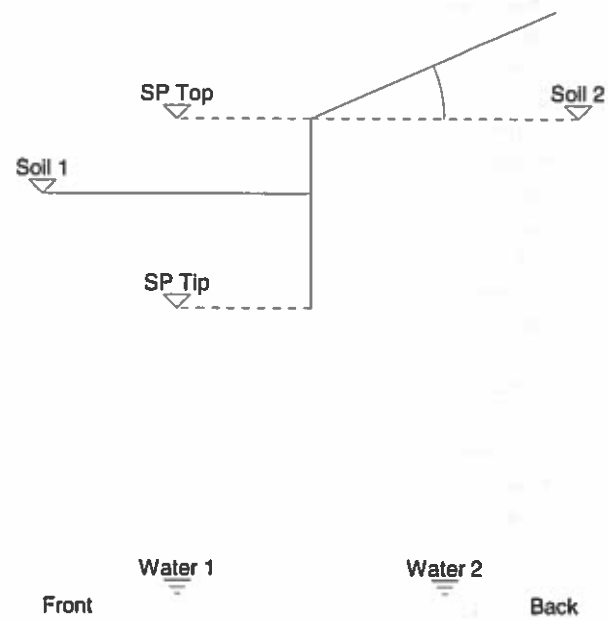
Sheet Pile Design According to Blum-Method

Design of Unbraced Shoring at the Southease Wingwall

Project Name:
Date: 5/9/2016
Author:
Company:
Comment:
Project Name:
Date: 5/9/2016
Author:
Company:
Comment:
Project Name:
Date: 5/9/2016
Author:
Company:
Comment:
Project Name:
Date: 5/9/2016
Author:
Company:
Comment:
Project Name:
Date: 5/9/2016
Author:
Company:
Comment:
Project Name:
Date: 5/9/2016
Author:
Company:
Comment:

Geodata

	Unit
Sheet Pile Top Level [ft]	0.000
Sheet Pile Tip Level [ft]	16.423
Soil Level in Front [ft]	6.500
Soil Level behind [ft]	0.000
Anchorlevel [ft]	0.000
Water Level in Front [ft]	40.000
Water Level behind [ft]	40.000
Soil Surface Inclination in Front [Deg]	0.000
Soil Surface Inclination behind [Deg]	23.500
Caquot Surcharge in Front [kip/ft2]	0.000
Caquot Surcharge behind [kip/ft2]	0.000
Anchor Inclination [Deg]	0.000
Earth Support	Cantilever



Soil Layers

Layers in Front

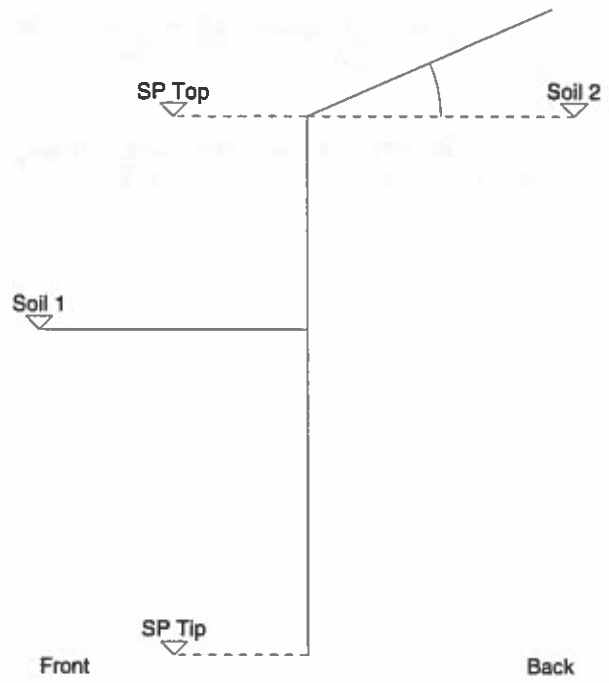
	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kph	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	30.000	0.125	0.145	1.932	31.500	15.500	0.000

Layers behind

	Layer Tip [ft]	Density Moist [kip/ft3]	Density Submerged [kip/ft3]	Kph	Phi [Deg]	Delta [Deg]	Cohesion [kip/ft2]
Layer 1	30.000	0.125	0.145	0.405	31.500	15.500	0.000

Boussinesq

	Distance Wall [ft]	Width Surcharge [ft]	Depth Surcharge [ft]	Surcharge [kip/ft2]

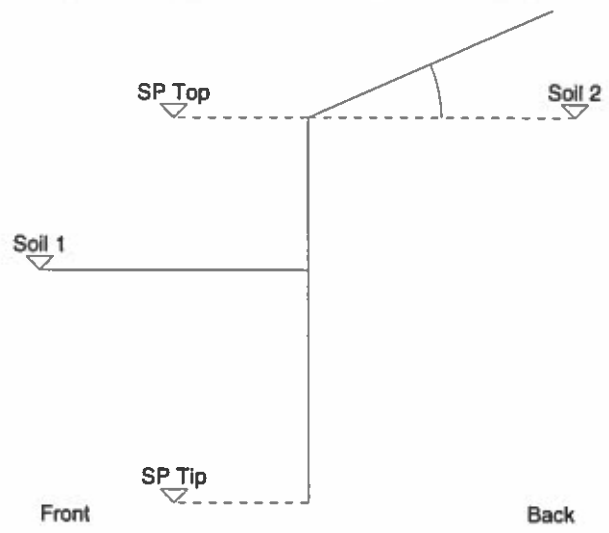


Userdefined Pressures

	Pressure Top [kip/ft2]	Pressure Tip [kip/ft2]	Depth Top [ft]	Depth Tip [ft]

Concentrated Forces

	Horiz. Component [kip/ft]	Vert. Component [kip/ft]	Depth Horiz. Comp. [ft]



Pile Section

Name	AZ 14-700
Inertia [in4/ft]	162.492
Modulus [in3/ft]	26.133
Area [in2/ft]	6.903
Mass [lbs/ft2]	23.492
Steel Grade [lb/in2]	34795.867
Requested Safety	1.500

All Values

Depth [ft]	Deflection [ft]	Rotation [Rad]	Cross Force [kip/ft]	Moment [kipft/ft]	Total Pressure [kip/ft2]	Earth Pressure in Front [kip/ft2]	behind [kip/ft2]	Water Pressure [kip/ft2]	Userdefined Pressure [kip/ft2]
0.000	-0.019	-0.002	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.820	-0.018	-0.002	0.017	0.005	0.042	0.000	0.042	0.000	0.000
0.820	-0.018	-0.002	0.017	0.005	0.042	0.000	0.042	0.000	0.000
1.640	-0.016	-0.002	0.068	0.037	0.083	0.000	0.083	0.000	0.000
1.640	-0.016	-0.002	0.068	0.037	0.083	0.000	0.083	0.000	0.000
2.461	-0.015	-0.002	0.153	0.126	0.125	0.000	0.125	0.000	0.000
2.461	-0.015	-0.002	0.153	0.126	0.125	0.000	0.125	0.000	0.000
3.281	-0.014	-0.002	0.273	0.298	0.166	0.000	0.166	0.000	0.000
3.281	-0.014	-0.002	0.273	0.298	0.166	0.000	0.166	0.000	0.000
4.101	-0.012	-0.002	0.426	0.582	0.208	0.000	0.208	0.000	0.000
4.101	-0.012	-0.002	0.426	0.582	0.208	0.000	0.208	0.000	0.000
4.921	-0.011	-0.002	0.613	1.006	0.249	0.000	0.249	0.000	0.000
4.921	-0.011	-0.002	0.613	1.006	0.249	0.000	0.249	0.000	0.000
5.742	-0.010	-0.002	0.835	1.598	0.291	0.000	0.291	0.000	0.000
5.742	-0.010	-0.002	0.835	1.598	0.291	0.000	0.291	0.000	0.000
6.500	-0.009	-0.002	1.070	2.319	0.329	0.000	0.329	0.000	0.000
6.500	-0.009	-0.002	1.070	2.319	0.329	0.000	0.329	0.000	0.000
7.320	-0.007	-0.001	1.276	3.289	0.173	-0.198	0.371	0.000	0.000
7.320	-0.007	-0.001	1.276	3.289	0.173	-0.198	0.371	0.000	0.000
8.140	-0.006	-0.001	1.353	4.377	0.016	-0.396	0.412	0.000	0.000
8.140	-0.006	-0.001	1.353	4.377	0.016	-0.396	0.412	0.000	0.000
8.961	-0.005	-0.001	1.302	5.475	-0.140	-0.594	0.454	0.000	0.000
8.961	-0.005	-0.001	1.302	5.475	-0.140	-0.594	0.454	0.000	0.000
9.781	-0.004	-0.001	1.123	6.478	-0.297	-0.792	0.495	0.000	0.000
9.781	-0.004	-0.001	1.123	6.478	-0.297	-0.792	0.495	0.000	0.000
10.601	-0.003	-0.001	0.815	7.282	-0.453	-0.990	0.537	0.000	0.000
10.601	-0.003	-0.001	0.815	7.282	-0.453	-0.990	0.537	0.000	0.000
11.421	-0.002	-0.001	0.379	7.781	-0.610	-1.189	0.579	0.000	0.000
11.421	-0.002	-0.001	0.379	7.781	-0.610	-1.189	0.579	0.000	0.000
12.242	-0.001	-0.001	-0.185	7.869	-0.767	-1.387	0.620	0.000	0.000
12.242	-0.001	-0.001	-0.185	7.869	-0.767	-1.387	0.620	0.000	0.000
13.062	-0.001	0.000	-0.878	7.442	-0.923	-1.585	0.662	0.000	0.000
13.062	-0.001	0.000	-0.878	7.442	-0.923	-1.585	0.662	0.000	0.000
13.882	0.000	0.000	-1.700	6.393	-1.080	-1.783	0.703	0.000	0.000
13.882	0.000	0.000	-1.700	6.393	-1.080	-1.783	0.703	0.000	0.000
14.702	0.000	0.000	-2.649	4.619	-1.236	-1.981	0.745	0.000	0.000
14.702	0.000	0.000	-2.649	4.619	-1.236	-1.981	0.745	0.000	0.000
15.522	0.000	0.000	-3.727	2.012	-1.393	-2.179	0.786	0.000	0.000
15.522	0.000	0.000	-3.727	2.012	-1.393	-2.179	0.786	0.000	0.000
16.047	0.000	0.000	-4.485	-0.141	-1.493	-2.306	0.813	0.000	0.000

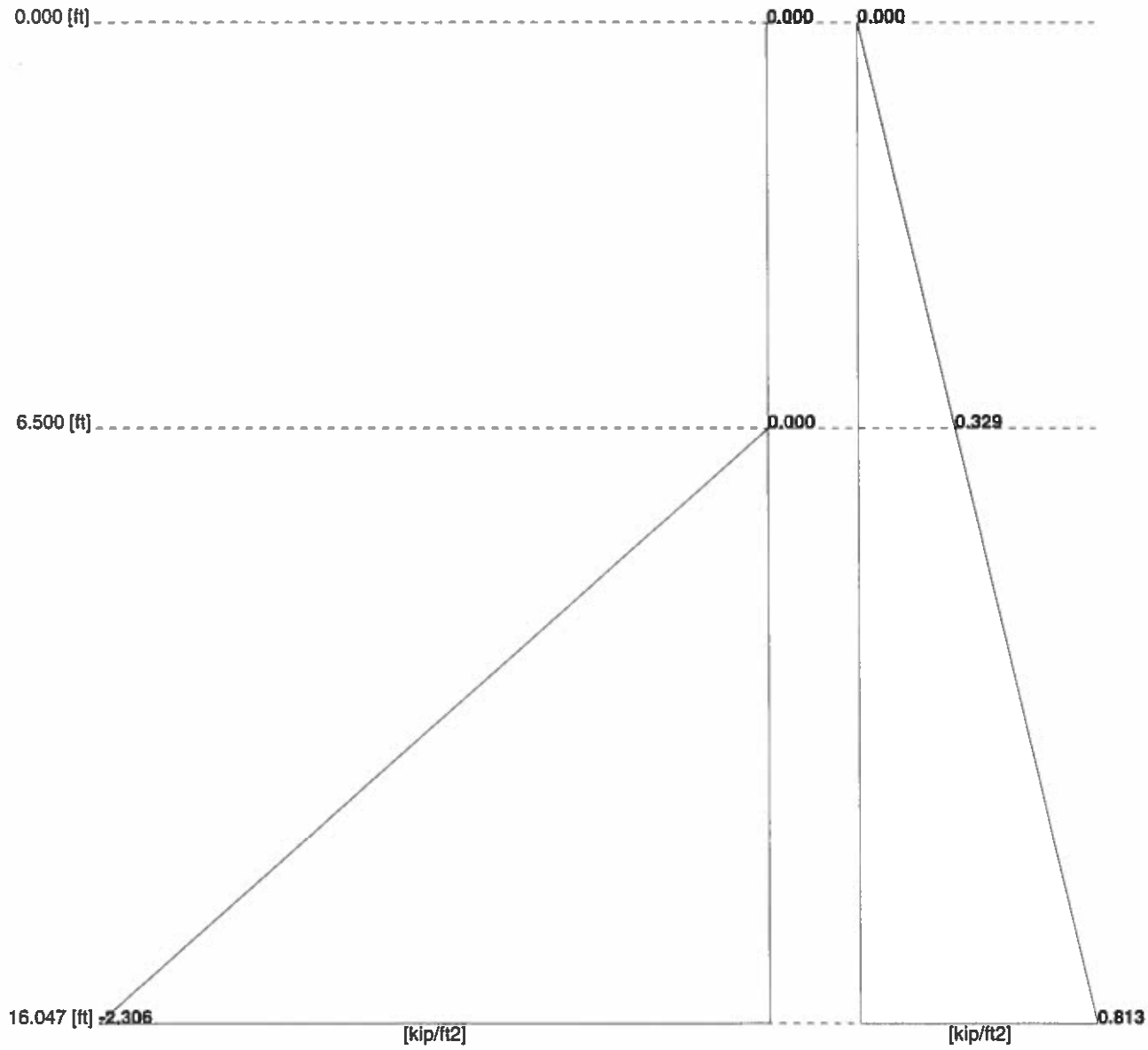
Extremal Values

	z Min [ft]	Min	z Max [ft]	Max
Deflection [ft]	0.000	-0.019	16.047	0.000
Cross Force [kip/ft]	16.047	-4.485	8.140	1.353
Moment [kipft/ft]	16.080	-0.289	12.012	7.892

Pile Check

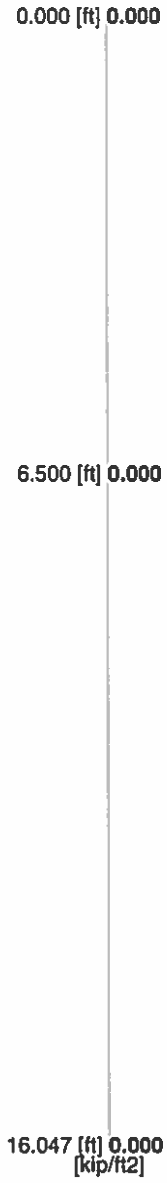
		Depth [ft]
Name	AZ 14-700	
Inertia [in4/ft]	162.492	
Modulus [in3/ft]	26.133	
Area [in2/ft]	6.903	
Mass [lbs/ft2]	23.492	
Steel Grade [lb/in2]	34795.867	
Minimal Moment [kipft/ft]	-0.289	16.080
Maxmimal Moment [kipft/ft]	7.892	12.012
Normal Forces at Max. Moment [kip/ft]	4.850	16.080
Normal Forces at Min. Moment [kip/ft]	2.157	12.012
Deflection at Min. Moment [ft]	0.000	16.080
Deflection at Max. Moment [ft]	-0.001	12.012
Min. Stress at Min. Moment [lb/in2]	570.086	16.080
Max. Stress at Min. Moment [lb/in2]	835.326	16.080
Min. Stress at Max. Moment [lb/in2]	-3312.493	12.012
Max. Stress at Max. Moment [lb/in2]	3937.343	12.012
Safety > Req. Safety = 1.500	8.837	
Sheet Pile Top Level [ft]	0.000	
Sheet Pile Tip Level [ft]	16.423	
Sheet Pile Length [ft]	16.423	
Included OverLength [ft]	0.376	
Vertical Equilibrium [kip/ft]	6.094	
Anchor Force (horiz.) [kip/ft]	0.000	

Earth Pressure Diagram



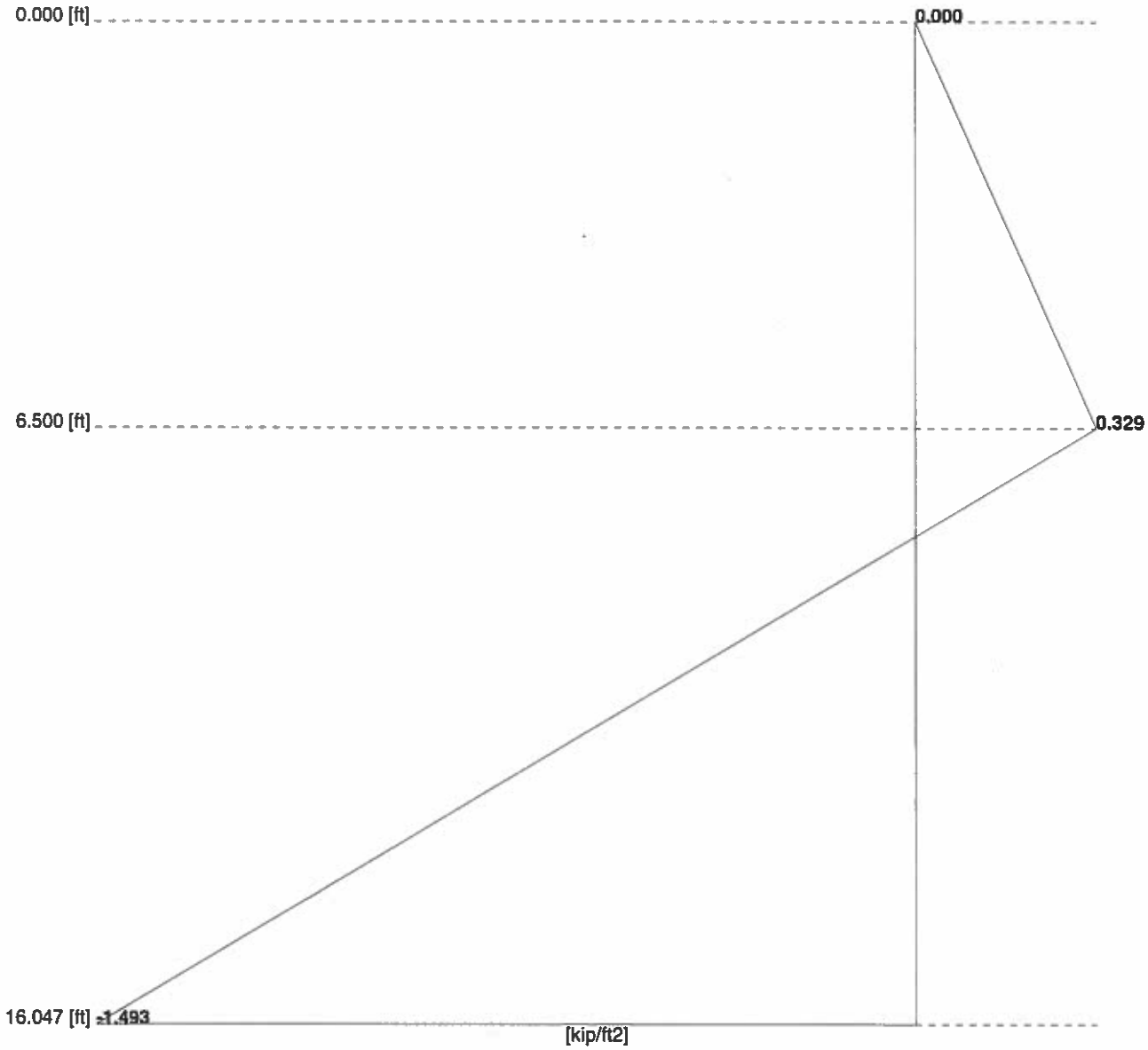
Water Pressure Diagram

Userdefined Pressure Diagram

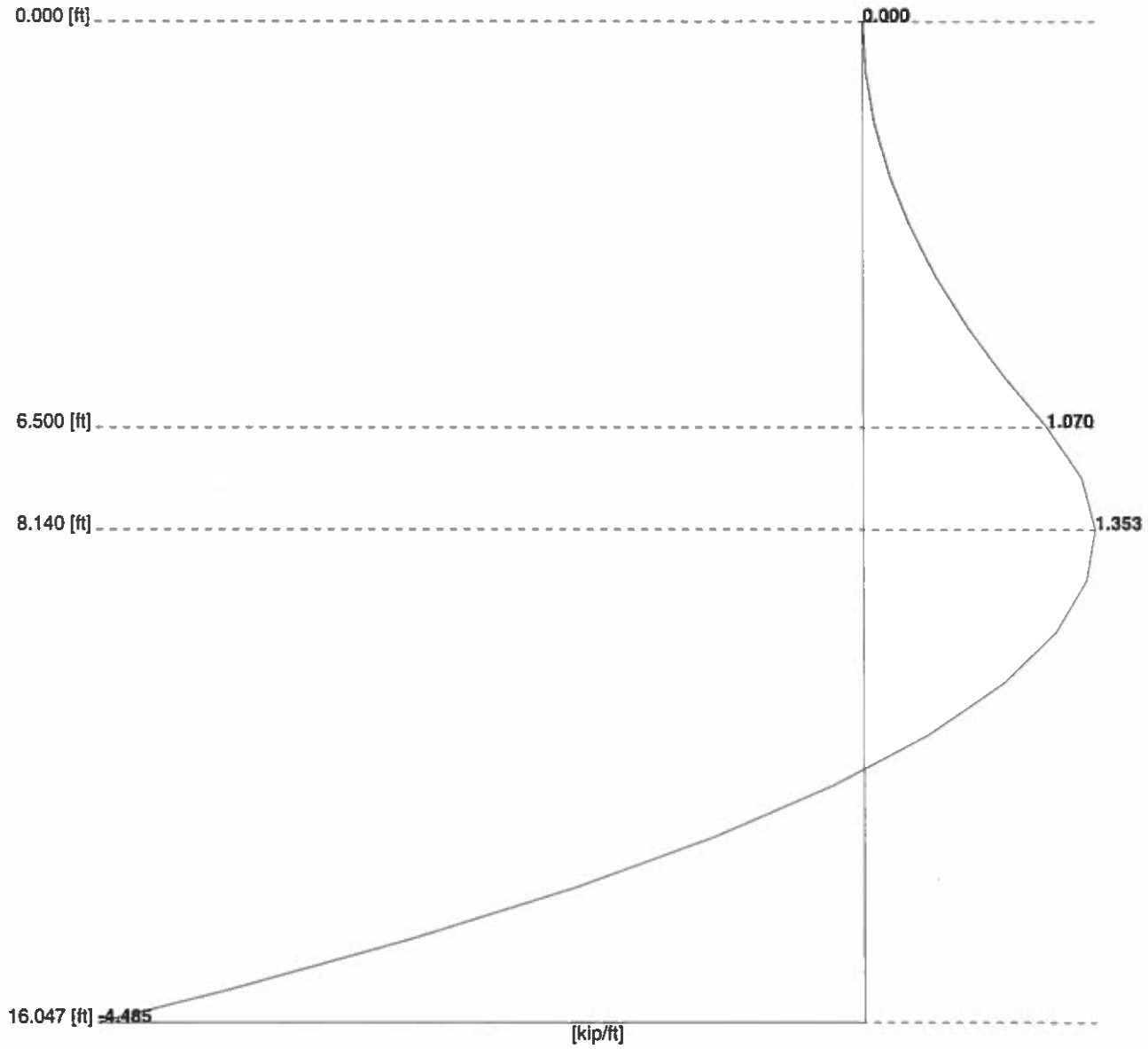


Boussinesq Diagram

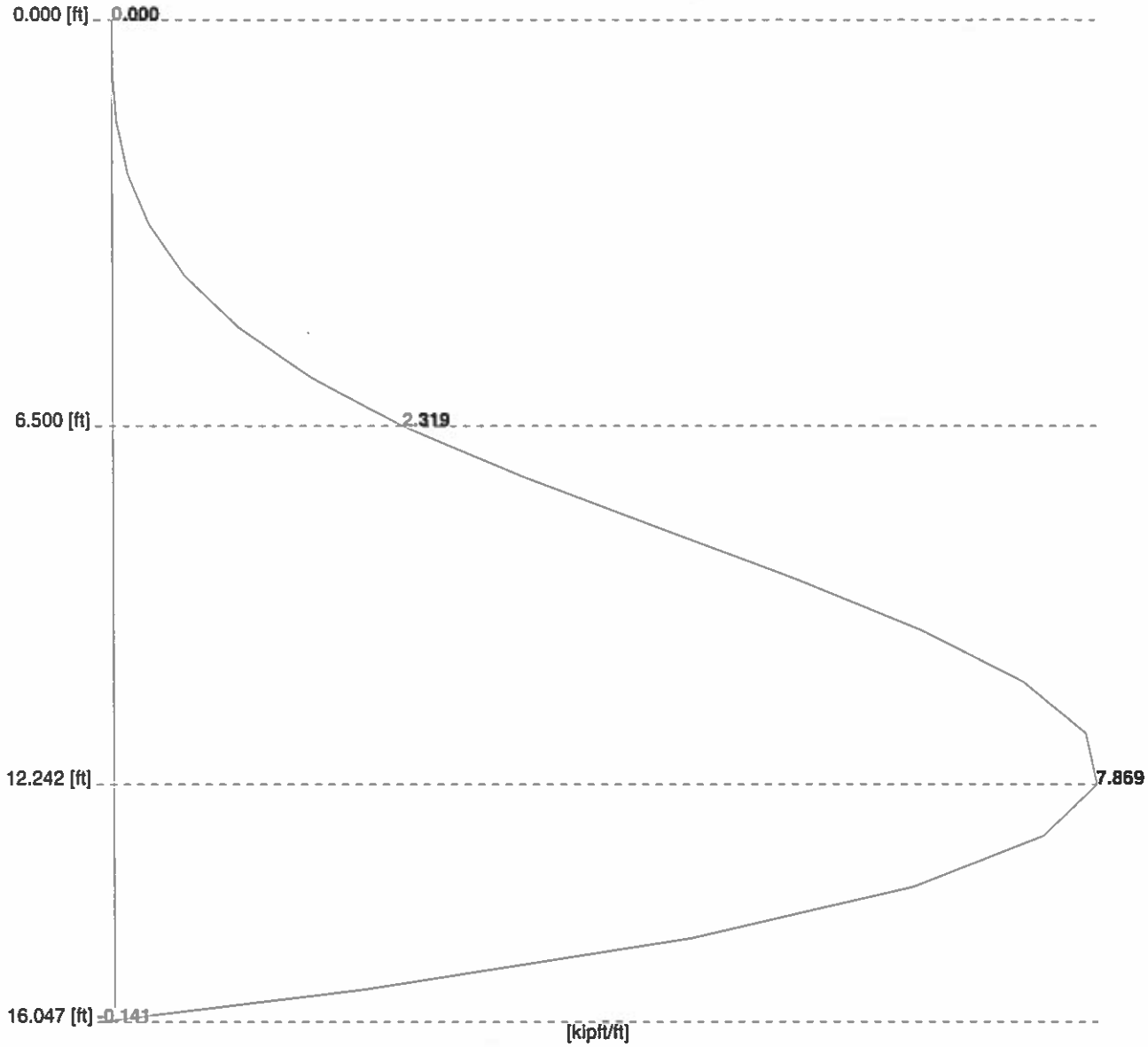
Total Pressure Diagram



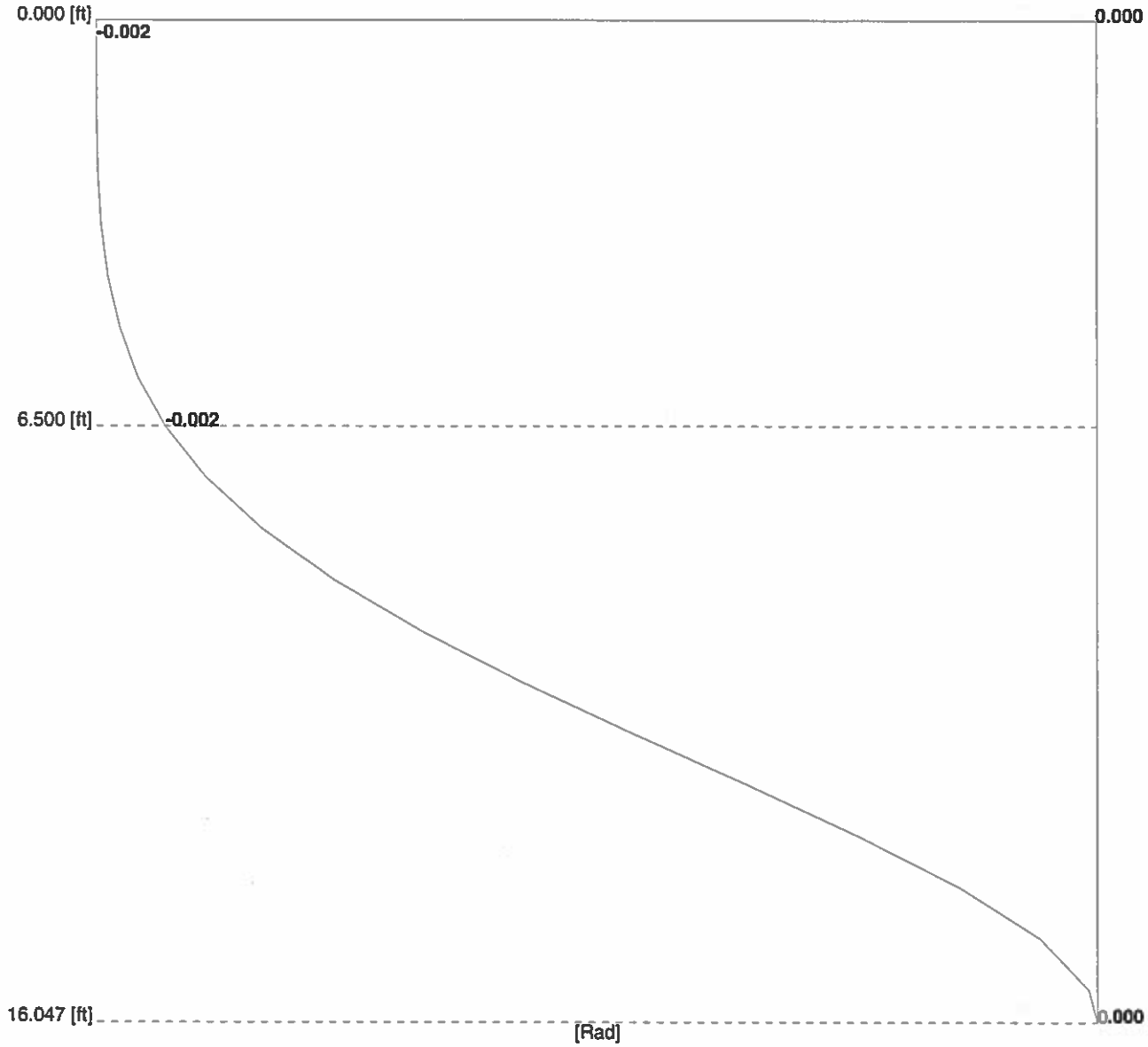
Cross Force Diagram



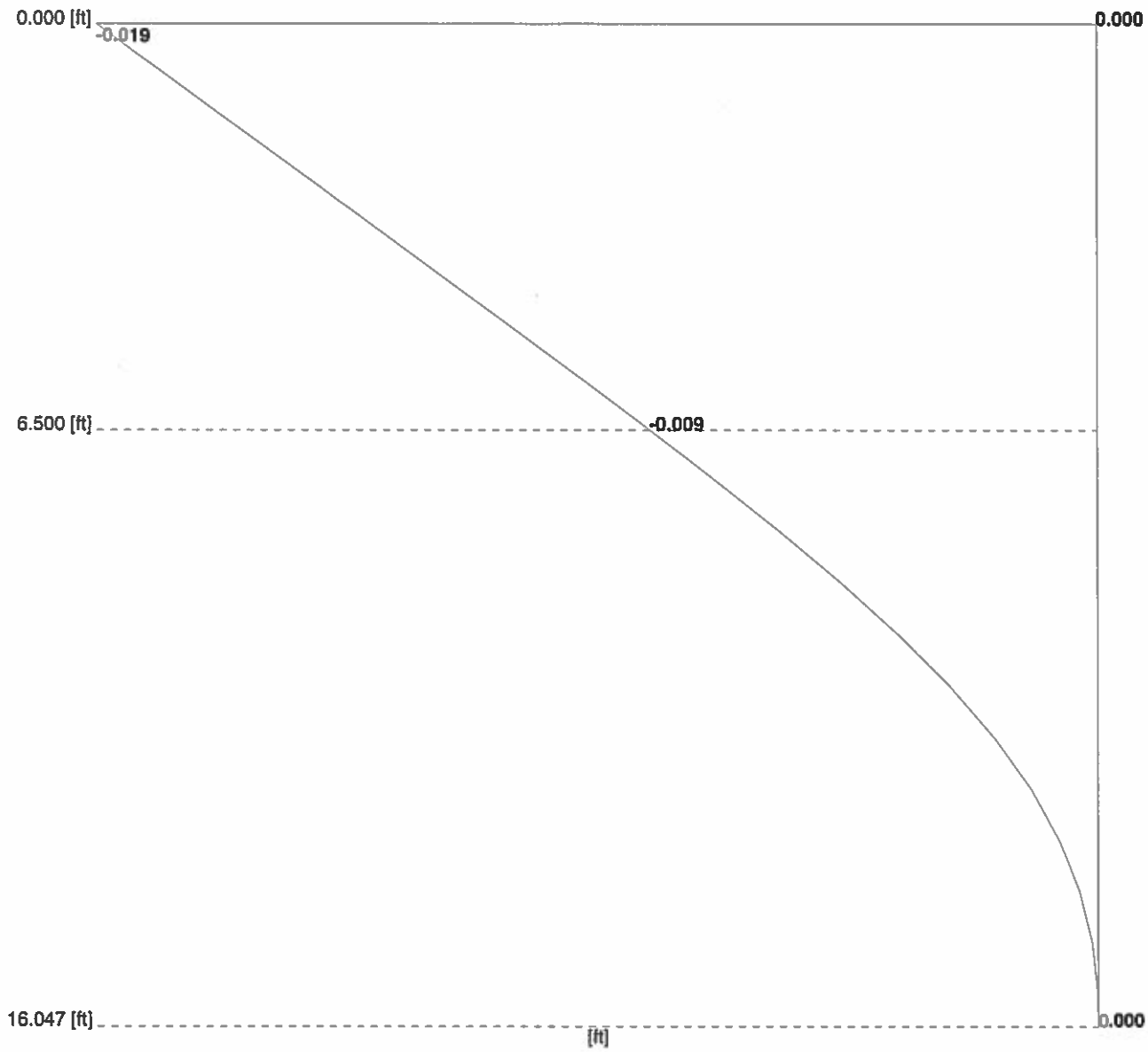
Moment Diagram



Rotation Diagram



Deflection Diagram



DESIGN CALCULATION COVER SHEET

Gannett Fleming
Engineers and Architects, P.C.
Excellence Delivered ***As Promised***

ISO 9001:2008 CERTIFIED

PROJECT: **ODOT/EMH&T: HAM-75-7.85**
BRIDGE NO: **HAM-75-PROS**
SUBJECT: ***Analysis***
Girder Connection Details
MATHCAD
FILE NAME(s): *NSRR Standard Connection Details*

ODOT PID NO: *77889*
EMH&T JOB NO:
GF JOB NO: 54682
CALC NO.: **HAM075_PROS_10**
REV NO.: **00**
SHEET: 1 OF 29

SUPERSEDES CALC NO: 00 (Stage 1 Calculation)

CALCULATION DESCRIPTION:

This will analyze all of the steel connections to the girder, such as end and intermediate diaphragms, stiffeners, and lower lateral bracing, both welded connections and bolted connections.

DESIGN BASIS OR REFERENCES:

NS Underpass Design Criteria
AREMA (2014) Chapter 15
HAM075_PROS_03
AISC Steel Construction Manual, Thirteenth Edition
BDM - ODOT Bridge Design Manual (2004) - Current Edition

MISCELLANEOUS NOTES:

RECORD OF ISSUES:

REV	DESCRIPTION	BY	DATE	VERIF'D	DATE
00	Stage 1 Calculation	VDK	04/16/14	EFD	05/29/14
01	Stage 3 Update	VDT	01/20/16	CTM	04/25/16

RECORD OF CHECK OF CURRENT CALCULATION VERSION

BY	DATE	CHKD	DATE	UPD.	DATE	VERIF'D	DATE
VDK	04/16/14	VDT	05/15/14	VDT	01/20/16	CTM	04/25/16

LOAD AND GEOMETRIC DATA

Design notes

- Variables with a box around them require review/user input.
- Code references are made using the following abbreviations:
  - AREMA -AREMA Chapter 15 - Current Edition
  - NS - Norfolk Southern Public Policy Manual - Current Edition
  - BDM - ODOT LFD Bridge Design Manual (2004) - Current Edition
  - AA - AASHTO LFD
  - AISC - AISC Steel Construction Manual, Thirteenth Edition

1.0 GEOMETRIC AND LOAD DATA

1.1 GIRDER AND SPAN DATA

Girder Flange width: $B_f := 15in$

Girder Flange thickness: $T_f := 2.75in$

Depth of web: $D_w := 42.5in$

Girder Moment of Inertia: $I_{x_pg} := 44926.0in^4$

Girder section properties spreadsheet

Girder Web thickness: $T_w := 0.875in$

SpanLength of the girder: $span := 64.0ft$

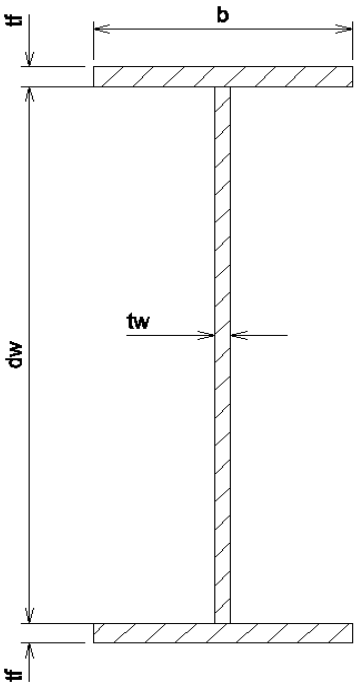
Skew angle of the bridge: $ang := 20.776deg$

Transverse Girder Spacing: $d_{girder} := 3.5ft$

Skewed girder spacing: $d_{skew} := d_{girder} \cdot \left(\frac{1}{\cos(ang)}\right) = 3.74\,ft$

1.2 END DIAPHRAGM AND JACKING STIFFENER DATA

1.2.1 SECTION 1: FULL SECTION



FULL SECTION

Flange width: $b_{f.ed} := 6 \cdot in$

Flange Thickness: $t_{f.ed} := 0.5 \cdot in$

Total depth of section: $d_{ed} := D_w - 2in = 40.5 \cdot in$

Height of web: $d_{w.ed} := d_{ed} - 2 \cdot t_{f.ed} = 39.5 \cdot in$

Thickness of Web: $t_{w.ed} := 0.5 \cdot in$

Area of the section: $A_{ed} := d_{w.ed} \cdot t_{w.ed} + b_{f.ed} \cdot \left(d_{ed} - d_{w.ed}\right) = 25.75 \cdot in^2$

Flange area: $A_{f.ed} := t_{f.ed} \cdot b_{f.ed} = 3 \cdot in^2$

Web area: $A_{w.ed} := d_{w.ed} \cdot t_{w.ed} = 19.75 \cdot in^2$

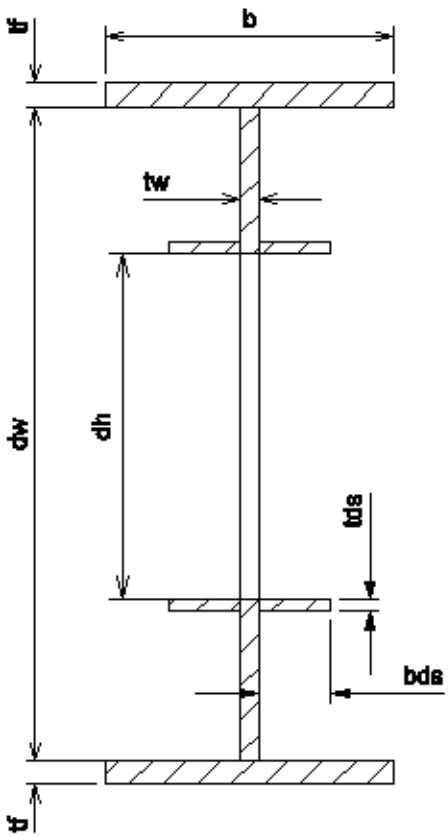
Mom. of Inertia: $I_{x.ed} := \frac{t_{w.ed} \cdot d_{w.ed}^3}{12} + \frac{b_{f.ed} \cdot \left(d_{ed}^3 - d_{w.ed}^3\right)}{12} = 4968.04 \cdot in^4$

Effective column height factor: $k := 0.75$ Table 15-1-11 Arema

Section Modulus: $S_{x.ed} := \frac{I_{x.ed}}{\left(\frac{d_{ed}}{2}\right)} = 245.34 \cdot in^3$

Radus of Gyration: $r_{x.ed} := \sqrt{\frac{I_{x.ed}}{A_{ed}}} = 13.89 \cdot in$

1.2.2 SECTION 2: SECTION AT ACCESS HOLE



SECTION AT ACCESS HOLE

Width of flange: $b_{f.ed} = 6 \cdot in$

Thickness of flange: $t_{f.ed} = 0.5 \cdot in$

Height of web: $d_{w.ed} = 39.5 \cdot in$

Total depth of section: $d_{ed} = 40.5 \cdot in$

Thickness of Web: $t_{w.ed} = 0.5 \cdot in$

Depth of hole: $d_{h.ed} := 24 \cdot in$

Half-Width of horizontal stiffener $b_{ds.ed} := 0 \cdot in$

Thickness of horizontal stiffener: $t_{ds.ed} := 0 \cdot in$

Minimum dia of access hole: $Dia_{access} := 15in$

Clearance to access hole from jacking points: $clear_{access} := .75in$

Minimum 15 in access hole to be provided as per NS Appendix H 2.5. 2.5 in spacing is provided on either sides to the jacking stiffener. It is assumed that a single jack will be used under each diaphragm, but to be conservative jacking stiffners will be analyzed to accept the full load

Area of section at hole:

$$A_{h.ed} := d_{w.ed} \cdot t_{w.ed} + b_{f.ed} \cdot \left(d_{ed} - d_{w.ed}\right) - d_{h.ed} \cdot t_{w.ed} + 4 \cdot b_{ds.ed} \cdot t_{ds.ed} \ldots = 13.75 \cdot in^2$$

Area of flange: $A_{f.ed} = 3 \cdot in^2$

Area of web with hole: $A_{w_h.ed} := d_{w.ed} \cdot t_{w.ed} - d_{h.ed} \cdot t_{w.ed} = 7.75 \cdot in^2$

Moment of Inertia of section at hole:

$$I_{x_h.ed} := \frac{t_{w.ed} \cdot d_{w.ed}^3}{12} - \frac{t_{w.ed} \cdot d_{h.ed}^3}{12} + \frac{b_{f.ed} \cdot \left(d_{ed}^3 - d_{w.ed}^3\right)}{12} + \frac{2 \cdot b_{ds.ed} \cdot \left[\left(d_{h.ed} + 2 \cdot t_{ds.ed}\right)^3 - d_{h.ed}^3\right]}{12} \ldots = 4392.04 \cdot in^4$$

Section modulus: $S_{_h.ed} := \dfrac{I_{x_h.ed}}{\left(\dfrac{d_{ed}}{2}\right)} = 216.89 \cdot in^3$

Radius of gyration: $r_{x_h.ed} := \sqrt{\dfrac{I_{x_h.ed}}{A_{h.ed}}} = 17.87 \cdot in$

1.2.3 JACKING STIFFENER DATA

Stiffener thickness: $t_{js} := 0.75 \cdot in$

Width of weld clip: $d_{weld} := 1 \cdot in$

Height of weld clip: $h_{weld} := 1 \cdot in$

1.3 END BEARING STIFFENER DATA

Stiffener plate thickness: $t_{end_stiff} := 0.75in$

Horizontal Leg of corner clip: $l_{cc.h.ebs} := 1in$

Stiffener width: $b_{end_stiff} := (B_f - T_w) \cdot 0.5 = 7.06 \cdot in$

Vertical Leg of corner clip: $l_{cc.v.ebs} := 1in$

Participating length of the web: $l_{web} := 25 \cdot t_{w.ed} = 12.5 \cdot in$ Arema 15-1.7.7. c

1.4 INTERMEDIATE STIFFENER AND INTERNAL DIAPHRAGM

1.4.1 STIFFENER SECTION PROPERTIES

Stiffener plate thickness: $t_{int_stiff} := 0.5in$

Horizontal Leg of corner clip: $l_{cc.h.ibs} := 1in$

Stiffener width: $b_{int_stiff} := 5in$

Vertical leg of corner clip: $l_{cc.v.ibs} := 2in$

Participating length of the web: $l_{int_web} := 25 \cdot T_w = 21.87 \cdot in$ Arema 15-1.7.7. c

Clear distance between stiffeners: $d_{int_stiff} := 42.0in$

Area of stiffener plate: $A_{int_stiff} := t_{int_stiff} \cdot b_{int_stiff} = 2.5 \cdot in^2$

Distance from centroid of plates to center of single plate: $d_{cent} := (T_w + b_{int_stiff}) \cdot 0.5 = 2.94 \cdot in$

Thickness of the stifffener connection plate: $t_{plate} := 0.5in$ NS Appendix H.2-4 B-3.b

1.4.2 DIAPHRAGM SECTION PROPERTIES

Width of flange: $b_{f_id} := 6 \cdot in$

Thickness of flange: $t_{f_id} := .5 \cdot in$

Height of web: $d_{w_id} := d_{w.ed} = 39.5 \cdot in$

Thickness of Web: $t_{w_id} := 0.5 \cdot in$

Total depth of section: $d_{id} := d_{w_id} + 2 \cdot t_{f_id} = 40.5 \cdot in$

Flange area: $A_{f_id} := t_{f_id} \cdot b_{f_id} = 3 \cdot in^2$

Web area: $A_{w_id} := d_{w_id} \cdot t_{w_id} = 19.75 \cdot in^2$

Area of the section: $A_{id} := d_{w_id} \cdot t_{w_id} + b_{f_id} \cdot (d_{id} - d_{w_id}) = 25.75 \cdot in^2$

Moment of Inertia of the section:

$$I_{x_id} := \frac{t_{w_id} \cdot d_{w_id}^3}{12} + \frac{b_{f_id} \cdot \left(d_{id}^3 - d_{w_id}^3\right)}{12} = 4968.04 \cdot in^4$$

Section Modulus: $S_{id} := \frac{I_{x_id}}{\left(\frac{d_{id}}{2}\right)} = 245.34 \cdot in^3$

Radus of Gyration: $r_{x_id} := \sqrt{\frac{I_{x_id}}{A_{id}}} = 13.89 \cdot in$

1.5 LATERAL BRACING

A steel angle of 4"x4"x0.5" is used for the lateral bracing.

Depth of lateral bracing: $D_{lat} := 4in$

Flange thickness: $T_{f.lat} := 0.5in$

Flange width: $B_{f.lat} := 4in$

Area of bracing: $A_{lat} := 3.8in^2$

Moment of inertia of the bracing: $I_{lat} := 5.56in^4$

Longitudinal spacing between bracing points: $Spa_{lat.l} := 4ft$

Dimensional parameters of the chosen angle are taken from AISC steel tables.

Lateral spacing between girders: $Spa_{lat.g} := d_{girder} = 3.5\,ft$

Length of the diagonal lateral brace: $Len_{lat} := \sqrt{Spa_{lat.l}^2 + Spa_{lat.g}^2} = 5.32\,ft$

Radius of gyration: $r_{lat} := \sqrt{\frac{I_{lat}}{A_{lat}}} = 1.21 \cdot in$

1.6 GIRDER LOADS

1.6.1 GIRDER REACTIONS

from girder design spreadsheet

Reaction due to dead load from span1: $R_{1T} := 59.21kip$

Reaction due to dead load from span2: $R_{2T} := 59.21kip$

Loads that act on the end bearing stiffener:

Maximum load on the stiffener from girder: $R_{max} := 217.5kip$ *Vertical forces due to DL+LL+I+CF+LF*

Fatigue load on the stiffener: $R_{sr} := 157.84kip$ *Vertical forces due to LL+I+CF+LF*

Loads that act on the Intermediate bearing stiffener:

Uniformly distributed load on the girder: $W_{load} := 5.927klf$ *Vertical forces due to DL+LL+I+CF+LF*

Concentrated derail load acting on the girder: $P_{derail} := 56.7kip$

Direct load acting on girder (vertical shear): $F_{v.pg} := 6.217klf$ *vertical load due to DL+LL+I+CF+LF+W*

Direct load is used in design of girder flange to web - full penetration groove weld.

Shear Stress in Girder: $R_{vg} := 6ksi$

Stress due to LL+I in Girder: $R_{LLI} := 15.54ksi$

1.6.2 LATERAL LOADS ON GIRDER

B1 Load combination with 125% overstress is considered for the lateral bracing design. Table 4.5 "Design of Modern Steel Railway Bridges" by John Unsworth.

There is no CWR force applicable for this particular bridge type and no lateral force due to LL+I+DL.

Lateral forces to be considered:

Lateral wind load on loaded bridge: $P_{lat.w} := 306.90plf + 300plf$ $P_{lat.w} = 0.61\,klf$ *wind on Superstructure+Live load*

Lateral force due to Nosing: $P_{lat.N} := 25kip$

Total lateral load acting on each diagonal brace: $P_{tot} := P_{lat.w} \cdot Spa_{lat.l} + P_{lat.N} = 27.43 \cdot kip$

2.0 MATERIAL DATA

2.1 PRIMARY GIRDER

STRUCTURAL STEEL: A709 GRADE 50S YIELD STRENGTH: $f_y := 50 \cdot ksi$ ULTIMATE STRENGTH: $f_u := 65 \cdot ksi$

Elastic modulus of steel: $E_s := 29000 \cdot ksi$ Unit weight of steel: $w_s := 490pcf$

$$ESFY := \sqrt{\frac{E_s}{f_y}} = 24.08 \qquad FYES := \frac{1}{ESFY} = 0.04$$

Number of Stress cycles is greater than 2,000,000 (for span lengths less than 100 ft) AREMA Table 15-1-7

Allowable Fatigue Stress: $F_f := 10ksi$ AREMA Table 15-1-10 Category C, Descr. 1.3

Allowable bearing stress: $F_{bear} := 0.83 \cdot f_y = 41.5\,ksi$ AREMA Table 15-1-11

2.2 BOLT AND WELD CONNECTIONS

2.2.1 BOLTS AND WELDS ALLOWABLE STRESSES

Allowable shear stress in A 325 bolts Table 15-1-11 Arema $S_{v.blt} := 17ksi$ *slip critical connection*

Allowable shear stress in fillet welds Table 15-1-13 Arema $S_{v.weld} := \min(19ksi, 0.35 \cdot f_y) = 17.5\,ksi$

Allowable shear stress in groove welds Table 15-1-13 Arema $S_{gr.weld} := 0.35 \cdot f_y = 17.5\,ksi$

Allowable Tension in A 325 bolts: Table 15-1-11 Arema $S_{p.blt.max} := 44000psi$

Acting on gross section as per AREMA Table 15-1-11 & Table 15-1-11a - Class A Standard Hole

2.2.2 BOLT CONNECTION PROPERTIES (7/8" Bolt):

STRUCTURAL BOLTS: 7/8" DIAMETER A325 SC (CLASS B, STANDARD HOLES)
Appendix H.4.1-2 -4.1 NS Steel grade

$d_{b78} := \frac{7}{8} \cdot in$ Appendix H.2-3 NS min. bolt diameter

These bolts will be used for the Lower Lateral Bracing

Allowable shear force in A 325 bolts: $R_{v78} := S_{v.blt} \cdot \pi \cdot d_{b78}^2 \cdot 0.25$

$R_{v78} = 10.22 \cdot kip$ (single shear)

Gross area of bolt: $A_{blt.g78} := \frac{\pi \cdot d_{b78}^2}{4}$ $A_{blt.g78} = 0.6 \cdot in^2$

Diameter of bolt hole: $Hole_{dg78} := d_{b78} + 0.125in$ AREMA 15-1.5.8(e) $Hole_{dg78} = 1.000 \cdot in$

Minimum bolt spacing: $spa_{min78} := 3.0 \cdot d_{b78} = 2.63 \cdot in$ AREMA 15-1.9.3 b

Minimum edge distance: $edge_{min78} := 1.75 \cdot d_{b78} = 1.53 \cdot in$ AREMA 15-1.9.4

2.2.2 BOLT CONNECTION PROPERTIES

STRUCTURAL BOLTS: 1" DIAMETER A325 SC (CLASS B, STANDARD HOLES)
Appendix H.4.1-2 -4.1 NS Steel grade

$d_{b1} := 1 \cdot in$ Appendix H.2-3 NS min. bolt diameter

These bolts will be used for all Girder to Diaphragm Connections

Allowable shear force in A 325 bolts: $R_{v1} := S_{v.blt} \cdot \pi \cdot d_{b1}^2 \cdot 0.25$

$R_{v1} = 13.35 \cdot kip$ (single shear)

Gross area of bolt: $A_{blt.g1} := \frac{\pi \cdot d_{b1}^2}{4}$ $A_{blt.g1} = 0.79 \cdot in^2$

Diameter of bolt hole: $Hole_{dg1} := d_{b1} + 0.125in$ AREMA 15-1.5.8(e) $Hole_{dg1} = 1.125 \cdot in$

Minimum bolt spacing: $spa_{min1} := 3.0 \cdot d_{b1} = 3 \cdot in$ AREMA 15-1.9.3 b

Minimum edge distance: $edge_{min1} := 1.75 \cdot d_{b1} = 1.75 \cdot in$ AREMA 15-1.9.4

2.2.3 END DIAPHRAGM TO BEARING STIFFENER CONNECTION

	Bolt Column 1 (Furthest from the Web):	Bolt Column 2
Total Number of Bolts in a Column:	$Bolts_{C1_ED} := 13$	$Bolts_{C2_ED} := 0$
Bolt Pitch:	$Pitch_{C1_ED} := 3 \cdot in$	$Pitch_{C2_ED} := Pitch_{C1_ED}$
Distance of First Bolt from Lowest Bolt (Stagger):	$BoltFirst_{C1_ED} := 0 \cdot in$	$BoltFirst_{C2_ED} := 0in$

Lateral Spacing of Bolt Column (C1 is the zero point... Set to ZERO if only one column) $Gage_{ED} := 0 \cdot in$

C/L of bolt to edge of diphragm web: $Db_{dwED} := 1.75in$

Edge distance of bolts (to top edge of web: $Edge_{dist.ED} := Round\left[\left[d_{w_id} - \left[\left(Bolts_{C1_ED} - 1\right) \cdot Pitch_{C1_ED}\right]\right] \cdot 0.50, 0.01in\right] = 1.75 \cdot in$

$CHK_{bED} := if\left(Db_{dwED} \geq edge_{min1} \wedge Edge_{dist.ED} \geq edge_{min1}, \text{"OK"}, \text{"NO GOOD"}\right)$ $CHK_{bED} = \text{"OK"}$

2.2.4 GIRDER WEB TO INTERMEDIATE DIAPHRAGM WEB CONNECTION

	Bolt Column 1 (Furthest from the Web):	Bolt Column 2
Total Number of Bolts in a Column:	$Bolts_{C1_ID} := 13$	$Bolts_{C2_ID} := 0$
Bolt Pitch:	$Pitch_{C1_ID} := 3.0 \cdot in$	$Pitch_{C2_ID} := Pitch_{C1_ID}$
Distance of First Bolt from Lowest Bolt (Stagger):	$BoltFirst_{C1_ID} := 0 \cdot in$	$BoltFirst_{C2_ID} := 0in$

Lateral Spacing of Bolt Column (C1 is the zero point... Set to ZERO if only one column) $Gage_{ID} := 0 \cdot in$

C/L of bolt to edge of diaphragm web: $Db_{dwID} := 1.75 \cdot in$

Edge distance of bolts (to top edge of web):

$$Edge_{dist.ID} := Round\left[\left[d_{w_id} - \left[\left(Bolts_{C1_ID} - 1\right) \cdot Pitch_{C1_ID}\right]\right] \cdot 0.50, .01in\right] = 1.75 \cdot in$$

$$CHK_{bID} := if\left(Db_{dwID} \geq edge_{min1} \wedge Edge_{dist.ID} \geq edge_{min1}, \text{"OK"}, \text{"NO GOOD"}\right)$$

$$CHK_{bID} = \text{"OK"}$$

2.2.5 LATERAL BRACING TO STIFFENER CONNECTION PLATE

	Bolt Column 1 (Furthest from the Web):	Bolt Column 2
Total Number of Bolts in a Column:	$Bolts_{C1_lt} := 4$	$Bolts_{C2_lt} := 0 \cdot Bolts_{C1_lt}$
Bolt Pitch:	$Pitch_{C1_lt} := 3.0 \cdot in$	$Pitch_{C2_lt} := Pitch_{C1_lt}$
Distance of First Bolt from Lowest Bolt (Stagger):	$BoltFirst_{C1_lt} := 0 \cdot in$	$BoltFirst_{C2_lt} := 0in$

Lateral Spacing of Bolt Column (C1 is the zero point... Set to ZERO if only one column) $Gage_{lt} := 0 \cdot in$

Horizontal distance between centroid of bolts and Point Load: $e_{lt} := 0 \cdot in$ *To Diaphragm Stiffening Girder*

Edge distance of bolts: $Edge_{dist.lt} := 1.75in$

2.2.6 WELD CONNECTION PROPERTIES

WELDING ELECTRODES: E70XX
Appendix H.4.1-2 -4.c NS

STIFFENER WELD: 5/16" $t_{weld} := \frac{5}{16} \cdot in$

AASHTO TABLE 6.13.3.4.1 - as nothing is given in AREMA

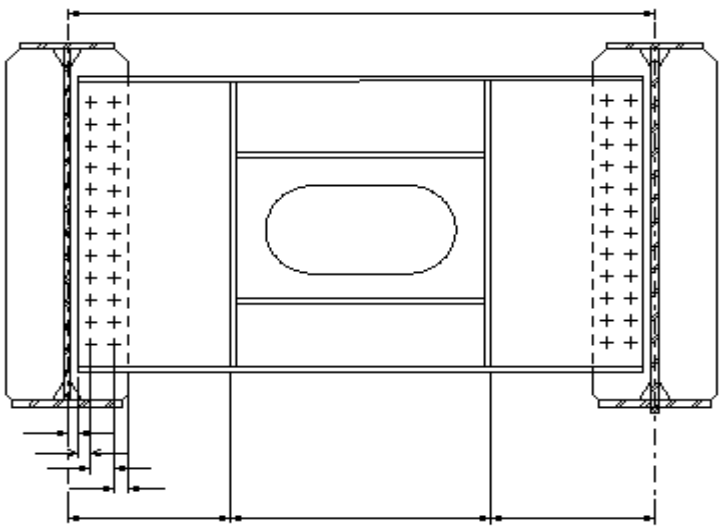
assume all fillet weld connections use same size.

Allowable shear force on weld: $R_{weld} := S_{v.weld} \cdot t_{weld} \cdot 0.707 = 3.87 \cdot \frac{kip}{in}$

END DIAPHRAGM (JACKING)

(AREMA/NSRR Design: Prosser Avenue Bridge)

DETERMINE THE ADEQUACY OF THE END DIAPHRAGM DURING JACKING. JACKING WILL BE REQUIRED TO REPLACE THE BEARINGS. THE JACKING OPERATION WILL BE CONDUCTED UNDER FULL DEAD LOAD ONLY (NO LIVE LOAD).

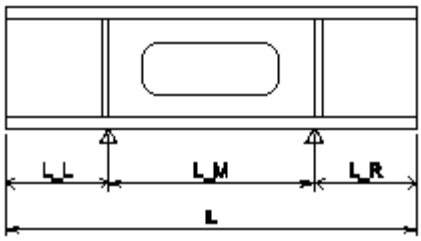


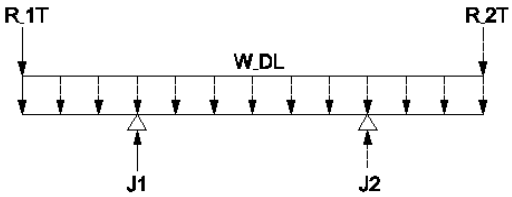
END DIAPHRAGM

3.0 DESIGN OF END DIAPHRAGM AND JACKING STIFFENER

3.1 LOAD CALCULATIONS FOR END DIAPHRAGM

assumed simply supported.





Distance between the jacking points: $L_M := Dia_{access} + 2clear_{access}$ $L_M = 16.5 \cdot in$

Distance from jacking stiffener to center of beam: $L_L := \left(d_{skew} - L_M\right) \cdot 0.5$ $L_L = 14.21 \cdot in$

Assuming the jacking points are symmetrical: $L_R := L_L$ $L_R = 14.21 \cdot in$

3.1.1. DEAD LOAD OF DIAPHRAGM:

Dead load of the diaphragm: $W_{DL} := A_{ed} \cdot w_s = 0.09\,klf$

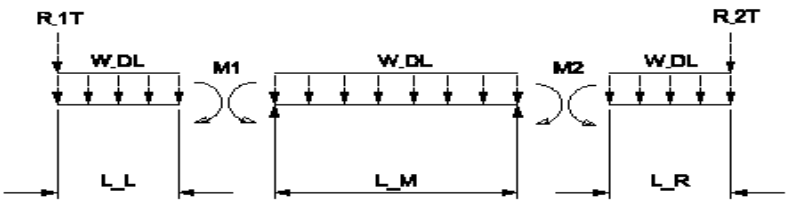
weight due to hole section is not reduced for conservative purpose

Additional dead load: $W_{ADDL} := 0 \cdot W_{DL} = 0$

Additional dead load due to the connections is incorporated in the girder reactions.

Total dead load: $W_{DL_T} := W_{DL} + W_{ADDL} = 0.09\,klf$

3.1.2 DETERMINE THE FORCES ON THE DIAPHRAGM:



Moment at left jacking stiffener: $M_1 := R_{1T} \cdot L_L + W_{DL_T} \cdot L_L \cdot \frac{L_L}{2} = 70.18 \cdot ft \cdot kip$

Moment at right jacking stiffener: $M_2 := R_{2T} \cdot L_R + W_{DL_T} \cdot L_R \cdot \frac{L_R}{2} = 70.18 \cdot ft \cdot kip$

Reactions at the location of left and right Jacking stiffener respectively:

$$V_1 := \left(W_{DL_T} \cdot L_L + R_{1T} + W_{DL_T} \cdot \frac{L_M}{2}\right) + \frac{(M_1 - M_2)}{L_M} = 59.37 \cdot kip \qquad J_1 := V_1 = 59.37 \cdot kip$$

$$V_2 := \left(W_{DL_T} \cdot L_R + R_{2T} + W_{DL_T} \cdot \frac{L_M}{2}\right) - \frac{(M_1 - M_2)}{L_M} = 59.37 \cdot kip \qquad J_2 := V_2 = 59.37 \cdot kip$$

3.2 CHECK FOR ADEQUACY OF END DIAPHRAGM:

3.2.1 CHECK BENDING STRESS IN THE MEMBER: AREMA 15-1.4.1

Allowable Diagonal tension in webs of girders: $F_{b_T} := 0.55 \cdot f_y = 27.5 \cdot ksi$ Table 15-1-11 AREMA

COMPRESSION LIMIT: CONSIDER ONLY SECTION ABOVE HOLE

Moment of Inertia of the section: $I_C := \frac{t_{f.ed} \cdot b_{f.ed}^3}{12} + \frac{\frac{(d_{w.ed} - d_{h.ed})}{2} \cdot t_{w.ed}^3}{12} = 9.08 \cdot in^4$

Area of the section: $A_C := t_{f.ed} \cdot b_{f.ed} + \frac{(d_{w.ed} - d_{h.ed})}{2} \cdot t_{w.ed} = 6.87 \cdot in^2$

Radius of gyration: $r_C := \sqrt{\frac{I_C}{A_C}} = 1.15 \cdot in$

Compression in extreme fibers of flexural members symmetrical about the principal axis

$$FC := \max\left[\frac{0.131 \cdot \pi \cdot E_s}{\frac{L_M \cdot d_{ed} \cdot \sqrt{1.3}}{A_{f.ed}}}, 0.55 \cdot f_y - \frac{0.55 \cdot f_y^2}{6.3 \cdot \pi^2 \cdot E_s} \cdot \left(\frac{L_M}{r_C}\right)^2\right] = 46.99 \cdot ksi$$

Table 15-1-11 AREMA

$$F_{b_C} := if\left(0.55 \cdot f_y < FC, 0.55 \cdot f_y, FC\right) = 27.5\,ksi$$

Bending Stress in Member: $f_b := \frac{\max(M_1, M_2)}{S_h.ed} = 3.88 \cdot ksi$ $F_b := \min(F_{b_T}, F_{b_C}) = 27.5 \cdot ksi$

$CHK_b := if(f_b < F_b, "OK", "NO GOOD")$ $CHK_b = "OK"$

3.2.2 CHECK SHEAR STRESS IN THE MEMBER: AREMA 15-1.4.1

Allowable Shear in webs of rolled beams: $F_v := 0.35 \cdot f_y = 17.5 \cdot ksi$ Table 15-1-11 Arema

Shear Stress in the section: $f_v := \frac{\max(V_1, V_2)}{A_{h.ed}} = 4.32 \cdot ksi$

$CHK_v := if(f_v < F_v, "OK", "NO GOOD")$ $CHK_v = "OK"$

$DIAPHRAGM := if(CHK_b = "OK" \wedge CHK_v = "OK", "OK", "NO GOOD")$ $DIAPHRAGM = "OK"$

3.3 CHECK THE REQUIRED WELD SIZE FOR THE WEB-FLANGE CONNECTION:

Allowable shear force for Weld in consideration: $R_{weld} = 3.87 \cdot \frac{kip}{in}$

First moment of flange area: $Q_{ed} := A_{f.ed} \cdot \frac{(d_{w.ed} + t_{f.ed})}{2} = 60 \cdot in^3$

Horizontal shear force acting on the weld: $r_w := \frac{\max(V_1, V_2) \cdot Q_{ed}}{2 \cdot I_{x_h.ed}} = 0.41 \cdot \frac{kip}{in}$ *similar to shear between concrete slab and steel beam*

$CHK_W := \text{if}(r_w < R_{weld}, \text{"OK"}, \text{"NO GOOD"})$ $CHK_W = \text{"OK"}$

NO LIVE LOAD ON JACKING, THEREFORE NO FATIGUE CHECK IS REQUIERED

3.4 CHECK THE JACKING STIFFENERS

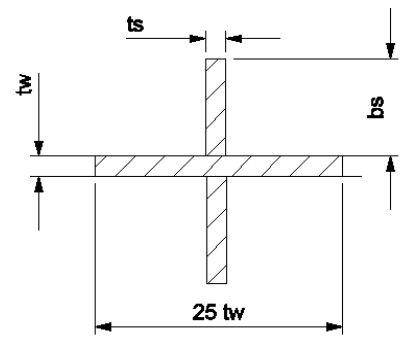
AREMA 15-1.7.7

Flange thickness of diaphragm: $t_{f.ed} = 0.5 \cdot in$

Flange width of diaphragm: $b_{f.ed} = 6 \cdot in$

Web depth of diaphragm: $d_{w.ed} = 39.5 \cdot in$

Stiffener thickness: $t_{js} = 0.75 \cdot in$



3.5 CHECK THE MINIMUM THICKNESS OF THE JACKING STIFFENER

Outstanding width: $l_s := 0.5(b_{f.ed} - t_{w.ed}) = 2.75 \cdot in$ *Equal to the outstanding width of the end diaphragm section*

Minimum thickness of stiffener: $t_{min} := \frac{l_s}{0.43ESFY} = 0.27 \cdot in$ AREMA 15.1.6.2 (a) (2)

$CHK_t := \text{if}(t_{js} > t_{min}, \text{"OK"}, \text{"NO GOOD"})$ $CHK_t = \text{"OK"}$

3.6 CHECK THE STRENGTH OF THE JACKING STIFFENER

Area of the column section: $A_c := 25 \cdot t_{w.ed} \cdot t_{w.ed} + 2 \cdot (l_s \cdot t_{js}) = 10.38 \cdot in^2$ AREMA 15.1.7.7 c

Moment of inertia of the section: $I_{y_s} := \frac{t_{js} \cdot (l_s + t_{w.ed} + l_s)^3}{12} = 13.5 \cdot in^4$

Radius of gyration: $r_{y_s} := \sqrt{\frac{I_{y_s}}{A_c}} = 1.14 \cdot in$

Stress in the member: $f_a := \frac{\max(J_1, J_2)}{A_c} = 5.72 \cdot ksi$

Slenderness of the member: $K_{js} := k \cdot \frac{d_{w.ed}}{r_{y_s}} = 25.97$

Lower limit of slenderness: $K_1 := \frac{0.629}{FYES} = 15.15$

Higher limit of slenderness: $K_2 := \frac{5.034}{FYES} = 121.23$

Allowable stress in the section:

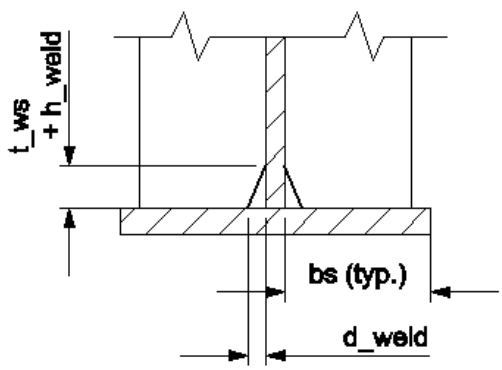
$$F_a := \text{if}\left[K_{js} < K_1, 0.55 \cdot f_y, \text{if}\left[K_{js} > K_1 \wedge K_{js} < K_2, 0.6 \cdot f_y - \left(17500 \cdot \frac{f_y}{E_s}\right)^{\frac{3}{2}} \cdot K_{js} \cdot psi, \frac{0.514 \cdot \pi^2 \cdot E_s}{K_{js}^2}\right]\right]$$

AREMA 15.1.4.1

$F_a = 25.7 \cdot ksi$

$CHK_a := \text{if}(f_a < F_a, \text{"OK"}, \text{"NO GOOD"})$ $CHK_a = \text{"OK"}$

3.7 CHECK THE BEARING STRESS ON THE BOTTOM FLANGE:



Stiffener weld thickness: $t_{weld} = 0.31 \cdot in$

Width of the stiffener: $l_s = 2.75 \cdot in$

Height of weld clip: $h_{weld} = 1 \cdot in$

Width of weld clip: $d_{weld} = 1 \cdot in$

Net width of stiffener: $b_{s_net} := l_s - d_{weld} = 1.75 \cdot in$

Area outside the corner clip of plate stiffener: $A_{b.js} := 2 \cdot b_{s_net} \cdot t_{js} = 2.62 \cdot in^2$

AREMA 15- 1.7.7 d

Bearing Stress in the member: $f_{b_f} := \frac{\max(J_1, J_2)}{A_{b.js}} = 22.62 \cdot ksi$

ALLOWABLE STRESS: $F_{bear} = 41.5\,ksi$ AREMA Table 15-1-11

$CHK_F := \text{if}(f_{b_f} < F_{bear}, \text{"OK"}, \text{"NO GOOD"})$ $CHK_F = \text{"OK"}$

3.8 CHECK THE WELD OF THE STIFFENER TO THE WEB:

Length of the weld: $L_{w.ed} := d_{w.ed} - 2\left(h_{weld} + t_{weld}\right) = 36.87 \cdot in$

Shear force in the weld: $r_{w_s.ed} := \dfrac{\max\left(J_1, J_2\right)}{4 \cdot L_{w.ed}} = 0.4 \cdot \dfrac{kip}{in}$ *Both side weld*

Allowable shear force in weld: $R_{weld} = 3.87 \cdot \dfrac{kip}{in}$

$CHK_{w_s} := if\left(r_{w_s.ed} < R_{weld}, "OK", "NO\ GOOD"\right)$ $CHK_{w_s} = "OK"$

$STIFFENER := if\left(CHK_t = "OK" \wedge CHK_a = "OK" \wedge CHK_F = "OK" \wedge CHK_{w_s} = "OK", "OK", "NO\ GOOD"\right)$

$STIFFENER = "OK"$

3.9 CONNECTION OF END DIAPHRAGM TO BEARING STIFFENER

3.9.1 BOLT CONNECTION DETAILS

Neutral Axis of Rows (Y-Bar) From Bottom Most Bolt

$$Ybar_{ED} := \frac{\sum_{\Delta = Bolts_{C1_ED}-1}^{0} \left(BoltFirst_{C1_ED} + \Delta \cdot Pitch_{C1_ED}\right) \ldots + \sum_{\sigma = Bolts_{C2_ED}-1}^{0} \left(BoltFirst_{C2_ED} + \sigma \cdot Pitch_{C2_ED}\right)}{Bolts_{C1_ED} + Bolts_{C2_ED}}$$

$Ybar_{ED} = 17.77 \cdot in$

Sum of Y Squared

$$SumY2_{ED} := \sum_{\Delta = Bolts_{C1_ED}-1}^{0} \left(BoltFirst_{C1_ED} + \Delta \cdot Pitch_{C1_ED} - Ybar_{ED}\right)^2 \ldots + \sum_{\sigma = Bolts_{C2_ED}-1}^{0} \left(BoltFirst_{C2_ED} + \sigma \cdot Pitch_{C2_ED} - Ybar_{ED}\right)^2$$

$SumY2_{ED} = 2385.8 \cdot in^2$

Neutral Axis of Columns (X-Bar) From Column 1 Bolts

$$Xbar_{ED} := \frac{Bolts_{C2_ED} \cdot Gage_{ED}}{Bolts_{C1_ED} + Bolts_{C2_ED}}$$

$Xbar_{ED} = 0 \cdot in$

Sum of X Squared $SumX2_{ED} := Bolts_{C1_ED} \cdot Xbar_{ED}^2 + Bolts_{C2_ED} \cdot \left(Gage_{ED} - Xbar_{ED}\right)^2$ $SumX2_{ED} = 0 \cdot in^2$

$$OffsetGage_{ED} := \sqrt{Gage_{ED}^2 + \min\left(Pitch_{C1_ED} - BoltFirst_{C1_ED}, Pitch_{C2_ED} - BoltFirst_{C2_ED}\right)^2} = 3 \cdot in$$

$$BoltSpacing := if\left(BoltFirst_{C1_ED} = 0 \wedge BoltFirst_{C2_ED} = 0 \wedge Gage_{ED} \neq 0, Gage_{ED}, OffsetGage_{ED}\right) = 3 \cdot in$$

Polar moment of Inertia of bolt configuration: $J_{ED} := SumX2_{ED} + SumY2_{ED}$ $J_{ED} = 2385.8 \cdot in^2$

Total Height of Bolt Array:

$$Bolt_{Ht} := \max\left[\left(Bolts_{C1_ED} - 1\right) \cdot Pitch_{C1_ED} + BoltFirst_{C1_ED}, \left(Bolts_{C2_ED} - 1\right) \cdot Pitch_{C2_ED} + BoltFirst_{C2_ED}\right] = 36 \cdot in$$

Number of bolts: $n_{ED} := Bolts_{C1_ED} + Bolts_{C2_ED} = 13$

Vertical location of centroid of bolt configuration: $y_{ED} := Ybar_{ED} = 17.77 \cdot in$

Horizontal location of centroid of bolt configuration: $x_{ED} := Xbar_{ED} = 0 \cdot in$

MinimumSpacing of bolts: $spa_{ED} := BoltSpacing = 3 \cdot in$

3.9.2 CHECK THE BOLT CONFIGURATION

$CHK_{Spa} := if\left(spa_{ED} \geq spa_{min1}, \text{"OK"}, \text{"NO GOOD"}\right)$ $CHK_{Spa} = \text{"OK"}$

$CHK_{Edge} := if\left(Edge_{dist.ED} \geq edge_{min1}, \text{"OK"}, \text{"NO GOOD"}\right)$ $CHK_{Edge} = \text{"OK"}$

3.9.3 CHECK THE SHEAR IN THE BOLTS:

Allowable Diaphragm Shear Stress: $\sigma_{a.v} := 0.35 \cdot f_y$ REF: AREMA Table 15-1-11 $\sigma_{a.v} = 17.5 \cdot ksi$

Required Shear Capacity of the connection: AREMA 15-1.11.4 (c) $P_{d.blt} := d_{w.ed} \cdot t_{w.ed} \cdot \sigma_{a.v} \cdot 0.5$ $P_{d.blt} = 172.81 \cdot kip$

Case 1 Vertical Shear/ Bolt: AREMA 15-11.4 (c) $F_{d.blt} := \frac{P_{d.blt}}{n_{ED}} \cdot 1.0$ $F_{d.blt} = 13.29 \cdot kip$

Bolt Eccentricity: $e_{ED} := \left(L_L + L_M + L_R\right) \cdot 0.5 - \left(Db_{dwED} + Xbar_{ED}\right) = 20.71 \cdot in$

Force acting on the bolt configuration: $V_{ED} := \max\left(R_{1T}, R_{2T}\right) \cdot \left(\frac{6}{5}\right) \cdot 0.5 = 35.53 \cdot kip$

Moment acting on the bolt configuration: $M_{ED} := V_{ED} \cdot e_{ED} = 61.31 \cdot kip \cdot ft$

Horizontal shear component due to reaction: $r_{vx} := 0$

Horizontal shear component due to moment: $r_{mx} := \frac{M_{ED} \cdot y_{ED}}{J_{ED}} = 5.48 \cdot kip$

Vertical shear component due to reaction: $r_{vy} := \frac{V_{ED}}{n_{ED}} = 2.73 \cdot kip$

Vertical shear component due to moment: $r_{my} := \frac{M_{ED} \cdot x_{ED}}{J_{ED}} = 0 \cdot kip$

Case 2 Shear force PER BOLT $r_v := \sqrt{\left(r_{vx} + r_{mx}\right)^2 + \left(r_{vy} + r_{my}\right)^2} = 6.12 \cdot kip$

$r_{vmax} := \max\left(r_v, F_{d.blt}\right) = 13.29 \cdot kip$

Allowable Shear force PER BOLT: $R_{v1} = 13.35 \cdot kip$

$CHK_{BOLT} := \text{if}\left(r_{vmax} < R_{v1}, \text{"OK"}, \text{"NO GOOD"}\right)$ $CHK_{BOLT} = \text{"OK"}$

End Bearing Stiffener

DETERMINE THE REQUIRED WIDTH AND THICKNESS OF THE END BEARING STIFFENER

4.0 DESIGN END BEARING STIFFENER

4.1 CHECK THE ADEQUACY OF END BEARING STIFFENER

4.1.1 CHECK STIFFENER DIMENSIONS

Loads computed in section 3.1 under end diaphragm are used for the design of end bearing stiffener.

Max Stiffener Width, Lesser of the Following:

$$b_{s.max1} := \frac{(B_f - T_w)}{2} = 7.06 \cdot in \qquad b_{s.max2} := 0.43 \cdot t_{end_stiff} \cdot \sqrt{\frac{E_s}{f_y}} = 7.77 \cdot in$$

Arema 15-1.6.2 a (2)

Minimum thickness of Bearing stiffener:

$$t_{min.ebs} := \frac{b_{s.max1}}{0.43ESFY} = 0.68 \cdot in$$

AREMA 15-1.6.2 (a) (2)

$$CHK_{t.ebs} := if(t_{end_stiff} > t_{min.ebs}, "OK", "NO\ GOOD") \qquad CHK_{t.ebs} = "OK"$$

$$b_{s.max} := min(b_{s.max1}, b_{s.max2}) \qquad b_{s.max} = 7.06 \cdot in$$

$$CHK_w := if(b_{end_stiff} \le b_{s.max}, "OK", "NO\ GOOD") \qquad CHK_w = "OK"$$

4.1.2 STRENGTH OF THE BEARING STIFFENER

Designing the stiffener as a column, Contributing Area:

$$A_{ebs_col} := (l_{web} \cdot T_w) + 2 \cdot (b_{end_stiff} \cdot t_{end_stiff}) = 21.53 \cdot in^2$$

Moment of inertia of the section:

$$I_{yy} := \frac{1}{12} \cdot t_{end_stiff} \cdot (b_{end_stiff} + T_w + b_{end_stiff})^3 = 210.94 \cdot in^4$$

Radius of gyration:

$$r_{ebs} := \sqrt{\frac{I_{yy}}{A_{ebs_col}}} = 3.13 \cdot in$$

Slenderness ratio:

$$K_{ebs} := \frac{k \cdot D_w}{r_{ebs}} = 10.18$$

Depth of web is the length of the stiffener.

Allowable stress for high slender members:

$$F_{a1} := \frac{0.514 \cdot \pi^2 \cdot E_s}{(K_{ebs})^2} = 1418.55\ ksi$$

Table 15-1-11 Arema

Allowable stress for medium slender members: $F_{a2} := 0.60 \cdot f_y - \left(17500 \frac{f_y}{E_s}\right)^{\frac{3}{2}} \cdot K_{ebs} \cdot psi = 28.31\,ksi$

Allowable stress for thicker members: $F_{a3} := 0.55 \cdot f_y = 27.5\,ksi$

Allowable stress for the stiffener section: $F_{a_ebs} := if\left(K_{ebs} \ge \frac{5.034}{FYES}, F_{a1}, if\left(\frac{0.629}{FYES} < K_{ebs} < \frac{5.034}{FYES}, F_{a2}, if\left(K_{ebs} \le \frac{0.629}{FYES}, F_{a3}, \text{"Error"}\right)\right)\right)$

$F_{a_ebs} = 27.5 \cdot ksi$

Stress in the stiffener section: $f_{a_ebs} := \frac{R_{max}}{A_{ebs_col}} = 10.1 \cdot ksi$

$CHK_{sh.ebs} := if\left(f_{a_ebs} \le F_{a_ebs}, \text{"OK"}, \text{"NO GOOD"}\right)$ $CHK_{sh.ebs} = \text{"OK"}$

4.1.3 BEARING STRESS ON THE BOTTOM FLANGE

Bearing stress caused by the stiffeners on the bottom flange is to be checked.

Allow 1" for corner clip for weld at the flange - web juncture.

Net width of the stiffener: $b_{ebs_net} := b_{end_stiff} - l_{cc.h.ebs} = 6.06 \cdot in$ *reducing width of clip*

Area in bearing due to 2 stiffeners: $A_{ebs} := 2 \cdot b_{ebs_net} \cdot t_{end_stiff} = 9.09 \cdot in^2$

Allowable bearing stress: $F_{bear} = 41.5\,ksi$ (AREMA 15-1-38)

Bearing stress in the stiffener: $f_{ebs} := \frac{R_{max}}{A_{ebs}} = 23.92 \cdot ksi$

$CHK_{bea.ebs} := if\left(f_{ebs} \le F_{bear}, \text{"OK"}, \text{"NO GOOD"}\right)$ $CHK_{bea.ebs} = \text{"OK"}$

Check Plate Design

$PLATE := if\left(CHK_w = \text{"OK"} \wedge CHK_{t.ebs} = \text{"OK"} \wedge CHK_{sh.ebs} = \text{"OK"} \wedge CHK_{bea.ebs} = \text{"OK"}, \text{"OK"}, \text{"NO GOOD"}\right)$

$PLATE = \text{"OK"}$

4.1.4 WELD CONNECTION FOR END STIFFENERS

Allowable Shear force in weld: $R_{weld} = 3.87 \cdot \frac{kip}{in}$

Number of weld lines, $N_{ebs} := 2 \cdot 2 = 4$

Stress in weld: $f_{w_ebs} := \frac{R_{max}}{N_{ebs} \cdot \left(D_w - 2l_{cc.v.ebs}\right)} = 1.34 \cdot \frac{kip}{in}$

$WELD := if\left(f_{w_ebs} \le R_{weld}, \text{"OK"}, \text{"NO GOOD"}\right)$ $WELD = \text{"OK"}$

For a redundant load path structure, the allowable weld stress for transverse stiffeners corresponds to stress category C'. Arema Table 15-1-9, Description 4.1

Number of Stress cycles is greater than 2,000,000 (for span lengths less than 100 ft) Arema Table 15-1-7

Allowable Fatigue Stress: $F_{fw} := 12ksi$ Arema Table 15-1-10

Area of the weld: $A_{weld} := N_{ebs} \cdot \left(D_w - 2l_{cc.v.ebs}\right) \cdot 0.707 \cdot t_{weld} = 35.79 \cdot in^2$

Fatigue stress in weld: $f_f := \frac{R_{sr}}{A_{weld}} = 4.41 \cdot ksi$

$FATIGUE := if\left(f_f \le F_{fw}, "OK", "NO\ GOOD"\right)$

$FATIGUE = "OK"$

Min weld for 1.125" plate = 5/16" (AWS Table 2.1)

Appendix H.2-4 4b NS- Bearing stiffeners shall be milled to bear.

Check Design

$DESIGN := if(PLATE = "OK" \wedge WELD = "OK" \wedge FATIGUE = "OK", "OK", "NO\ GOOD")$

$DESIGN = "OK"$

Intermediate Stiffener and Diaphragm

DETERMINE THE REQUIRED WIDTH AND THICKNESS OF THE INTERNAL DIAPHRAGM AND INTERMEDIATE BEARING STIFFENER. DESIGN THE BOLTED CONNECTION BETWEEN THE STIFFENER AND THE DIAPHRAGM.

5.0 DESIGN INTERMEDIATE STIFFENER AND DIAPHRAGM

5.1 LOAD CALCULATIONS

5.1.1 Normal loading condition

Load under the normal conditions is due to the differential deflection of the adjacent girders caused because of the bridge skew.
Total Dead Load + Live Load + Impact Deflection (Equivalent Uniform Load) is used to ensure conservative design.

Deflection at mid span due to the UDL: $Delta_{mid} := \frac{5 \cdot W_{load} \cdot span^4}{384 \cdot E_s \cdot I_{x_pg}} = 1.72 \cdot in$

For any position of the rail load on the bridge, there will be a distance between the corresponding points over the adjacent girders due to the skew angle of the bridge. This difference causes a differential deflection that creates load onto the diaphragm. Calculations are done about mid point for simplicity.

Distance between corresponding points over the adjacent girder: $dist := d_{girder} \cdot tan(ang) = 15.93 \cdot in$

Point of interest from the support is at a distance: $a := span \cdot 0.5 - dist = 30.67\ ft$

Deflection at the point of interest: $Delta_a := \left(\frac{W_{load} \cdot a^4 + W_{load} \cdot span^3 \cdot a}{24 \cdot E_s \cdot I_{x_pg}} \right) - \left(\frac{W_{load} \cdot span \cdot a^3}{12 \cdot E_s \cdot I_{x_pg}} \right) = 1.71 \cdot in$

Differential deflection: $Diff := Delta_{mid} - Delta_a = 0.0035 \cdot in$

Assuming the structural configuration of intermediate diaphragm connected to the girder as a cantilever, back calculating the concentrated load that can cause the above computed differential deflection to get the load.
Deflection at the cantilever due to concentrated load at tip of the beam is Pl^3/3EI , where P is the load and l is the span of the beam.

Equivalent load for the differential deflecion: $P := 3 \cdot \frac{E_s \cdot I_{x_id} \cdot Diff}{d_{girder}^3} = 20.69 \cdot kip$

End moments (assuming fixed-free cantilever condition): $M_{ser} := P \cdot d_{girder} = 72.43\ ft \cdot kip$

5.1.2 Derail Condition

"A single line of wheel loads (Q) equal to the design load per rail (Article 1.3.3) including full design impact is to be applied at an eccentricity of 5 feet from the centerline of track as shown in Figure 15-1-4, but no further than the edge of the deck or, for open decks, the bridge ties."

The entire load is assumed to act on a single girder for conservative purpose

NSRR standards allow 50% overstress in the derail load combination G

End moments (assuming fixed-free cantilever condition): $M_{derail} := P_{derail} \cdot d_{girder} = 198.45\ ft \cdot kip$

5.2 CHECK THE ADEQUACY OF INTERMEDIATE STIFFENER

5.2.1 CHECK INTERMEDIATE STIFFENER DIMENSIONS

Maximum depth of web without transverse stiffener: $D_{w_max} := 2.12 \cdot T_w \sqrt{\frac{E_s}{f_y}} = 44.67 \cdot in$ Arema 15-1.7.8 a

$CHK_{depth} := if\left(D_w > D_{w_max}, \text{"Stiffener req'd."}, \text{"Stiffener not req'd."}\right)$ $CHK_{depth} = \text{"Stiffener not req'd."}$

Maximum clear spacing between the stiffeners: $d_{int_stiff_max} := \min\left(96in, D_w, 1.95 \cdot t_{int_stiff} \cdot \sqrt{\frac{E_s}{R_{vg}}}\right) = 42.5 \cdot in$
Arema 15-1.7.8 a

$CHK_{dist} := if\left(d_{int_stiff} < d_{int_stiff_max}, \text{"OK."}, \text{"NO GOOD"}\right)$ $CHK_{dist} = \text{"OK."}$

Minimum moment of inertia of the section: $I_{min} := 2.5 \cdot t_{int_stiff}^3 \cdot d_{int_stiff} \cdot \left(\frac{D_w^2}{d_{int_stiff_max}^2} - 0.7\right) = 3.94 \cdot in^4$
Arema 15-1.7.8 a

Moment of inertia of stiffeners: $I_{int_stiff} := 2\left(A_{int_stiff} \cdot d_{cent}^2 + t_{int_stiff} \cdot \frac{b_{int_stiff}^3}{12}\right) = 53.56 \cdot in^4$

$CHK_{inertia} := if\left(I_{min} < I_{int_stiff}, \text{"OK."}, \text{"NO GOOD"}\right)$ $CHK_{inertia} = \text{"OK."}$

Minimum width of the stiffener plate: $b_{int_stiff_min} := 2in + \frac{D_w + 2 \cdot T_f}{30} = 3.6 \cdot in$
Arema 15-1.7.8 b

Maximum width of the stiffener plate: $b_{int_stiff_max} := 16 \cdot t_{int_stiff} = 8 \cdot in$

$CHK_{width} := if\left(b_{int_stiff_min} < b_{int_stiff} < b_{int_stiff_max}, \text{"OK"}, \text{"NO GOOD"}\right)$ $CHK_{width} = \text{"OK"}$

5.2.2 STRENGTH OF THE INTERMEDIATE BEARING STIFFENER

Designing the stiffener as a column, Contributing Area: $A_{ibs} := \left(l_{int_web} \cdot T_w\right) + 2 \cdot \left(b_{int_stiff} \cdot t_{int_stiff}\right) = 24.14 \cdot in^2$
Arema 15-1.7.7 c

Moment of inertia of the considered section: $I_{yy_int} := \frac{1}{12} \cdot t_{int_stiff} \cdot \left(b_{int_stiff} + T_w + b_{int_stiff}\right)^3 = 53.59 \cdot in^4$

Radius of gyration: $r_{int} := \sqrt{\frac{I_{yy_int}}{A_{ibs}}} = 1.49 \cdot in$

Slenderness ratio: $K_{ibs} := \frac{k \cdot \left(D_w - t_{plate}\right)}{r_{int}} = 21.14$

Allowable stress for high slender members: $F_{a1.ibs} := \frac{0.514 \cdot \pi^2 \cdot E_s}{(K_{ibs})^2} = 329.13\,ksi$ Table 15-1-11 Arema

Allowable stress for medium slender members: $F_{a2.ibs} := 0.60 \cdot f_y - \left(17500 \frac{f_y}{E_s}\right)^{\frac{3}{2}} \cdot K_{ibs} \cdot psi = 26.5\,ksi$

Allowable stress for thicker members: $F_{a3.ibs} := 0.55 \cdot f_y = 27.5\,ksi$

Allowable stress for the stiffener section:

$$F_{a_ibs} := if\left(K_{ibs} \ge \frac{5.034}{FYES}, F_{a1.ibs}, if\left(\frac{0.629}{FYES} < K_{ibs} < \frac{5.034}{FYES}, F_{a2.ibs}, if\left(K_{ibs} \le \frac{0.629}{FYES}, F_{a3.ibs}, "Error"\right)\right)\right)$$

$F_{a_ibs} = 26.5 \cdot ksi$

Stress in the stiffener section: $f_{a_ibs} := \frac{P}{A_{ibs}} = 0.86 \cdot ksi$

$CHK_1 := if\left(f_{a_ibs} \le F_{a_ibs}, "OK", "NO\ GOOD"\right)$ $CHK_1 = "OK"$

5.3 WELD CONNECTION FOR INTERMEDIATE STIFFENERS

Allowable Shear force in weld: $R_{weld} = 3.87 \cdot \frac{kip}{in}$

Number of weld lines, $N_{ibs} := 2 \cdot 2 = 4$

Stress in weld: $f_{w_ibs} := \frac{P}{N_{ibs} \cdot (D_w - 2l_{cc.v.ibs} - t_{plate})} = 0.14 \cdot \frac{kip}{in}$

$f_{w_ibs.derail} := \frac{P_{derail}}{N_{ibs} \cdot (D_w - 2l_{cc.v.ibs} - t_{plate})} = 0.37 \cdot \frac{kip}{in}$

$WELD_{ibs} := if\left(f_{w_ibs} < R_{weld} \wedge f_{w_ibs.derail} < 1.5 \cdot R_{weld}, "OK", "NO\ GOOD"\right)$ $WELD_{ibs} = "OK"$

5.4 CHECK THE ADEQUACY OF INTERNAL DIAPHRAGM

5.4.1 DIAPHRAGM MOMENT CAPACITY

Flexural stress in serviceable condition: $\sigma_{b.ser} := \frac{M_{ser}}{S_{id}} = 3.54 \cdot ksi$

Flexural stress in derail condition: $\sigma_{b.derail} := \frac{M_{derail}}{S_{id}} = 9.71 \cdot ksi$

Allowable flexural stress: $\sigma_b := 0.55 \cdot f_y = 27.5 \cdot ksi$ Arema Table 15-1-11

$CHK_2 := if\left(\sigma_{b.ser} < \sigma_b \wedge \sigma_{b.derail} < 1.5 \cdot \sigma_b, "OK", "NO\ GOOD"\right)$ $CHK_2 = "OK"$

5.4.2 DIAPHRAGM SHEAR CAPACITY

Shear stress in serviceable condition: $\sigma_{v.ser} := \frac{P}{A_{w_id}} = 1.05 \cdot ksi$

Shear stress in derail condition: $\sigma_{v.derail} := \frac{P_{derail}}{A_{w_id}} = 2.87 \cdot ksi$

Allowable shear stress: $\sigma_v := 0.35 \cdot f_y = 17.5 \cdot ksi$ Arema Table 15-1-11

$CHK_3 := if\left(\sigma_{v.ser} < \sigma_v \wedge \sigma_{v.derail} < 1.5 \cdot \sigma_v, \text{"OK"}, \text{"NO GOOD"}\right)$ $CHK_3 = \text{"OK"}$

5.5 WEB BOLT SHEAR CAPACITY

(Shear Connection - Girder Web to Diaphragm Web Connection)

5.5.1 BOLT CONNECTION DETAILS

Neutral Axis of Rows (Y-Bar) From Bottom Most Bolt

$$Ybar_{ID} := \frac{\sum_{\Delta = Bolts_{C1_ID}-1}^{0} \left(BoltFirst_{C1_ID} + \Delta \cdot Pitch_{C1_ID}\right) \ldots + \sum_{\sigma = Bolts_{C2_ID}-1}^{0} \left(BoltFirst_{C2_ID} + \sigma \cdot Pitch_{C2_ID}\right)}{Bolts_{C1_ID} + Bolts_{C2_ID}}$$

$Ybar_{ID} = 17.77 \cdot in$

Sum of Y Squared

$$SumY2_{ID} := \sum_{\Delta = Bolts_{C1_ID}-1}^{0} \left(BoltFirst_{C1_ID} + \Delta \cdot Pitch_{C1_ID} - Ybar_{ID}\right)^2 \ldots + \sum_{\sigma = Bolts_{C2_ID}-1}^{0} \left(BoltFirst_{C2_ID} + \sigma \cdot Pitch_{C2_ID} - Ybar_{ID}\right)^2$$

$SumY2_{ID} = 2385.8 \cdot in^2$

Neutral Axis of Columns (X-Bar) From Column 1 Bolts

$$Xbar_{ID} := \frac{Bolts_{C2_ID} \cdot Gage_{ID}}{Bolts_{C1_ID} + Bolts_{C2_ID}}$$

$Xbar_{ID} = 0 \cdot in$

Sum of X Squared

$$SumX2_{ID} := Bolts_{C1_ID} \cdot Xbar_{ID}^2 + Bolts_{C2_ID} \cdot \left(Gage_{ID} - Xbar_{ID}\right)^2$$

$SumX2_{ID} = 0 \cdot in^2$

$$EccentricBolts := \sqrt{Gage_{ID}^2 + \min\left(Pitch_{C1_ID} - BoltFirst_{C1_ID}, Pitch_{C1_ID} - BoltFirst_{C2_ID}\right)^2} = 3 \cdot in$$

$$BoltSpacing_{ID} := if\left(BoltFirst_{C1_ID} = 0 \wedge BoltFirst_{C2_ID} = 0 \wedge Gage_{ID} \neq 0, Gage_{ID}, EccentricBolts\right) = 3 \cdot in$$

Polar moment of Inertia of bolt configuration: $J_{ID} := SumX2_{ID} + SumY2_{ID}$ $J_{ID} = 2385.8 \cdot in^2$

Total Height of Bolt Array:

$$Bolt_{Ht.Id} := \max\left[\left(Bolts_{C1_ID} - 1\right)\cdot Pitch_{C1_ID} + BoltFirst_{C1_ID}, \left(Bolts_{C2_ID} - 1\right)\cdot Pitch_{C2_ID} + BoltFirst_{C2_ID}\right] = 36 \cdot in$$

Number of bolts: $n_{ID} := Bolts_{C1_ID} + Bolts_{C2_ID} = 13$

Vertical location of centroid of bolt configuration: $y_{ID} := Ybar_{ID} = 17.77 \cdot in$

Horizontal location of centroid of bolt configuration: $x_{ID} := Xbar_{ID} = 0 \cdot in$

MinimumSpacing of bolts: $spa_{ID} := BoltSpacing_{ID} = 3 \cdot in$

$CHK_{Spa.ID} := if\left(spa_{ID} \geq spa_{min1}, \text{"OK"}, \text{"NO GOOD"}\right)$ $CHK_{Spa.ID} = \text{"OK"}$

$CHK_{Edge.ID} := if\left(Edge_{dist.ID} \geq edge_{min1}, \text{"OK"}, \text{"NO GOOD"}\right)$ $CHK_{Edge.ID} = \text{"OK"}$

5.5.2 DESIGN OF BOLT CONNECTION

Allowable Diaphragm Shear Stress: $\sigma_{a.v} := 0.35 \cdot f_y$ *REF: AREMA Table 15-1-11* $\sigma_{a.v} = 17.5 \cdot ksi$

Required Shear Capacity of the connection: $P_{d.bltVdg} := d_{w_id} \cdot t_{w_id} \cdot \sigma_{a.v} \cdot 0.5$ $P_{d.bltVdg} = 172.81 \cdot kip$
AREMA 15-1.11.4 (c)

Vertical Shear/ Bolt: $F_{d.bltVcf} := \frac{P_{d.bltVdg}}{n_{ID}} \cdot 1.0$ $F_{d.bltVcf} = 13.29 \cdot kip$
AREMA 15-11.4 (c)

Horizontal distance between centroid of bolts and centerline of girder web: $e_{ID} := 0.5B_f - Db_{dwID} - 0.5 \cdot Gage_{ID} = 5.75 \cdot in$ *0.5*thk of flange-edge distance-0.5*hor spacing of bolts*

Moment acting on the bolt configuration: $M_{blt.ser} := P \cdot e_{ID} = 9.92\, ft \cdot kip$ *Service condition*

Moment acting on the bolt configuration: $M_{blt.derail} := P_{derail} \cdot e_{ID} = 27.17\, ft \cdot kip$ *Derail condition*

Shear due to bending moment acting in the y direction on the bolt group

Force to bolt group at C/L of bolt pair-serviceable condition: $F_{blt.v.ser.y} := \frac{M_{blt.ser} \cdot x_{ID}}{J_{ID}}$ $F_{blt.v.ser.y} = 0 \cdot kip$

Force to bolt group at C/L of bolt pair- Derail condition: $F_{blt.v.derail.y} := \frac{M_{blt.derail} \cdot x_{ID}}{J_{ID}}$ $F_{blt.v.derail.y} = 0 \cdot kip$

Shear due to bending moment acting in the x direction on the bolt group

Force to bolt group at C/L of bolt pair-serviceable condition: $F_{blt.v.ser.x} := \frac{M_{blt.ser} \cdot y_{ID}}{J_{ID}}$ $F_{blt.v.ser.x} = 0.89 \cdot kip$

Force to bolt group at C/L of bolt pair- Derail condition: $F_{blt.v.derail.x} := \frac{M_{blt.derail} \cdot y_{ID}}{J_{ID}}$ $F_{blt.v.derail.x} = 2.43 \cdot kip$

Shear stress due to vertical shear acting in the y direction on the bolt group

Vertical design shear force per bolt-serviceable condition: $F_{d.bltV.ser} := \frac{P}{n_{ID}}$ $F_{d.bltV.ser} = 1.59 \cdot kip$

Vertical design shear force per bolt-Derail condition: $F_{d.bltV.derail} := \frac{P_{derail}}{n_{ID}}$ $F_{d.bltV.derail} = 4.36 \cdot kip$

Total Shear on a bolt: $F_{d.vT.ser} := \sqrt{F_{blt.v.ser.x}^2 + \left(F_{blt.v.ser.y} + F_{d.bltV.ser}\right)^2}$ $F_{d.vT.ser} = 1.82 \cdot kip$ *Service condition*

Total Shear on a bolt: $F_{d.vT.derail} := \sqrt{F_{blt.v.derail.x}^2 + \left(F_{blt.v.derail.y} + F_{d.bltV.derail}\right)^2}$

$F_{d.vT.derail} = 4.99 \cdot kip$ *Derail condition*

Shear Checks:

1/2 Diaphragm condition $CHK_4 := if\left(F_{d.bltVcf} \le R_{v1}, \text{"OK"}, \text{"NO GOOD"}\right)$ $CHK_4 = \text{"OK"}$

Service condition $CHK_5 := if\left(F_{d.bltV.ser} \le R_{v1}, \text{"OK"}, \text{"NO GOOD"}\right)$ $CHK_5 = \text{"OK"}$

Derail condition $CHK_6 := if\left(F_{d.bltV.derail} \le 1.5 \cdot R_{v1}, \text{"OK"}, \text{"NO GOOD"}\right)$ $CHK_6 = \text{"OK"}$

5.6 - BLOCK SHEAR CAPACITY - STIFFENER WEB

This assumes a vertical force on stiffener equal to half the shear capacity of the diaphragm. Due to the bolts being in single shear, entire amount of that force will be carried to stiffener. It will be assumed that all bolts must shear through the entire web to fail; i.e. all bolt spacings + end space, each in single shear

C/L of bolt to edge of stiffener: $Db_s := Db_{dwID} + Gage_{ID}$ $\quad Db_s = 1.75 \cdot in$

Vertical force causing rupture: $F_{bs} := P_{d.bltVdg}$ $\quad F_{bs} = 172.81 \cdot kip$

Gross area subject to tension: $A_{gt} := Db_s \cdot t_{int_stiff}$ $\quad A_{gt} = 0.87 \cdot in^2$

$hc := if\left(Gage_{ID} = 0, 0.5, 1.5\right) = 0.5$

Net area subject to tension: $A_{Nt} := A_{gt} - hc \cdot \left(Hole_{dg1} \cdot t_{int_stiff}\right)$ $\quad A_{Nt} = 0.59 \cdot in^2$

Gross area subject to shear: $A_{gv} := \left[Bolt_{Ht.Id} + 0.5 \cdot \left(D_w - Bolt_{Ht.Id}\right)\right] \cdot t_{int_stiff}$ $\quad A_{gv} = 19.62 \cdot in^2$

Net area subject to shear: $A_{Nv} := A_{gv} - \left(\max\left(Bolts_{C1_ID}, Bolts_{C2_ID}\right) - 0.5\right) \cdot \left(Hole_{dg1} \cdot t_{int_stiff}\right)$ $\quad A_{Nv} = 12.59 \cdot in^2$

Maximum allowable force allowed for block shear

$P_1 := 0.30 \cdot f_u \cdot A_{Nv} + 0.50 \cdot f_u \cdot A_{Nt} = 264.88 \cdot kip$

$P_2 := 0.30 \cdot f_y \cdot A_{gv} + 0.50 \cdot f_u \cdot A_{Nt} = 313.67 \cdot kip$ $\quad$ **AREMA 15-1.5.9 (e)**

$P_{bs.a} := \min\left(P_1, P_2\right) = 264.88 \cdot kip$

Shear Stress Check: $CHK_7 := if\left(F_{bs} \le P_{bs.a}, \text{"OK"}, \text{"NO GOOD"}\right)$ $\quad CHK_7 = \text{"OK"}$

5.7 - BLOCK SHEAR CAPACITY- FAILURE OF DIAPHRAGM WEB AT CONNECTION

(Shear Connection - Girder Web to Diaphragm Web Connection)

This assumes a vertical force on the web equal to the shear capacity of the diaphragm. It will be assumed that all bolts must shear through the entire web and top flange to fail; i.e. all bolt spacings + spacing to the outside of the flange

Vertical force causing rupture: $F_{bs.dw} := P_{d.bltVdg}$ $\quad F_{bs.dw} = 172.81 \cdot kip$

Gross area subject to tension: $A_{gt.dw} := \left(Db_{dwID} + Gage_{ID}\right) \cdot t_{w_id}$ $\quad A_{gt.dw} = 0.87 \cdot in^2$

Net area subject to tension: $A_{Nt.dw} := A_{gt.dw} - hc \cdot \left(Hole_{dg1} \cdot t_{w_id}\right)$ $\quad A_{Nt.dw} = 0.59 \cdot in^2$

Gross area subject to shear: $A_{gv.dw} := \left[Bolt_{Ht.Id} + 0.5 \cdot \left(d_{w_id} - Bolt_{Ht.Id}\right)\right] \cdot t_{w_id}$

$A_{gv.dw} = 18.87 \cdot in^2$

Net area subject to shear: $A_{Nv.dw} := A_{gv.dw} - \left(\max\left(Bolts_{C1_ID}, Bolts_{C2_ID}\right) - 0.50\right) \cdot Hole_{dg1} \cdot t_{w_id} = 11.84 \cdot in^2$

Maximum allowable force allowed for block shear: Reference: AREMA 15-1.5.9 (e)

$P_{dw1} := 0.30 \cdot f_u \cdot A_{Nv.dw} + 0.50 \cdot f_u \cdot A_{Nt.dw} = 250.25 \cdot kip$

$P_{dw2} := 0.30 \cdot f_y \cdot A_{gv.dw} + 0.50 \cdot f_u \cdot A_{Nt.dw} = 302.42 \cdot kip$

$Max_{P.dw} := \min(P_{dw1}, P_{dw2}) = 250.25 \cdot kip$

Shear Stress Check: $CHK_8 := if(F_{bs.dw} \le Max_{P.dw}, "OK", "NO GOOD")$ $CHK_8 = "OK"$

5.8 - LOAD INDUCED FATIGUE ON GIRDER FLANGE

Max Allowable Stress in Flange: $\sigma_{all.gf} := 16.0ksi$ Reference: AREMA Table 15-1-9 Description 2.1

$CHK_9 := if(R_{LLI} \le \sigma_{all.gf}, "OK", "NO GOOD")$ $CHK_9 = "OK"$

Lateral Bracing and Connections

DETERMINE THE REQUIRED SECTION OF THE LATERAL BRACING AND THE BOLT CONNECTION DETAILS OF LOWER LATERAL BRACING TO THE STIFFENER CONNECTION PLATE.

6.0 DESIGN LOWER LATERAL BRACING

(LLB may not be required for spans with concrete decks and girders less than 72". See AREMA 15-1.11.2)

6.1 CHECK FOR ADEQUACY OF BRACING AND CONNECTION

6.1.1 CHECK FOR COMPRESSION

Slenderness ratio: $K_{lat} := \frac{k \cdot Len_{lat}}{r_{lat}} = 39.55$

Allowable stress for high slender members: $\sigma_{a1} := \frac{0.514 \cdot \pi^2 \cdot E_s}{(K_{lat})^2} = 94.07\ ksi$ Table 15-1-13 Arema

Allowable stress for medium slender members: $\sigma_{a2} := 0.60 \cdot f_y - \left(17500 \cdot \frac{f_y}{E_s}\right)^{\frac{3}{2}} \cdot K_{lat}\ psi = 23.45\ ksi$

Allowable stress for thicker members: $\sigma_{a3} := 0.55 \cdot f_y = 27.5\ ksi$

Allowable stress for the stiffener section: $\sigma_{alo_lat} := \text{if}\left(K_{lat} \ge \frac{5.034}{FYES}, \sigma_{a1}, \text{if}\left(\frac{0.629}{FYES} < K_{lat} < \frac{5.034}{FYES}, \sigma_{a2}, \text{if}\left(K_{lat} \le \frac{0.629}{FYES}, \sigma_{a3}, \text{"Error"}\right)\right)\right)$

$\sigma_{alo_lat} = 23.45 \cdot ksi$

Stress in the lateral bracing member: $\sigma_{bra} := \frac{P_{tot}}{A_{lat}} = 7.22 \cdot ksi$

$CHK_{stress} := \text{if}\left(\sigma_{bra} \le 1.25 \cdot \sigma_{alo_lat}, \text{"OK"}, \text{"NO GOOD"}\right)$ $CHK_{stress} = \text{"OK"}$

6.2 BOLTED CONNECTION OF LATERAL BRACING TO STIFFENER CONNECTION PLATE

Stiffener connection plate is the plate bolted to the tension flange of the primary girder for bearing of the intermediate stiffener. This would avoid the category C type fatigue failure specified in AREMA.

Number of bolts per connection: $n_{lt} := 4$

Shear stress per bolt: $\sigma_{v.lat} := \frac{P_{tot}}{A_{blt.g78} \cdot n_{lt}}$ $\sigma_{v.lat} = 11.4 \cdot ksi$

Spacing used in the connection: $Spa_{x.lat} := Pitch_{C1_lt}$

Moment of inertia of bolt group: $J_{lat.blts} := 2 \cdot \left[\left(0.5 \cdot Spa_{x.lat}\right)^2 + \left(1.5 \cdot Spa_{x.lat}\right)^2\right]$ $J_{lat.blts} = 45 \cdot in^2$

Moment due to lateral force on the bracing: $M_{lat} := 0$ *Line of action of force passes through the connection.*

Force in the bolt group at the C/L of configuration: $F_{b.lat} := \frac{M_{lat} \cdot \left(1.5 Spa_{x.lat}\right)}{2 \cdot \left[\left(0.5 \cdot Spa_{x.lat}\right)^2 + \left(1.5 \cdot Spa_{x.lat}\right)^2\right]}$ $F_{b.lat} = 0 \cdot kip$

Shear stress per bolt: $\sigma_{b.lat} := \frac{F_{b.lat}}{A_{blt.g78}}$ $\sigma_{b.lat} = 0 \cdot ksi$

Total shear force in the bolt: $\sigma_{tot} := \sqrt{\sigma_{b.lat}^2 + \sigma_{v.lat}^2}$ $\sigma_{tot} = 11.4 \cdot ksi$

$CHK_{blt.sh} := \text{if}\left(\sigma_{tot} \le S_{v.blt}, \text{"OK"}, \text{"NO GOOD"}\right)$ $CHK_{blt.sh} = \text{"OK"}$

Primary Girder Flange to Web Weld Connection
(AREMA/NSRR Design: Prosser Avenue Bridge)

DETERMINE THE REQUIRED WELD PROPERTIES FOR THE FLANGE TO WEB OF THE PRIMARY DECK GIRDER.

Flange-to-web full penetration groove welds are designed to transmit horizontal shear due to the variation of the bending moment over the girder and the direct pressure due to applied distributed load

11.0 DESIGN OF WELD CONNECTION

NS Appendix H.2-4 B2 Full penetration groove welds

11.1 SECTION PROPERTIES AND LOADING

Material properties and weld electrodes are as defined in the above sections.

Thickness of weld: $t_{w_pg} := \min(T_w, T_f) = 0.88 \cdot in$

Allowable shear force on weld: $R_{w_pg} := S_{gr.weld} \cdot t_{w_pg} = 15.31 \cdot \frac{kip}{in}$

Shear force at section under consideration: $F_{h.pg} := R_{max} = 217.5 \cdot kip$

11.2 CHECK THE ADEQUACY OF THE WELD CONNECTION

First moment of flange area: $Q_{pg} := B_f \cdot T_f \cdot \frac{(D_w + T_f)}{2} = 933.28 \cdot in^3$

Horizontal shear force acting on the weld: $r_{wh_pg} := \frac{F_{h.pg} \cdot Q_{pg}}{I_{x_pg}} = 4.52 \cdot \frac{kip}{in}$

similar to shear between concrete slab and steel beam

Vertical shear force acting on the weld: $r_{wv_pg} := F_{v.pg} = 0.52 \cdot \frac{kip}{in}$

Total shear force acting on the weld: $r_{w_pg} := \sqrt{r_{wh_pg}^2 + r_{wv_pg}^2} = 4.55 \cdot \frac{kip}{in}$

$CHK_{sh.pg} := \text{if}(r_{w_pg} < R_{w_pg}, \text{"OK"}, \text{"NO GOOD"})$

$CHK_{sh.pg} = \text{"OK"}$

For a redundant load path structure, the allowable weld stress for complete joint penetration groove weld of web to flange corresponds to category C failure.

Arema Table 15-1-9 clause 5.3

Number of Stress cycles is greater than 2,000,000 (for span lengths less than 100 ft)

Arema Table 15-1-7

Horizontal shear force acting on the weld due to fatigue: $R_{wf_pg} := \frac{R_{sr} \cdot Q_{pg}}{I_{x_pg}} = 3.28 \cdot \frac{kip}{in}$

Stress due to fatigue: $f_{f.pg} := \frac{R_{wf_pg}}{t_{w_pg}} = 3.75\,ksi$

calculating the stress per unit inch of weld.

$FATIGUE_{pg} := \text{if}(f_{f.pg} \le F_f, \text{"OK"}, \text{"NO GOOD"})$

$FATIGUE_{pg} = \text{"OK"}$

DESIGN CALCULATION COVER SHEET

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PROJECT:	**ODOT/EMH&T: HAM-75-7.85**	*ODOT PID NO:*	*77889*
BRIDGE NO:	**HAM-PROSSER AVE**	*EMH&T JOB NO:*	
SUBJECT:	***Loads***	**GF JOB NO:**	**54682**
	Design for	CALC NO.:	**HAM075_PROS_11**
	Expansion and Fixed bearings	REV NO.:	**01**
FILE NAME(s):	*HAM075_PROS_11.xlsx*	**SHEET:**	**1 OF 6**

SUPERSEDES CALC NO: 00 (Stage 1 Calculation)

CALCULATION DESCRIPTION:

This will calculate all loads applied to the bearings per girder for use in design of fixed and expanion bearings.

Vertical Loads: The max girder reactions due to Dead load and Live load with impact is considered. Horizontal Loads: The horizontal girder loads due to centrifugal force, wind on live load, wind on susperstructure and nosing are considered in lateral direction and force due to braking is considered in longitudinal direction. Resultant of these forces is used to compute shear for the design of pintels and anchor bolts. Curved sole plate, Bronze sliding rocker plate and Masonry plate are designed for the expansion bearings. Fixed rocker sole plate and Masonry plate are designed for fixed bearings along with pintels. Anchor bolts are designed for fixed bearing and same will be used for expanion bearings.

DESIGN BASIS OR REFERENCES:

NS Underpass Design Criteria
AREMA (2013) Chapter 8, Section 5
AREMA (2014) Chapter 15, Section 5
Calculation: HAM075_PROSC_03 (Girder Loads and Reactions)
Revised Stage 1 Plans (after addressing comments)

MISCELLANEOUS NOTES:

Stage 3: Increased bearing sizes (minor, 1" width max change) to accommodate increased loading

RECORD OF ISSUES:

REV	DESCRIPTION	BY	DATE	VERIF'D	DATE
00	Stage 1 Calculation	VDK	04/15/14	EFD	08/12/14
01	Stage 3 Update	VDT	01/20/16	CTM	04/25/16

RECORD OF CHECK OF CURRENT CALCULATION VERSION

BY	DATE	CHKD	DATE	UPD.	DATE	VERIF'D	DATE
VDK	04/15/14	CTM	04/22/14	VDT	01/20/16	CTM	04/25/16

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Calculation	AREMA Sliding Brozne and Fixed Rocker Bearing Design (Steel Beam Bridge)
Description	HAM-PROSSER

GF Job No:	ODOT D8	Calculated	Checked	Updated	Review/Upd.	Verified
• 054682	• SFN: TBD	• VDK, 4/15/14	• CTM, 4/22/14	• CTM, 4/22/14	• VDK 4/23/14 • VDT, 1/20/16	• CTM, 4/25/16

REFERENCES

1. Norfolk Southern Standards - Norfolk
2. 2007 AREMA Manual for Railway Engineering - AREMA
3. ODOT BDM - BDM
4. AASHTO LRFD 6 - AA

REMARKS

1. To follow AREMA "Effective Length" notation, "Length" is measured transverse to girder
2. "Width" is measured parallel to girder

1.0 Inputs

1.1 Expansion bearing

Girder Input		Curved Sole Plate Input (Exp. Brg.)				Sliding Rocker plate Input (Exp. Brg.)				Masonry Plate Input (Exp. Brg.)			
Bot. Flange Thickness T_{BF} (in)	Bot. Flange Width B_{BF} (in)	Fy (Ksi)	Min. thickness of sole plate T_{SP}(in)	Width of sole plate W_{SP}(in)	Length of sole plate L_{SP}(in)	Max. thick. of bronze sliding plate T_{BR}(in)	Width of Bronze sliding plate W_{BR}(in)	Length of Bronze rocker L_{BR}(in)	Diameter of top curved surface D_{BR}(in)	F_c (Ksi)	Thickness of masonry plate T_{MP}(in)	Width of masonry plate W_{MP}(in)	Length of masonry plate L_{MP}(in)
2.75	15.0	36.0	1.5	11.0	18.5	1.000	7.5	17.0	36.0	4.0	3.25	15.00	28.5

Norfolk H.4.2-1 (3.B)

SETTING TEMP. (°F)	STEEL TEMPERATURE RANGE (°F)		α (ft/°F)	ΔT (°F) BDM S3.12.2		Expansion Length (ft)	ΔL-T Thermal Expansion	ΔL-A AREMA 15-5.1.2 c	ΔL(used) Max of ΔL-T & ΔL-A	Const. Tolerance AREMA 15-5.1.3 c	Anchor bolt (Fixed and Exp Brg.)		
	MIN.	MAX.		MIN.	MAX.						Diameter (in)	F_u Grade 105 Table 15-5-3	Penetration (in) AREMA 15-5.3.7c&e
60	-30	120	0.0000065	60	90	64	0.449 in	0.800 in	0.800 in	1.000 in	1.25	125.00 ksi	12.00

Design of anchor bolts is performed for fixed brg and same size is used for both bearings

1.2 Fixed Bearing

Fixed Rocker Sole Plate Input (Fixed Brg.)				Masonry Plate Input (Fixed Brg.)					Pintel Input (Fixed Brg.)			
Fy (Ksi)	Thickness of rocker sole plate T_{SP}(in)	Width of rocker sole plate W_{SP}(in)	Length of rocker sole plate L_{SP}(in)	Diameter of cylinder D_{SP}(in)	F_c (Ksi)	Thickness of masonry plate T_{MP}(in)	Width of masonry plate W_{MP}(in)	Length of masonry plate L_{MP}(in)	Class	$Diameter_{pin\ (in)}$	Fy (Ksi)	Penetration (in)
36.0	2.0	7.0	26.5	36.0	4.0	2.50	10.00	26.5	D	1.50	37.5	1.25

Norfolk H.4.2-1 (3.B) C, D, F or G AREMA Table 15-5-2

1.3 Loads

Total load (Max DL+LL w/I)		=	238.1	kips	Longitudinal force		=	33.3	kips
Weld Size	Sole Plate/Bottom Flange Conn.	=	0.3125	in	Allowable Weld Stress	F_v	=	12.6	kips
Lateral force		=	13.5	kips					

WL+WS+CF+N loads AREMA Table 15-1-13

Summary of Design checks

EXPANSION BEARING						
NS H.2-5.A.5	AREMA 15-5.3.5	AREMA 15-5.3.2.4	AREMA 15-5.3.4	AREMA 15-5.3.5	AREMA 15-5.3.5	Geometry
Steel Sole plate Thickness	Steel Sole Plate Line Stress	Welds	Bronze Rocker Bearing Stress	Masonry plate Thickness	Masonry plate brg. Stress	Masonry plate allow. expans.
% of Allow. = 67%	% of Allow. = 68%	% of Allow. = 39%	% of Allow. = 93%	% of Allow. = 46%	% of Allow. = 69%	% of Allow. = 96%
OK	OK	OK	OK	OK	OK	OK

FIXED BEARING				
AREMA 15-5.3.2.1	AREMA 15-5.3.5	AREMA 15-5.3.5	AREMA 15-5.3.2.4	AREMA 15-5.3.2.2
Pintel	Rocker Sole Plate	Masonry plate Stress	Welds	Anchor Bolt Shear
% of Allow. = 65%	% of Allow. = 61%	% of Allow. = 95%	% of Allow. = 86%	% of Allow. = 47%
OK	OK	OK	OK	OK

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Calculation	AREMA Sliding Brozne and Fixed Rocker Bearing Design (Steel Beam Bridge)
Description	HAM-PROSSER

GF Job No:	ODOT D8	Calculated	Checked	Updated	Review/Upd.	Verified
• 054682	• SFN: TBD	• VDK, 4/15/14	• CTM, 4/22/14	• CTM, 4/22/14	• VDK 4/23/14 • VDT, 1/20/16	• CTM, 4/25/16

2.0 Curved Sole Plate Design (Expansion Bearing.)

Thickness of Sole Plate	=	1.50 in	>	1.00 in	AREMA 15-5.3.5.d and NS H.2.3-5.A.5	OK
Effective Lengthh of Bearing Area	=	minimum of	17.00 in	Eff. Length of Bronze Rocker	AREMA 15-5.3.5.b	
			18.00 in	Eff. Length of Sole Plate (Flange Width + 1:1)		
	=	17.00 in				
Line Bearing Stress	=	14.01 kips/in				
Allowable Line Bearing Stress	=	20.70 kips/in			AREMA 15-5.2.1.c	

SOLE PLATE LINE BEARING STRESS CHECK: 14.01 kips/in < 20.70 kips/in **OK**

2.1 Curved Sole Plate to beam flange Weld Design (Expansion Bearing.)

Longitudinal Force		=	23.81 kips	= μ * load; μ = 0.1; Long. Force if fixed	AREMA 15-5.4.3.b
Allowable Shear Force	R_w	=	2.78 kips/in	=.707 * Fv * w	
Weld Length	L_w	=	22.00 in	= 2 * Sole Plate Width (Weld on each side of flange)	
Longitudinal Force in Welds	R_v	=	1.08 kips/in		

WELD STRESS CHECK: 1.08 kips/in < 2.78 kips/in **OK**

3.0 Curved Bronze sliding plate Design (Expansion Bearing)

Thickness of Bronze Rocker	=	1.00 in			
Effective Length of Bronze Rocker	=	minimum of	19.25 in	Eff. Length of Bronze Rocker	AREMA 15-5.3.4.1.d.(1 & 2)
			17.00 in	Actual Length of Bronze Rocker	
	=	17.00 in			
Width of Bronze Rocker	=	7.50 in			
Effective Bearing Area		127.50 sq.in			
Bearing Stress	=	1.87 ksi			
Allowable Bearing Stress	=	2.00 ksi			AREMA 15-5.2.3

BRONZE ROCKER LINE BEARING STRESS CHECK: 1.87 ksi < 2.00 ksi **OK**

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Calculation	AREMA Sliding Brozne and Fixed Rocker Bearing Design (Steel Beam Bridge)
Description	HAM-PROSSER

GF Job No:	ODOT D8	Calculated	Checked	Updated	Review/Upd.	Verified
• 054682	• SFN: TBD	• VDK, 4/15/14	• CTM, 4/22/14	• CTM, 4/22/14	• VDK 4/23/14 • VDT, 1/20/16	• CTM, 4/25/16

4.0 Masonry Plate Design

Thickness of Masonry Plate	=	3.25 in	>	1.50 in	AREMA 15-5.3.5.e	OK
Effective Length of Bearing Area	=	minimum of	23.50 in	Eff. Length of Bronze Rocker + 2*Masonry Plate Thickness	AREMA 15-5.3.5.b	
			28.50 in	Actual Length of Masonry Plate		
	=	23.50 in				
Effective Width of Bearing Area	=	minimum of	14.00 in	Eff. Width of Bronze Rocker + 2*Masonry Plate Thickness		
			15.00 in	Actual Width of Masonry Plate		
		14.00 in				
Effective Bearing Area	=	329.00 sq.in				
Bearing Stress	=	0.72 ksi				
Allowable Bearing Stress	=	1.05 ksi		Concrete Substructure will control	AREMA 8-2.26.c	
MASONRY PLATE BEARING STRESS CHECK:	0.72 ksi	<	1.05 ksi			OK

4.1 CHECK FOR THE WIDTH OF THE MASONRY PLATE

Anticipated longitudinal movement due to thermal expansion	=	0.80 in				
Allowance for construction tolerance	=	1.00 in				
Total anticipated longitudinal movement for expansion bearing	=	3.60 in				
Width available for Bronze Rocker to move on Masonry Plate	=	3.75 in				
CHECK FOR THE GENERAL WIDTH REQUIREMENT:	3.75 in	>	3.60 in		AREMA 15-5.1.2 & 5.1.3	OK

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Calculation	AREMA Sliding Brozne and Fixed Rocker Bearing Design (Steel Beam Bridge)
Description	HAM-PROSSER

GF Job No:	ODOT D8	Calculated	Checked	Updated	Review/Upd.	Verified
• 054682	• SFN: TBD	• VDK, 4/15/14	• CTM, 4/22/14	• CTM, 4/22/14	• VDK 4/23/14 • VDT, 1/20/16	• CTM, 4/25/16

5.0 Rocker Sole Plate Design (Fixed Bearing)

Thickness of Rocker Sole Plate	=	2.00 in	>	1.00 in	AREMA 15-5.3.5.d and NS H-2-5.A.5	OK
Effective Length of Bearing Area	=	minimum of	19.00 in	Eff. Length of Sole Plate (Flange Width + 1:1)	AREMA 15-5.3.5.b	
			26.50 in	actual length of sole plate		
	=	19.00 in				
Line Bearing Stress	=	12.53 kips/in				
Allowable Line Bearing Stress	=	20.70 kips/in			AREMA 15-5.2.1.c	
SOLE PLATE LINE BEARING STRESS CHECK:	12.53 kips/in	<	20.70 kips/in			OK

5.1 Rocker Sole Plate Weld Design (Fixed Bearing)

Longitudinal Force		=	33.33 kips	=μ*load; Coeff. of friction = 0.1; Long. Force if fixed	AREMA 15-5.4.3.b
Allowable Shear Force	R_w	=	2.78 kips/in	=.707 * Fv * w	
Weld Length	L_w	=	14.00 in	= 2 * Sole Plate Width (Weld on each side of flange)	
Longitudinal Force in Welds	R_v	=	2.38 kips/in		
WELD STRESS CHECK:	2.38 kips/in	<	2.78 kips/in		OK

6.0 Pintel Design (Fixed Bearing)

Penetration into the connected elements	=	1.25 in	>	0.25 in	AREMA 15-1.12.1	OK
Shear force per pintel	=	17.97 kips		Two pintels per steel bearing; Resultant of Lateral and longitudinal force is used.		
Cross sectional area of pin in shear	=	1.767 sq. in.				
Shear stress on XS area of pintel	=	10.17 ksi				
Allowable shear force in pins	=	15.75 ksi		0.42Fy	AREMA Table 15-1-11	
PINTEL SHEAR STRESS CHECK	10.17 ksi	<	15.75 ksi			OK

7.0 Masonry Plate Design (Fixed Bearing)

Thickness of Masonry Plate	=	2.50 in	>	1.50 in	AREMA 15-5.3.5.e	OK
Effective Length of Bearing Area	=	minimum of	24.00 in	Eff. Length of Rocker Plate + 2*Masonry Plate Thickness	AREMA 15-5.3.5.b	
			26.50 in	Actual Length of Masonry Plate		
	=	24.00 in				
Effective Width of Bearing Area	=	minimum of	10.00 in	4x Thickness		
			10.00 in	Actual Width of Masonry Plate		
		10.00 in				
Effective Bearing Area	=	240.00 sq.in				
Bearing Stress	=	0.99 ksi				
Allowable Bearing Stress	=	1.05 ksi		Concrete Substructure will control	AREMA 8-2.26.c	
MASONRY PLATE BEARING STRESS CHECK:	0.99 ksi	<	1.05 ksi			OK

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Calculation	AREMA Sliding Brozne and Fixed Rocker Bearing Design (Steel Beam Bridge)
Description	HAM-PROSSER

GF Job No:	ODOT D8	Calculated	Checked	Updated	Review/Upd.	Verified
• 054682	• SFN: TBD	• VDK, 4/15/14	• CTM, 4/22/14	• CTM, 4/22/14	• VDK 4/23/14 • VDT, 1/20/16	• CTM, 4/25/16

8.0 Anchor bolt Design (Fixed Bearing)

ASTM F 1554 standard bolts

Diameter of anchor bolt	=	1.25 in	>	1.25 in	AREMA 15-5.3.7b and e; NS H.2-3-5	OK
Shear force per anchor bolt	=	17.97 kips		Two anchor bolts per steel bearing; Resultant of Lateral and longitudinal force is used.		
Cross sectional area of bolt in shear	=	1.227 sq. in.				
Shear stress on XS area of bolt	=	14.65 ksi				
Allowable shear force in bolts	=	31.25 ksi		0.20Fu * 125% for load combination B1	AREMA 15-5.2.1 c	

ANCHOR BOLT SHEAR STRESS CHECK	14.65 ksi	<	31.25 ksi	OK

By inspection the seismic load per girder is less than the longitudinal force (considering the 50% overstress allowed in load combination E1)

DESIGN CALCULATION COVER SHEET

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PROJECT:	**ODOT/EMH&T: HAM-75-7.85**	*ODOT PID NO:*	*77889*
BRIDGE NO:	**HAM-75-PROS**	*EMH&T JOB NO:*	
SUBJECT:	***Analysis***	**GF JOB NO:**	**54682**
	Concrete Deck	CALC NO.:	**HAM075_PROS_12**
	MATHCAD	REV NO.:	**01**
FILE NAME(s):	*NSRR Standard Concrete Deck Design*	**SHEET:**	**1 OF 13**

SUPERSEDES CALC NO: 00 (Stage 1 Calculation)

CALCULATION DESCRIPTION:

This will analyze the concrete deck to be used on the bridge. The design looks at a 12" strip for the calculations and then applies this to the entire bridge.

DESIGN BASIS OR REFERENCES:

NS Underpass Design Criteria
AREMA (2008) Chapter 8, Section 5
BDM-ODOT LFD Bridge Design Manual (2004) - Current Edition
AISC Steel Construction Manual, Thirteenth Edition
Stage 1 Plans and Comments

MISCELLANEOUS NOTES:

Minimum 8" ballast used for worst case lateral live load distribution

Maximum (9"+6" future wearing surface) ballast used for worst case dead load applied

Maximum projected overhang used for all designs(rounded up

Minimum top flange used for all designs

Stage 3: Deck increased due to Derail Load being increased.

RECORD OF ISSUES:

REV	DESCRIPTION	BY	DATE	VERIF'D	DATE
00	Stage 1 Calculation	VDK	03/26/14	VDK	05/30/14
01	Stage 3 Update	VDT	01/14/16	CTM	04/25/16

RECORD OF CHECK OF CURRENT CALCULATION VERSION

BY	DATE	CHKD	DATE	UPD.	DATE	VERIF'D	DATE
VDK	03/26/14	CTM	04/17/14	VDT	01/20/16	CTM	04/25/16

Reinforced Deck Slab Design

(AREMA/NSRR LFD Deck Design)

Design notes

- Variables with a box around them require review/user input.
- Code references are made using the following abbreviations:
  - AREMA -AREMA Chapter 8 - Current Edition
  - NS - Norfolk Southern Public Policy Manual - Current Edition
  - BDM - ODOT LFD Bridge Design Manual (2004) - Current Edition
  - AA - AASHTO LFD
  - AISC - AISC Steel Construction Manual, Thirteenth Edition

- General Design data and assumptions:
  - A 12" wide strip is designed
  - Minimum (8") ballast is used for a worst-case lateral live load distribution
  - Maximum (9" + 6" future) ballast is used for worst-case dead load applied
  - Maximum project overhang is used for all designs (rounded up)
  - Minimum top flange width (rounded down) is used for all designs
  - Norfolk Southern derailed loads are assumed

1.0 INPUT:

1.1 SECTION PROPERTIES:

Girder Spacing $Spa := 3.5 \cdot ft$

Minimum Girder Flange Width $b_f := 15 \cdot in$

Minimum Girder Flange Thickness $t_f := 2 \cdot in$

Min. Gider Span: $Sp := 64 \cdot ft$

Effective Deck Span Length $Len := Spa - 0.5 \cdot b_f$ $Len = 2.87 \cdot ft$ AREMA 8-2.23.6.c(2)

Clear Cover (AREMA Table 8-2.7) Top of Slab: $Ct := 2.0 \cdot in$ Bottom of Slab: $Cb := 1.0 \cdot in$ *cover to the flange... knowing an additional Tf is below that*

Thickness of Slab for Negative Bending $h_n := 9.5in$ *minimum deck thickness is considered for conservative purposes. In reality, at the negative bending section, the deck will always be thicker due to cross-slope*

Thickness of Slab for Positive Bending $h_p := h_n + t_f = 11.5 \cdot in$

Max. Deck Overhang $Ovh := 36 \cdot in$ Parapet Width $Parp_W := 12 \cdot in$ Slab Design Width $b := 12 \cdot in$

1.2 REINFORCING BAR PROPERTIES:

TRANSVERSE BARS: Bar 1 - Positive bending (BOTTOM); Bar 2 - Negative bending (TOP)
LONGITUDINAL BARS: Bar 3 (BOTTOM) and Bar 4 (TOP)

BAR 1:	#5 Bar	$d_{b1} := 0.625 \cdot in$	$A_{b1} := 0.31 \cdot in^2$	$S1 := 8 \cdot in$	$A_{s1} := A_{b1} \cdot \frac{b}{S1}$	$A_{s1} = 0.47 \cdot in^2$
BAR 2:	#5 Bar	$d_{b2} := .625 \cdot in$	$A_{b2} := 0.31 \cdot in^2$	$S2 := 8 \cdot in$	$A_{s2} := A_{b2} \cdot \frac{b}{S2}$	$A_{s2} = 0.47 \cdot in^2$
BAR 3:	#5 Bar	$d_{b3} := 0.625 \cdot in$	$A_{b3} := 0.31 \cdot in^2$	$S3 := 9 \cdot in$	$A_{s3} := A_{b3} \cdot \frac{b}{S3}$	$A_{s3} = 0.41 \cdot in^2$
BAR 4:	#5 Bar	$d_{b4} := 0.625 \cdot in$	$A_{b4} := 0.31 \cdot in^2$	$S4 := 9 \cdot in$	$A_{s4} := A_{b4} \cdot \frac{b}{S4}$	$A_{s4} = 0.41 \cdot in^2$

1.3 MATERIAL PROPERTIES:

CONCRETE: $f'_c := 4500 \cdot psi$ $\quad \varepsilon_c := 0.003$ $\quad w_{conc} := 150 \cdot \frac{lbf}{ft^3}$ $\quad f_r := 7.5 \cdot \sqrt{f'_c \, psi}$ $\quad E_c := 57000 \cdot \sqrt{f'_c \cdot psi}$

$$\beta_1 := if\left[f'_c \le 4000 \cdot psi, 0.85, 0.85 - 0.05 \cdot \left(\frac{f'_c - 4000 \cdot psi}{1000 \cdot psi}\right)\right] \quad \beta_1 := if\left(\beta_1 \le 0.65, 0.65, \beta_1\right) \quad \beta_1 = 0.83$$

REINFORCING STEEL: $f_y := 60000 \cdot psi$ $\quad E_s := 29000000 \cdot psi$ $\quad n := round\left(\frac{E_s}{E_c}\right)$ $\quad n = 8$

Resistance Factors **Flexure:** $\phi_f := 0.9$

Shear: $\phi_s := 0.85$ AREMA 8-2.52

2.0 LOADS AND FORCES

- *Loads are calculated over a 12" wide strip (transverse to the beams).*
- *Positive and Negative Moment is calculated assuming 3 spans (4 supports/girders) with uniform loading. This is conservative because generally, a railroad bridge will have more than 4 girders; meaning moments and shears are slightly reduced*
- *Negative moment for the exterior deck*

2.1 DEAD & LIVE LOADS APPLIED

Max Live Load Axle (E-80 @ 5'-0" Max Spa) $LL := 80kip$ AREMA Figure 8.2-1

Minimum Ballast $W_{Bal.Min} := 8in \cdot (120pcf) = 80 \cdot psf$ *(Superelevated)*

Maximum Ballast $W_{Bal.Max} := (9 + 6)in \cdot (120pcf) = 150 \cdot psf$ *(Non-Super + Future)* NS H-2.6.D.6

Concrete Deck $W_{Deck} := h_p \cdot (150pcf) = 143.75 \cdot psf$

Concrete Parapet & Handrail (overhang) $W_{Parp} := (Parp_W)(30in + 6in + 12in) \cdot w_{conc} = 600 \cdot plf$

Conservative height, it is assumed that the extra 6 inches of concrete height covers the weight of the handrail and fence and the extra 12" accounts for the deck under the parapet

Track $W_{track} := 200plf$ AREMA 8-2.2.3.b (2)

2.2 TRACK DISTRIBUTION, IMPACT FACTOR & CENTRIFUGAL FORCE

Track Distribution Width: $Dist_W := 8.5ft + 8in = 9.17\,ft$ AREMA 8-2.2.3.c (3)

Track Distribution Length: $Dist_L := \min\left(3.0ft + 8in + 2 \cdot \min\left(h_p - Cb, h_n - Ct\right), 5ft\right) = 4.92\,ft$

AREMA 8-2.2.3.c (2); Assumes worst-case effective slab depth

Typical E-80 Live Load Impact Factor: $LL_{Imp} := 1 + \frac{2.25}{\sqrt{\frac{Sp}{ft}}} = 1.28$

AREMA 8-2.2.3.d (1):
Span between 14' & 127'

NSRR DL over Derail Limits
NS Appendix H.2.6.E.1

$$DL_{DR.min} := [(7.5ft)\cdot(5ft)]\cdot\left(W_{Bal.Min} + W_{Deck}\right) \ldots + (5ft)\cdot W_{track} = 9.39\cdot kip$$

NSRR E-80 Derail Impact Load: $LL_{Derail} := \frac{LL\cdot LL}{LL + DL_{DR.min}} = 71.6\cdot kip$

NS Appendix H.2.6.E.1

Note: A maximum overhang of 3'-3" and a 12" parapet width would mean that the 7'-6" wide derail load would always extend past the second beam in from the outside on a standard NSRR bridge. Due to this, the negative overhang moment will be calculated using the maximum allowable overhang and will assume it is a uniformly loaded beam on two supports (at 3.5' spacing) and a 3.25' overhang. The negative overhang moment will be multiplied by 0.75 to reflect the fact that a 25% overstress is allowed. The worst negative moment (interior under live load + impact or overhang under live load + derail load) will be used for the design of the top mat of steel consistently throughout the project.

Centrifugal Force (% of LL) $LL_{cent} := 0.00117\cdot 20^2\cdot 10 = 4.68$

AREMA 8-2.2.3.e (4)
(S = 20mph, D = 10)

2.3 DEAD LOADS & LIVE LOADS ON THE DESIGN WIDTH OF DECK

Uniform Dead Load $DL_{typ} := b\cdot\left[\left(W_{Bal.Max} + W_{Deck}\right) + \left(\frac{W_{track}}{Dist_W}\right)\right] = 315.57\cdot plf$

Interior Uniform LL+Impact $LL_{typ} := b\cdot\left(\frac{LL}{Dist_W\cdot Dist_L}\right)\cdot LL_{Imp} = 2274.27\cdot plf$

Interior Centrifugal Force $LL_{cf} := \frac{LL_{cent}}{100}\cdot LL_{typ} = 106.44\cdot plf$

Exterior Uniform LL+Derail $LL_{ext} := b\cdot\left[\frac{LL + LL_{Derail}}{(5.0ft)\cdot(7.5ft)}\right] = 4042.56\cdot plf$

NS 6.E Derail Area

2.4 DESIGN MOMENTS

Interior Max Positive DL $M_{int.DL.p} := 0.08\cdot DL_{typ}\cdot Len^2 = 208.67\cdot lbf\cdot ft$

Interior Max Negative DL $M_{int.DL.n} := 0.10\cdot DL_{typ}\cdot Len^2 = 260.84\cdot lbf\cdot ft$

Interior Max Positive LL+I $M_{int.LL.p} := 0.08\cdot LL_{typ}\cdot Len^2 = 1503.86\cdot lbf\cdot ft$

Interior Max Negative LL+I $M_{int.LL.n} := 0.10\cdot LL_{typ}\cdot Len^2 = 1879.82\cdot lbf\cdot ft$

Interior Max Positive CF $M_{int.CF.p} := 0.08\cdot LL_{cf}\cdot Len^2 = 70.38\cdot lbf\cdot ft$

Interior Max Negative CF $M_{int.CF.n} := 0.10\cdot LL_{cf}\cdot Len^2 = 87.98\cdot lbf\cdot ft$

Exterior Parapet Moment $M_{ext.parapet} := b \cdot \left[W_{Parp} \cdot \left(Ovh - 0.5 \cdot Parp_W - 0.25 \cdot b_f\right)\right] = 1312.5 \cdot lbf \cdot ft$ *at face of support*

Exterior Negative DL $M_{ext.DL.n} := 0.5 \cdot DL_{typ} \cdot \left(Ovh - Parp_W - 0.25 \cdot b_f\right)^2 + M_{ext.parapet} = 1761.81 \cdot lbf \cdot ft$

Exterior Negative LL $M_{ext.LL.n} := 0.5 \cdot LL_{ext} \cdot \left(Ovh - Parp_W - 0.25 \cdot b_f\right)^2 = 5755.91 \cdot lbf \cdot ft$

2.5 DESIGN SHEARS

Internal reduction for Shear at critical section 75% of "d" away from edge of flange (conservative)

$$Int_{VRed} := \frac{Len - 0.75\left(h_p - Cb - t_f\right)}{Len} = 0.8152$$

Reduction is to account for the fact that the maximum shear calculated below is for that of the girder edge of the flange. In actuality, the shear can be taken at a location "d" away from the support (edge of flange). In this case, to be conservative, the critical section shear will be calculated at a distance d/2 from the support.

External Reduction for Shear at critical section 75% of "d" away from edge of flange

$$Ext_{VRed} := \frac{Ovh - 0.25 \cdot b_f - 0.75\left(h_p - Cb - t_f\right)}{Ovh - 0.25 \cdot b_f} = 0.8023$$

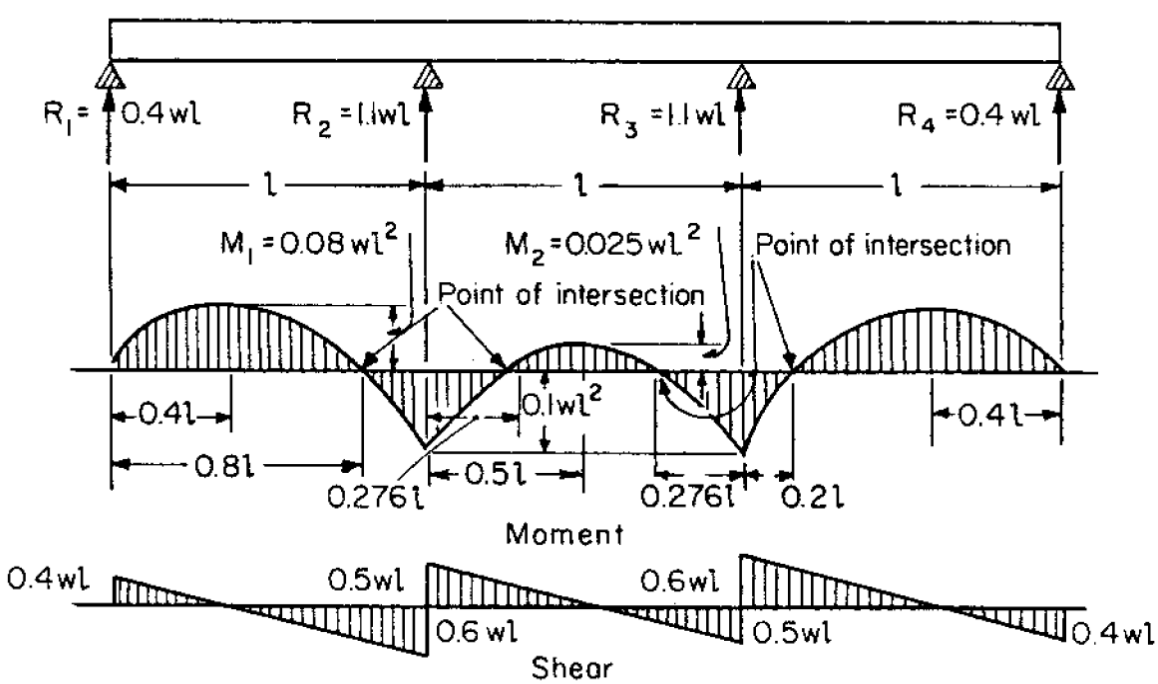
Interior DL $V_{int.DL} := 0.60 \cdot DL_{typ} \cdot Len \cdot Int_{VRed} = 443.77 \cdot lbf$

Interior LL+I $V_{int.LL} := 0.60 \cdot LL_{typ} \cdot Len \cdot Int_{VRed} = 3198.19 \cdot lbf$

Interior CF $V_{int.CF} := 0.60 \cdot LL_{cf} \cdot Len \cdot Int_{VRed} = 149.68 \cdot lbf$

Exterior DL $V_{ext.DL} := Ext_{VRed} \cdot \left[DL_{typ} \cdot \left(Ovh - Parp_W - 0.25 \cdot b_f\right) + \left(W_{Parp} \cdot b\right)\right] = 908.65 \cdot lbf$

Exterior LL $V_{ext.LL} := LL_{ext} \cdot \left(Ovh - Parp_W - 0.25 \cdot b_f\right) \cdot Ext_{VRed} = 5473.32 \cdot lbf$



3.0 LOAD COMBINATIONS TO BE RESISTED

3.1 SERVICE MOMENTS (AREMA TABLE 8-2.4)

Interior Negative $M_{n.int} := M_{int.DL.n} + M_{int.LL.n} + M_{int.CF.n}$ $M_{n.int} = 2.23 \cdot kip \cdot ft$

Exterior Negative $M_{n.ext} := \left(M_{ext.DL.n} + M_{ext.LL.n}\right) \cdot 0.80$ $M_{n.ext} = 6.01 \cdot kip \cdot ft$

Multiplied by 0.80 to account for the fact that this derail load allows for a 25% overstress

Design Negative Moment: $M_n := \max\left(M_{n.int}, M_{n.ext}\right)$ $M_n = 6.01 \cdot kip \cdot ft$

Interior Positive $M_p := M_{int.DL.p} + M_{int.LL.p} + M_{int.CF.p}$ $M_p = 1.78 \cdot kip \cdot ft$

3.2 FACTORED (ULTIMATE) LOADS (AREMA TABLE 8-2.5)

Interior Negative $M_{un.int} := 1.4\left(M_{int.DL.n} + \frac{5}{3} \cdot M_{int.LL.n} + M_{int.CF.n}\right)$ $M_{un.int} = 4.87 \cdot kip \cdot ft$

Exterior Negative $M_{un.ext} := 1.4\left(M_{ext.DL.n} + \frac{5}{3} \cdot M_{ext.LL.n}\right) \cdot 0.80$ $M_{un.ext} = 12.72 \cdot kip \cdot ft$

Multiplied by 0.80 to account for the fact that this derail load allows for a 25% overstress

Design Negative Moment: $M_{un} := \max\left(M_{un.int}, M_{un.ext}\right)$ $M_{un} = 12.72 \cdot kip \cdot ft$

Interior Positive $M_{up} := 1.4\left(M_{int.DL.p} + \frac{5}{3} \cdot M_{int.LL.p} + M_{int.CF.p}\right)$ $M_{up} = 3.90 \cdot kip \cdot ft$

3.3 SERVICE SHEAR (AREMA TABLE 8-2.4)

Interior $V_{s.int} := V_{int.DL} + V_{int.LL} + V_{int.CF}$ $V_{s.int} = 3.79 \cdot kip$

Exterior $V_{s.ext} := \left(V_{ext.DL} + V_{ext.LL}\right) \cdot 0.80$ $V_{s.ext} = 5.11 \cdot kip$

Multiplied by 0.80 to account for the fact that this derail load allows for a 25% overstress

Design Shear $V_s := \max\left(V_{s.int}, V_{s.ext}\right)$ $V_s = 5.11 \cdot kip$

3.4 FACTORED (ULTIMATE) SHEAR (AREMA TABLE 8-2.5)

Interior $V_{u.int} := 1.4\left(V_{int.DL} + \frac{5}{3} \cdot V_{int.LL} + V_{int.CF}\right)$ $V_{u.int} = 8.29 \cdot kip$

Exterior $V_{u..ext} := 1.4\left(V_{ext.DL} + \frac{5}{3} \cdot V_{ext.LL}\right) \cdot 0.80$ $V_{u..ext} = 11.23 \cdot kip$

Multiplied by 0.80 to account for the fact that this derail load allows for a 25% overstress

Design Shear $V_u := \max\left(V_{u.int}, V_{u..ext}\right)$ $V_u = 11.23 \cdot kip$

4.0 CHECK ADEQUACY OF POSITIVE BENDING

Distance to tension steel from top fiber $d_p := h_p - Cb - t_f - 0.5 \cdot d_{b1}$ $d_p = 8.19 \cdot in$

Depth of concrete in compression *(assumed rectangular stress block)* $a_p := \frac{A_{s1} \cdot f_y}{0.85 \cdot f'_c \cdot b}$ $a_p = 0.61 \cdot in$

Design moment capacity $M_{rp} := \phi_f \cdot A_{s1} \cdot f_y \cdot \left(d_p - \frac{a_p}{2}\right)$ $M_{rp} = 16.50 \cdot kip \cdot ft$

Actual Moment $M_{up} = 3.90 \cdot kip \cdot ft$

$Chk_{bend_p} := if\left(M_{rp} \geq M_{up}, "OK", "NO\ GOOD"\right)$ $Chk_{bend_p} = "OK"$

4.1 CHECK THE MINIMUM REINFORCEMENT (AREMA SECTION 8-2.7)

Section Modulus $S_{x.p} := \frac{d_p^2}{6} \cdot b$ $S_{x.p} = 134 \cdot in^3$

Modulus of rupture $f_r = 503.12 \cdot psi$ $M_{rp} = 16.5\ ft \cdot kip$

Cracking moment $M_{cr} := f_r \cdot S_{x.p}$ $M_{cr} = 5.62 \cdot kip \cdot ft$

$Chk_{min_p} := if\left(M_{rp} \geq 1.2 \cdot M_{cr}, "OK", "NO\ GOOD"\right)$ $Chk_{min_p} = "OK"$

4.2 CHECK THE MAXIMUM REINFORCEMENT (AREMA SECTION 8-2.32)

Strain in bottom steel $\varepsilon_{sb} := \frac{f_y}{E_s}$ $\varepsilon_{sb} = 2.07 \times 10^{-3}$

Depth of neutral axis $c_b := \frac{\varepsilon_c \cdot d_p}{\left(\varepsilon_c + \varepsilon_{sb}\right)}$ $c_b = 4.85 \cdot in$

$a_b := c_b \cdot \beta_1$ $a_b = 4 \cdot in$

Balanced reinforcement ratio $\rho_b := \frac{a_b \cdot .85 \cdot f'_c}{d_p \cdot f_y}$ $\rho_b = 0.0311$

Provided reinforcement ratio $\rho := \frac{a_p \cdot .85 \cdot f'_c}{d_p \cdot f_y}$ $\rho = 0.0047$

$Chk_{max_p} := if\left(\rho \leq 0.75 \cdot \rho_b, "OK", "NO\ GOOD"\right)$ $Chk_{max_p} = "OK"$

4.3 CHECK THE SPACING OF THE REINFORCEMENT (AREMA SECTION 8-2.5)

Maximum spacing $s_{max.p} := \min(18 \cdot in, 1.5 \cdot h_p)$ $s_{max.p} = 17.25 \cdot in$

Minimum spacing $s_{min.p} := \max(1.5 \cdot in, 1.5 \cdot d_{b1})$ $s_{min.p} = 1.5 \cdot in$

$Chk_{Spacing_p} := \text{if}(S1 \le s_{max.p} \wedge S1 \ge s_{min.p}, \text{"OK"}, \text{"NO GOOD"})$ $Chk_{Spacing_p} = \text{"OK"}$

4.4 CHECK DISTRIBUTION OF FLEXURAL STEEL (AREMA SECTION 8-2.39)

$Z := 130 \cdot \frac{kip}{in}$ *Based on severe exposure conditions*

Thickness of concrete cover from extreme tension fiber to center of bar $d_{c.p} := \min\left(2in + \frac{d_{b1}}{2}, t_f + Cb + \frac{d_{b1}}{2}\right)$ $d_{c.p} = 2.31 \cdot in$

Effective tension area of concrete surrounding main bar $A_{s.p} := 2 \cdot d_{c.p} \cdot S1$ $A_{s.p} = 37 \cdot in^2$

Allowable stress in reinforcement steel $f_{s_allow.p} := \min\left(0.5 \cdot f_y, \frac{Z}{\sqrt[3]{d_{c.p} \cdot A_{s.p}}}\right)$ $f_{s_allow.p} = 29.5 \cdot ksi$

Reinforcement ratio $\rho_p := \frac{A_{s1}}{d_p \cdot b}$ $\rho_p = 0.0047$

Position of neutral axis $K_p := \sqrt{(\rho_p \cdot n)^2 + 2 \cdot \rho_p \cdot n} - \rho_p \cdot n$ $K_p = 0.24$

$K_p \cdot d_p = 1.96 \cdot in$

Distance from C.G. to extreme tension fiber $c_p := d_p - K_p \cdot d_p$ $c_p = 6.22 \cdot in$

Effective Moment of Inertia of section
Assumes zero compression reinforcement (Conservative)
$I_p := \frac{b \cdot (K_p \cdot d_p)^3}{3} + n \cdot A_{s1} \cdot c_p^2$ $I_p = 174.39 \cdot in^4$

Actual stress in reinforcement
at service load
$f_{s_act.p} := \frac{n \cdot M_p \cdot c_p}{I_p}$ $f_{s_act.p} = 6.11 \cdot ksi$

$Chk_{flex_p} := \text{if}(f_{s_act.p} \le f_{s_allow.p}, \text{"OK"}, \text{"NO GOOD"})$ $Chk_{flex_p} = \text{"OK"}$

4.5 CHECK OVERALL DESIGN FOR POSITIVE BENDING:

$Chk_{positive} := \text{if}(Chk_{bend_p} = \text{"OK"} \wedge Chk_{min_p} = \text{"OK"} \wedge Chk_{max_p} = \text{"OK"} \wedge Chk_{flex_p} = \text{"OK"}, \text{"OK"}, \text{"NO GOOD"})$

$Chk_{positive} = \text{"OK"}$

5.0 DETERMINE THE MOMENT CAPACITY OF THE SLAB IN NEGATIVE BENDING

Distance to tension steel from bottom fiber $d_n := h_n - Ct - 0.5 \cdot d_{b2}$ $d_n = 7.19 \cdot in$

Depth of concrete in compression
(assumed rectangular stress block)
$a_n := \frac{A_{s2} \cdot f_y}{0.85 \cdot f'_c \cdot b}$ $a_n = 0.61 \cdot in$

Design moment capacity $M_{rn} := \phi_f \cdot A_{s2} \cdot f_y \cdot \left(d_n - \frac{a_n}{2}\right)$ $M_{rn} = 14.4 \cdot kip \cdot ft$

Actual Moment $M_{un} = 12.72 \cdot kip \cdot ft$

$Chk_{bend_n} := if\left(M_{rn} \geq M_{un}, "OK", "NO\ GOOD"\right)$ $Chk_{bend_n} = "OK"$

5.1 CHECK THE MINIMUM REINFORCEMENT (AREMA SECTION 8-2.7)

Section Modulus $S_{x.n} := \frac{d_n^{\ 2}}{6} \cdot b$ $S_{x.n} = 103 \cdot in^3$

Modulus of rupture $f_r = 503.12 \cdot psi$ $M_{rn} = 14.4\ ft \cdot kip$

Cracking moment $M_{cr.n} := f_r \cdot S_{x.n}$ $M_{cr.n} = 4.33 \cdot kip \cdot ft$

$Chk_{min_n} := if\left(M_{rn} \geq 1.2 \cdot M_{cr.n}, "OK", "NO\ GOOD"\right)$ $Chk_{min_n} = "OK"$

5.2 CHECK THE MAXIMUM REINFORCEMENT (AREMA SECTION 8-2.32)

Strain in the steel reinforcement $\varepsilon_{sb} = 2.07 \times 10^{-3}$

Strain in the concrete $\varepsilon_c = 3 \times 10^{-3}$

Depth of neutral axis $c_{b.n} := \frac{\varepsilon_c \cdot d_n}{\left(\varepsilon_c + \varepsilon_{sb}\right)}$ $c_{b.n} = 4.25 \cdot in$

$a_{b.n} := c_{b.n} \cdot \beta_1$ $a_{b.n} = 3.51 \cdot in$

Balanced reinforcement ratio $\rho_{b.n} := \frac{a_{b.n} \cdot .85 \cdot f'_c}{d_n \cdot f_y}$ $\rho_{b.n} = 0.0311$

Provided reinforcement ratio $\rho_n := \frac{a_n \cdot .85 \cdot f'_c}{d_n \cdot f_y}$ $\rho_n = 0.0054$

$Chk_{max_n} := if\left(\rho_n \leq 0.75 \cdot \rho_{b.n}, "OK", "NO\ GOOD"\right)$ $Chk_{max_n} = "OK"$

5.3 CHECK THE SPACING OF THE REINFORCEMENT (AREMA SECTION 8-2.5)

Maximum Spacing $s_{max.n} := \min(18 \cdot in, 1.5 \cdot h_n)$ $s_{max.n} = 14.25 \cdot in$

Minimum Spacing $s_{min.n} := \max(1.5 \cdot in, 1.5 \cdot d_{b2})$ $s_{min.n} = 1.5 \cdot in$

$Chk_{Spacing_n} := \text{if}(S1 \le s_{max.n} \wedge S1 \ge s_{min.n}, \text{"OK"}, \text{"NO GOOD"})$ $Chk_{Spacing_n} = \text{"OK"}$

5.4 CHECK DISTRIBUTION OF FLEXURAL STEEL (AREMA SECTION 8-2.39)

$Z := 130 \cdot \frac{kip}{in}$ *Based on severe exposure conditions*

Thickness of concrete cover from extreme tension fiber to center of bar $d_{c.n} := \min\left(2in + \frac{d_{b2}}{2}, Ct + \frac{d_{b2}}{2}\right)$ $d_{c.n} = 2.31 \cdot in$

Effective tension area of concrete surrounding main bar $A_{s.n} := 2 \cdot d_{c.n} \cdot S2$ $A_{s.n} = 37 \cdot in^2$

Allowable stress in reinforcement steel $f_{s_allow.n} := \min\left(0.5 \cdot f_y, \frac{Z}{\sqrt[3]{d_{c.n} \cdot A_{s.n}}}\right)$ $f_{s_allow.n} = 29.5 \cdot ksi$

Reinforcement ratio $\rho_n := \frac{A_{s2}}{d_n \cdot b}$ $\rho_p = 0.0047$

Position of neutral axis $K_n := \sqrt{(\rho_n \cdot n)^2 + 2 \cdot \rho_n \cdot n} - \rho_n \cdot n$ $K_n = 0.25$

$K_n \cdot d_n = 1.82 \cdot in$

Distance from C.G. to extreme tension fiber $c_n := d_n - K_n \cdot d_n$ $c_n = 5.36 \cdot in$

Effective Moment of Inertia of section
Assumes zero compression reinforcement (Conservative)
$I_n := \frac{b \cdot (K_n \cdot d_n)^3}{3} + n \cdot A_{s2} \cdot c_n^2$ $I_n = 131.29 \cdot in^4$

Actual stress in reinforcement
at service load
$f_{s_act.n} := \frac{n \cdot M_n \cdot c_n}{I_n}$ $f_{s_act.n} = 23.59 \cdot ksi$

$Chk_{flex_n} := \text{if}(f_{s_act.n} \le f_{s_allow.n}, \text{"OK"}, \text{"NO GOOD"})$ $Chk_{flex_n} = \text{"OK"}$

5.5 CHECK OVERALL DESIGN FOR NEGATIVE BENDING:

$Chk_{negative} := \text{if}(Chk_{bend_n} = \text{"OK"} \wedge Chk_{min_n} = \text{"OK"} \wedge Chk_{max_n} = \text{"OK"} \wedge Chk_{flex_n} = \text{"OK"}, \text{"OK"}, \text{"NO GOOD"})$

$Chk_{negative} = \text{"OK"}$

6.0 DETERMINE THE REQUIRED LONGITUDINAL (DISTRIBUTION) REINFORCEMENT AND SPACING:

6.1 CHECK REQUIRED DISTRIBUTION REINFORCEMENT PER AASHTO 9.7.3.2:

$$A_{Distrib} := \min\left(\frac{67}{100}, \frac{220 \cdot in}{\sqrt{Len \cdot in}}\right) \cdot A_{s1}$$

$$A_{Distrib} = 0.31 \cdot in^2$$

$$Chk_{AASHTO} := if\left(A_{s3} \geq A_{Distrib} \wedge A_{s4} \geq A_{Distrib}, \text{"OK"}, \text{"NO GOOD"}\right)$$

$$Chk_{AASHTO} = \text{"OK"}$$

6.2 CHECK REQUIRED SHIRINKAGE & TEMPERATURE REINFORCEMENT PER AREMA SECTION 8-2.12:

$$A_{ShrnkTemp} := 0.25 \cdot in^2$$

$$Chk_{A_AREMA} := if\left(A_{s3} \geq A_{ShrnkTemp} \wedge A_{s4} \geq A_{ShrnkTemp}, \text{"OK"}, \text{"NO GOOD"}\right)$$

$$Chk_{A_AREMA} = \text{"OK"}$$

$$S_{ShrnkTemp} := \min\left(18 \cdot in, 3 \cdot h_n\right)$$

$$S_{ShrnkTemp} = 18 \cdot in$$

$$Chk_{S_AREMA} := if\left(S3 \leq S_{ShrnkTemp} \wedge S4 \leq S_{ShrnkTemp}, \text{"OK"}, \text{"NO GOOD"}\right)$$

$$Chk_{S_AREMA} = \text{"OK"}$$

$$Chk_{Distrib} := if\left(Chk_{AASHTO} = \text{"OK"} \wedge Chk_{A_AREMA} = \text{"OK"} \wedge Chk_{S_AREMA} = \text{"OK"}, \text{"OK"}, \text{"NO GOOD"}\right)$$

$$Chk_{Distrib} = \text{"OK"}$$

7.0 CHECK SHEAR CAPACITY:

7.1 DETERMINE SHEAR CAPACITY OF THE SLAB (AREMA SECTION (8-2.35)

In most designs with average overhangs, the negative moment section will control the shear capacity at the overhang under the NSRR derail condition.

$\lambda := 1$ *(Normal Weight Concrete)*

Shear stress carried by concrete shall not exceed

$V_{c.max} := 2 \cdot \lambda \sqrt{f'_c \cdot psi}$ $\qquad V_{c.max} = 134.16 \cdot psi$

Ratio of reinforcement

$\rho_v := \frac{A_{s2}}{b \cdot d_p} = 0.0047$ $\qquad d_p = 8.19 \cdot in$

Shear stress that can be carried by concrete sections subject to shear & flexure only

$V_{c.max.alt} := 1.9 \cdot \lambda \sqrt{f'_c \cdot psi} + 2500psi \cdot \rho_v \cdot min\left(1, \frac{V_u \cdot d_p}{M_{un}}\right)$

$V_u = 11.23 \cdot kip$ $\qquad V_{c.max.alt} = 134.59 \cdot psi$

Factored Shear stress

$V_{u.max} := \frac{V_u}{\phi_s \cdot b \cdot d_p}$ $\qquad V_{u.max} = 134.53 \cdot psi$

$Chk_{Vc} := if\left(max\left(V_{c.max.alt}, V_{c.max}\right) \geq V_{u.max}, "OK", "NO\ GOOD"\right)$ $\qquad Chk_{Vc} = "OK"$

NO SHEAR REINFORCEMENT REQUIRED IN SLABS PER AREMA CHAPTER 8 SECTION 2.10.1.a

7.2 CHECK PUNCHING SHEAR

As there is no reference of Rail contact area in AREMA,
Train wheel estimated as 6" x 12" contact area.

$A_{pll} := 6 \cdot in$ $\qquad A_{perp} := 12 \cdot in$ $\qquad V_{ps} := 60kip$

(1/2 of Rail Axle Load)

$AREA := A_{pll} \cdot A_{perp}$ $\qquad AREA = 72 \cdot in^2$

$D := h_p - Cb - .5 \cdot d_{b1}$ $\qquad D = 10.19 \cdot in$

Critical Area Located D/2 from Contact Area
AREMA 8-2.35.6 a

$CA_{pll} := A_{pll} + D$ $\qquad CA_{pll} = 16.19 \cdot in$

$CA_{perp} := A_{perp} + D$ $\qquad CA_{perp} = 22.19 \cdot in$

Ratio of long side to short side $\beta_c := \frac{CA_{perp}}{CA_{pll}}$ $\beta_c = 1.37$

Interior Reaction Area $\alpha_s := 40$

Perimeter of the critical section $b_o := 2(CA_{pll} + CA_{perp})$ $b_o = 6.4 \cdot ft$

Allowable shear force

$$V_{ps_c} := \min\left[4 \cdot \lambda \sqrt{f'_c \cdot psi} \cdot b_o \cdot D, \left(2 + \frac{4}{\beta_c}\right) \cdot \lambda \sqrt{f'_c \cdot psi} \cdot b_o \cdot D, \left(2 + \frac{\alpha_s \cdot D}{b_o}\right) \cdot \lambda \sqrt{f'_c \cdot psi} \cdot b_o \cdot D\right] \qquad V_{ps_c} = 209.8 \cdot kip$$

$Chk_{V_ps} := \text{if}\left(V_{ps} \le \phi_s \cdot V_{ps_c}, \text{"OK"}, \text{"NO GOOD"}\right)$ $Chk_{V_ps} = \text{"OK"}$

7.3 CHECK OVERALL DESIGN FOR SHEAR

$Chk_V := \text{if}\left(Chk_{Vc} = \text{"OK"} \wedge Chk_{V_ps} = \text{"OK"}, \text{"OK"}, \text{"NO GOOD"}\right)$ $Chk_V = \text{"OK"}$

8.0 CHECK FULL DESIGN:

$DESIGN := \text{if}\left(Chk_{positive} = \text{"OK"} \wedge Chk_{negative} = \text{"OK"} \wedge Chk_{Distrib} = \text{"OK"} \wedge Chk_V = \text{"OK"}, \text{"OK"}, \text{"NO GOOD"}\right)$

$DESIGN = \text{"OK"}$

DESIGN CALCULATION COVER SHEET

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PROJECT:	**ODOT/EMH&T: HAM-75-7.85**	*ODOT PID NO:*	*77889*
BRIDGE NO:	**HAM-75-PROS**	*EMH&T JOB NO:*	
SUBJECT:	***Analysis***	**GF JOB NO:**	**54682**
	Shear Stud design	CALC NO.:	**HAM075_PROS_13**
	MATHCAD	REV NO.:	**01**
FILE NAME(s):	*HAM075_Prosc_13 Studs.xmcd*	**SHEET:**	**1 OF 8**

SUPERSEDES CALC NO: 00 (Stage 1 Calculation)

CALCULATION DESCRIPTION:

This will analyze and design the shear stud connectors for composite action of the deck slab.

DESIGN BASIS OR REFERENCES:

NS Underpass Design Criteria
AREMA (2014) Chapter 15
HAM075_PROS_03
AISC Steel Construction Manual, Thirteenth Edition
BDM - ODOT Bridge Design Manual (2004) - Current Edition

MISCELLANEOUS NOTES:

Stage 3 Update: Updated girder section properties

RECORD OF ISSUES:

REV	DESCRIPTION	BY	DATE	VERIF'D	DATE
00	Stage 1 Calculation	VDK	04/10/14	VDK	05/30/14
01	Stage 3 Update	VDT	01/20/16	CTM	04/25/16

RECORD OF CHECK OF CURRENT CALCULATION VERSION

BY	DATE	CHKD	DATE	UPD.	DATE	VERIF'D	DATE
VDK	04/10/14	CTM	04/18/14	VDT	01/20/16	CTM	04/25/16

SHEAR STUD CONNECTORS

(AREMA/NSRR Design: Prosser Avenue Bridge)

Design notes

- Variables with a box around them require review/user input.
- Code references are made using the following abbreviations:
  - AREMA -AREMA Chapter 15 - Current Edition
  - NS - Norfolk Southern Public Policy Manual - Current Edition
  - BDM - ODOT LFD Bridge Design Manual (2004) - Current Edition
  - AA - AASHTO LFD
  - AISC - AISC Steel Construction Manual, Thirteenth Edition

Design the shear stud configuration for the girder. Check for the adequacy of the stud connectors.

1.0 INPUT DATA

1.1 GIRDER DATA

Girder Flange Width: $B_f := 15in$

Girder Web Thickness: $T_w := 0.875in$ HAM-75-PROS_03

Girder Flange Thickness: $T_f := 2.75in$

Depth of web: $D_w := 44in$

Web area: $A_w := T_w \cdot D_w = 38.5 \cdot in^2$

Flange area: $A_f := B_f \cdot T_f = 41.25 \cdot in^2$

Total area of girder: $A_g := A_w + 2 \cdot A_f = 121 \cdot in^2$

Span length (C/C Bearings): $Len := 64.0ft$

Lateral Spacing of the beams (C/C): $Spa := 3.5ft$

Thickness of the concrete slab: $thk := 8.5in$ *effective slab thickness is taken as slab thickness minus one inch.*

1.2 SHEAR CONNECTOR DATA

studs transverse spacing: $spa_{trans} := 4in$

Height of stud: $H_{stud} := thk - 2in = 6.5 \cdot in$ AREMA 15-1.7.9.3.d

Diameter of stud: $Dia_{stud} := 0.875in$

Nominal cross section area of the stud: $XS_{stud} := 3.142 \cdot 0.25 \cdot Dia_{stud}^2 = 0.6 \cdot in^2$

Number of studs per row: $N_{stud} := 3$

Height to diameter ratio: $Ratio := \frac{H_{stud}}{Dia_{stud}} = 7.43$ AREMA 15-1.7.9.3.3.a

$CHK_{HeightRatio} := if(Ratio > 4, "OK", "NO GOOD")$ $CHK_{HeightRatio} = "OK"$

1.3 LOADING DATA

Shear stud designs are verified at every tenth point along the beam length (L).

Rows $i := 1 .. 11$ **Columns** $j := 1 .. 2$ $ORIGIN := 1$ $V_{LL_MAX} := 183.12$ $V_{DL_MAX} := 30.50$

Unfactored shears and moments $V_{LL_MIN} := 60$ $V_{DL_MIN} := 10$

GIRDER DESIGN spreadsheet HAM075_PROS_03

Minimum dead load for 3.5' of tributary area due to local load of beam weight, concrete deck, and ballast

$$X := \begin{pmatrix} 0 \\ 0.1\cdot Len \\ 0.2\cdot Len \\ 0.3\cdot Len \\ 0.4\cdot Len \\ 0.5\cdot Len \\ 0.6\cdot Len \\ 0.7\cdot Len \\ 0.8\cdot Len \\ 0.9\cdot Len \\ Len \end{pmatrix}$$

$$V_{LL_I} := \begin{pmatrix} \max(1.0\cdot V_{LL_MAX}, V_{LL_MIN}) & 0 \\ \max(0.8\cdot V_{LL_MAX}, V_{LL_MIN}) & 0 \\ \max(0.6\cdot V_{LL_MAX}, V_{LL_MIN}) & 0 \\ \max(0.4\cdot V_{LL_MAX}, V_{LL_MIN}) & 0 \\ \max(0.2\cdot V_{LL_MAX}, V_{LL_MIN}) & 0 \\ \max(0.00\cdot V_{LL_MAX}, V_{LL_MIN}) & 0 \\ \max(0.2\cdot V_{LL_MAX}, V_{LL_MIN}) & 0 \\ \max(0.4\cdot V_{LL_MAX}, V_{LL_MIN}) & 0 \\ \max(0.6\cdot V_{LL_MAX}, V_{LL_MIN}) & 0 \\ \max(0.8\cdot V_{LL_MAX}, V_{LL_MIN}) & 0 \\ \max(1.0\cdot V_{LL_MAX}, V_{LL_MIN}) & 0 \end{pmatrix} kip$$

$$V_{DL} := \begin{pmatrix} \max(1.0\cdot V_{DL_MAX}, V_{DL_MIN}) \\ \max(0.8\cdot V_{DL_MAX}, V_{DL_MIN}) \\ \max(0.6\cdot V_{DL_MAX}, V_{DL_MIN}) \\ \max(0.4\cdot V_{DL_MAX}, V_{DL_MIN}) \\ \max(0.2\cdot V_{DL_MAX}, V_{DL_MIN}) \\ \max(0.0\cdot V_{DL_MAX}, V_{DL_MIN}) \\ \max(0.2\cdot V_{DL_MAX}, V_{DL_MIN}) \\ \max(0.4\cdot V_{DL_MAX}, V_{DL_MIN}) \\ \max(0.6\cdot V_{DL_MAX}, V_{DL_MIN}) \\ \max(0.8\cdot V_{DL_MAX}, V_{DL_MIN}) \\ \max(1.0\cdot V_{DL_MAX}, V_{DL_MIN}) \end{pmatrix} kip$$

$V_{LL_I} =$

183.12	0
146.5	0
109.87	0
73.25	0
60	0
60	0
60	0
73.25	0
109.87	0
146.5	0
183.12	0

$\cdot kip$

$V_{DL} =$

30.5
24.4
18.3
12.2
10
10
10
12.2
18.3
24.4
30.5

$\cdot kip$

TOTAL DL = 64.31 kips
V.DL = 64.31 - 15.13 (steel) - 18.68 (Non-composite) = 30.5 kips

TOTAL LL = 122.62 + 51.22 + 2.03 + 7.25 = 183.12 kips
Min. (maximum) LL = 1/2 axle load + 50% Impact = 60 kips
Minimum LL = unloaded = 0 kips (No negative moment - simple span)

Intermediate supports are not provided during the casting and curing of the concrete. Therefore, self weight is not considered in maximum shear force as described in AREMA 15-1.7.9.2.h

1.4 MATERIAL PROPERTIES

Concrete Strength: $f'_c := 4500 \cdot psi$

Concrete Unit Weight: $\gamma_c := 150 \cdot pcf$

Steel Beam Yield Strength: $F_{y_girder} := 50ksi$

Deck Rebar Yield Strength: $F_{y_rebar} := 60ksi$

Top Concrete Clear Cover: $cc_{top} := 2.0in$

AREMA Table 8-2.7

Bott. Concrete Clear Cover: $cc_{bot} := 1.50in$

Modulus of Elasticity - Girder: $E_s := 29000ksi$

Modulus of Elasticity - Concrete: $E_c := 57000 \cdot \sqrt{f'_c \cdot psi} = 3823.68\ ksi$

Modulus ratio for n: $n := round\left(\frac{E_s}{E_c}\right) = 8$

2.0 SHEAR STUD DESIGN

2.1 TRANSFORMED COMPOSITE SECTION PROPERTIES

Effective width of flange on either side of beam: $b_{eff} := \min(Spa \cdot 0.5, 0.125 \cdot Len, 6 \cdot thk) = 1.75\,ft$ AREMA 15-1.7.9.2 c

Total effective flange width: $B_{f.eff} := 2 \cdot b_{eff} = 3.5\,ft$

Transformed compressive concrete area: $A_{c.tra} := \frac{B_{f.eff} \cdot thk}{n} = 44.62 \cdot in^2$

Neutral axis of the composite section from bottom of beam:

$$y_{NA} := \frac{B_f \cdot T_f \cdot T_f \cdot 0.5 + D_w \cdot T_w \cdot (0.5 \cdot D_w + T_f) + B_f \cdot T_f \cdot (D_w + T_f \cdot 1.5) + A_{c.tra} \cdot (D_w + 2 \cdot T_f + 0.5 \cdot thk)}{2B_f \cdot T_f + D_w \cdot T_w + A_{c.tra}} \qquad y_{NA} = 32.56 \cdot in$$

MOI of bottom flange: $I_{bf} := \frac{(B_f \cdot T_f^3)}{12} + (T_f \cdot B_f) \cdot (y_{NA} - 0.5 \cdot T_f)^2 \qquad I_{bf} = 40151.02 \cdot in^4$

MOI of web section: $I_w := \left(\frac{T_w \cdot D_w^3}{12}\right) + (D_w \cdot T_w) \cdot (T_f + 0.5 \cdot D_w - y_{NA})^2 \qquad I_w = 8561.84 \cdot in^4$

MOI of top flange: $I_{tf} := \frac{(B_f \cdot T_f^3)}{12} + (T_f \cdot B_f) \cdot (D_w + 1.5 \cdot T_f - y_{NA})^2 \qquad I_{tf} = 10015 \cdot in^4$

MOI of transformed concrete: $I_{tc} := \frac{\left(\frac{B_{f.eff}}{n} \cdot thk^3\right)}{12} + A_{c.tra} \cdot (D_w + 2 \cdot T_f + 0.5 \cdot thk - y_{NA})^2 \qquad I_{tc} = 20299.24 \cdot in^4$

Total moment of inertia of composite section: $I_{comp} := I_{bf} + I_{tf} + I_w + I_{tc} \qquad I_{comp} = 79027.1 \cdot in^4$

Static moment of the transformed compressive concrete area about N.A. of composite section: $Q_{cc} := A_{c.tra} \cdot (D_w + 2 \cdot T_f + 0.5thk - y_{NA}) = 945.44 \cdot in^3$

2.2 SPACING OF SHEAR STUDS

2.2.1 Fatigue

Number of Stress Cycles > 2,000,000 AREMA Table 15-1.7.3.1.b

Allowable horizontal fatigue shear force range: $S_{r.des.stud} := 7000psi \cdot XS_{stud} = 4.21 \cdot kip$

Range of Vertical shear due to unfactored live load and impact (Difference in the max and min shear envelopes excluding dead loads): $V_{r_i} := V_{LL_I_{i,1}} - V_{LL_I_{i,2}}$

Range of horizontal shear: $S_{r_i} := V_{r_i} \cdot \frac{Q_{cc}}{I_{comp}}$

Spacing of Shear Studs (Fatigue): $spa_{fatigue_i} := \frac{N_{stud} \cdot S_{r.des.stud}}{S_{r_i}}$

2.2.2 Strength

Strength of Individual Shear Connector: $S_{m.des.stud} := 20000psi \cdot XS_{stud} = 12.03 \cdot kip$

Maximum vertical shear due to live load and impact load combined with any portion of dead load superimposed on composite span: $V_{m_i} := V_{LL_I_{i,1}} + V_{DL_i}$

Maximum horizontal shear: $S_{m_i} := V_{m_i} \cdot \frac{Q_{cc}}{I_{comp}}$

Spacing of Shear Studs (Strength): $spa_{strength_i} := \frac{N_{stud} \cdot S_{m.des.stud}}{S_{m_i}}$

2.2.3 Check Spacing

Clear distance between beam flange and edge of shear connector:

$$Distance_{clear} := B_f \cdot 0.5 - spa_{trans} - Dia_{stud} \cdot 0.5 = 3.06 \cdot in$$

$$check_{trans} := if\left(Distance_{clear} > 1.5in \wedge spa_{trans} > 4 \cdot Dia_{stud}, "OK", "NO\ GOOD"\right)$$

AREMA 15-1.7.9.3.e and AA 6.10.10.1.3

$$check_{trans} = "OK"$$

Design Spacing of Shear Studs:

$$spa_{max_i} := 24 \cdot in$$

$$spa_{long_i} := if\left(spa_{strength_i} > spa_{fatigue_i}, spa_{fatigue_i}, spa_{strength_i}\right)$$

$$spa_{final_i} := min\left(spa_{max_i}, spa_{strength_i}, spa_{fatigue_i}\right)$$

AREMA 15-1.7.9.3.b

$$check_{long_i} := if\left(spa_{final_i} \le spa_{max_i} \wedge spa_{final_i} > 6 \cdot Dia_{stud}, "OK", "NO\ GOOD"\right)$$

$X_i =$	$spa_{fatigue_i} =$	$spa_{strength_i} =$	$spa_{long_i} =$	$spa_{final_i} =$	$check_{long_i} =$
0 ft	5.76 ·in	14.12 ·in	5.76 ·in	5.76 ·in	"OK" ·in
6.4	7.21	17.65	7.21	7.21	"OK"
12.8	9.61	23.53	9.61	9.61	"OK"
19.2	14.41	35.3	14.41	14.41	"OK"
25.6	17.59	43.09	17.59	17.59	"OK"
32	17.59	43.09	17.59	17.59	"OK"
38.4	17.59	43.09	17.59	17.59	"OK"
44.8	14.41	35.3	14.41	14.41	"OK"
51.2	9.61	23.53	9.61	9.61	"OK"
57.6	7.21	17.65	7.21	7.21	"OK"
64	5.76	14.12	5.76	5.76	"OK"

Gannett Fleming Engineers and Architects, P.C. — Excellence Delivered As Promised		
Calculation	Stage 3 Quantities: Norfolk Southern Railroad Bridge	
Description	HAM-075-PROSC, NSRR over Prosser avenue (Single Track Deck Girder)	

GF Job No:	Bridge SFN	Calculated	Checked	Updated	Verified	Level 2 Review
• 054682	• XXXXXXX	• SNH, 10/31/14	• CTM, 11/10/14	• SNH, 11/17/14 • VDT, 04/18/16	• CTM, 11/19/14 • CTM, 4/25/16	•

This calculation will use ODOT Bid Elements. Reinforcing ratios will be used to determine reinforcing until the Stage 3 reinforcing calculations are complete. Framing is normal to the chord for each span, with end diaphragms being placed along the skew angle at bearing locations.

INPUT: Superstructure

General Data

Minimum Deck Thickness:	9.500 in	Curb Thickness:	1.000 ft
Deck Cross Slope:	1.55%	Curb Height:	2.000 ft
Typical C/C Drains:	14.000 ft	Total Number of Girder Lines:	6 girder(s)
		Girder Spacing, Cent-Cent:	3.500 ft
Diaphragm Total Depth:	40.500 in		
Diaphragm Web Thickness:	0.500 in	Diaphragm Flange Thickness:	0.500 in
Diaphragm Drain Hole Area:	178.54 SqIn	Diaphragm Flange Width:	6.000 in
End Diaphragm Hole Area:	311.7 SqIn	No. of Lines of Diaphragms with Holes:	5 lines
LLB Member Weight (none):	0.00 plf	Bays containing LLB:	0 bays

CIP Deck Reinforcing Ratio:	270 lb/cy	Abutment & W.W. Reinforcing Ratio:	120 lb/cy
Pier Cap Reinforcing Ratio:	200 lb/cy	Footing Reinforcing Ratio:	140 lb/cy
Abutment Backwall Reinforcing Ratio:	300 lb/cy	Parapet/Curb Reinforcing Ratio:	170 lb/cy

Span #1 Specific Data

Design Girder (Chord) Length:	64.000 ft	C/C Bearing	Total Concrete Deck Length:	67.563 ft	Parallel to Chord
Length of Girder Overhangs (Total):	2.917 ft		Deck Width, Out-Out:	23.500 ft	Normal to Chord
			Span #1 Skew:	20.776 deg	0.36261 rad
Intermediate Dprm Conn Thickness:	0.500 in		Girder Total Depth:	48.000 in	
Intermediate Dprm Conn Width:	5.000 in		Girder Flange Width:	15.000 in	
Intermediate Dpr Conn Spacing:	10.500 ft		Girder Flange Thickness:	2.750 in	
Bearing Stiffener Thickness:	0.750 in		Girder Web Thickness:	0.875 in	
Bearing Stiffener Width:	7.000 in		Typical Stiffener Clip	1.250 in	
Typical Diaphragm Spacing:	10.50 ft	NSRR Max is 12'	Girder Paint Perimeter	133.75 in	Excl. Top Flng
LLB Paint Perimeter:	0.0 in				

INPUT: Substructure

Rear (East) Abutment

Bottom of Abutment Footing Elevation:	521.00		Footing Length:	-	
Top of Stem Elevation:	546.88		Footing Width:	-	
Top of Backwall Shoulder Elevation:	554.31		Footing Toe Width:	5.000 ft	
Median Top of Backwall Elevation:	552.44	Betwn Curbs	Footing Toe Thickness:	5.500 ft	
Pile Tip Elevation:	448.00		Pile Embedment into Footing:	2.000 ft	
Finished Ground Elevation:	528.60		Footing Heel Thickness	5.250 ft	
			Backwall Length (Along Skew):	22.995 ft	Betwn Curbs
Stem Thickness:	5.750 ft		Stem Length:	32.000 ft	Total
Backwall Thickness:	1.750 ft		Backwall Extension Plan Area:	21.99 SqFt	
Pile Row #1 Batter:	4.0 on 1	103.08%	Pile Row #2 Batter:	4.0 on 1	103.08%
Pile Row #1 Piles:	21 ea		Pile Row #2 Piles:	19 ea	
Pile Row #3 Batter:	4.0 on 1	103.08%	Pile Row #4 Batter:	Vertical	100.00%
Pile Row #3 Piles:	12 ea	8 Vert	Pile Row #4 Piles:	8 ea	

This calculation will use ODOT Bid Elements. Reinforcing ratios will be used to determine reinforcing until the Stage 3 reinforcing calculations are complete. Framing is normal to the chord for each span, with end diaphragms being placed along the skew angle at bearing locations.

Rear Wingwall Data (Facing the front face of abutment)

Left Wingwall			Right Wingwall		
Total Wingwall Length:	26.250 ft		Total Wingwall Length:	41.250 ft	
Wingwall Shoulder Length:	5.750 ft		Wingwall Shoulder Length:	5.750 ft	
Wingwall Shoulder Elevation:	554.31		Wingwall Shoulder Elevation:	554.31	
Wingwall Lowest Elevation:	539.00		Wingwall Lowest Elevation:	546.00	
Average Battered Wingwall Thickness:	2.578 ft		Average Battered Wingwall Thickness:	2.742 ft	
Pile Cutoff Elevation:	528.00		Pile Cutoff Elevation:	528.00	
Pile Tip Elevation:	478.00		Pile Tip Elevation:	478.00	
Battered Piles - Average Batter:	4.0 on 1	103.08%	Battered Piles - Average Batter:	4.0 on 1	103.08%
Battered Piles - Total Number:	12 batt		Battered Piles - Total Number:	22 batt	
Vertical Piles - Total Number:	4 Vert		Vertical Piles - Total Number:	6 Vert	

Forward (West) Abutment

Bottom of Footing Elevation:	522.00		Footing Length:	-	
Top of Stem Elevation:	545.67		Footing Width:	-	
Top of Backwall Shoulder Elevation:	552.88		Footing Toe Width:	5.000 ft	
Median Top of Backwall Elevation:	551.00	Betwn Curbs	Footing Toe Thickness:	5.500 ft	
Pile Tip Elevation:	449.00		Pile Embedment into Footing:	2.000 ft	
Finished Ground Elevation:	530.20		Footing Heel Thickness	5.250 ft	
			Backwall Length (Along Skew):	22.995 ft	Betwn Curbs
Stem Thickness:	5.750 ft		Stem Length:	29.000 ft	Total
Backwall Thickness:	1.750 ft		Backwall Extension Plan Area:	16.26 SqFt	
Pile Row #1 Batter:	4.0 on 1	103.08%	Pile Row #2 Batter:	4.0 on 1	103.08%
Pile Row #1 Piles:	16 ea	1 Vert	Pile Row #2 Piles:	15 ea	1 Vert
Pile Row #3 Batter:	4.0 on 1	103.08%	Pile Row #4 Batter:	Vertical	100.00%
Pile Row #3 Piles:	9 ea	7 Vert	Pile Row #4 Piles:	7 ea	

Forward Wingwall Data (Facing the front face of abutment)

Left Wingwall			Right Wingwall		
Total Wingwall Length:	28.250 ft		Total Wingwall Length:	27.250 ft	
Wingwall Shoulder Length:	7.500 ft		Wingwall Shoulder Length:	6.750 ft	
Wingwall Shoulder Elevation:	552.88		Wingwall Shoulder Elevation:	552.88	
Wingwall Lowest Elevation:	546.79		Wingwall Lowest Elevation:	543.00	
Wingwall Thickness:	1.750 ft	Ave. if Battered	Average Battered Wingwall Thickness:	2.498 ft	
Pile Cutoff Elevation:	0.00		Pile Cutoff Elevation:	529.25	
Pile Tip Elevation:	0.00		Pile Tip Elevation:	479.25	
Battered Piles - Average Batter:	4.0 on 1	103.08%	Battered Piles - Average Batter:	4.0 on 1	103.08%
Battered Piles - Total Number:	0 batt		Battered Piles - Total Number:	12 batt	
Vertical Piles - Total Number:	0 Vert		Vertical Piles - Total Number:	4 Vert	

This calculation will use ODOT Bid Elements. Reinforcing ratios will be used to determine reinforcing until the Stage 3 reinforcing calculations are complete. Framing is normal to the chord for each span, with end diaphragms being placed along the skew angle at bearing locations.

Quantities

202E 11003	**STRUCTURE REMOVED, OVER 20 FOOT SPAN, AS PER PLAN**						**(LS) = 1.0**	0
Super and Sub Removed	**(Assume $120,000)**							
202E 23010	**PAVEMENT REMOVED, ASPHALT**						**(SY) = 250.0**	0
	250.0 SqYd *Area from microstation*							
202E 75000	**FENCE REMOVED**						**(FT) = 60.0**	0
	60.0 ft *Linear feet from microstation*							
503E 11101	**COFFERDAMS AND EXCAVATION BRACING, AS PER PLAN**						**(LS) = 1.0**	0
	(Assume $25,000)							
503E 21101	**UNCLASSIFIED EXCAVATION, AS PER PLAN**						**(CY) = 905.0**	*(CHECK UNIT OF MEASURE)*
Rear Abutment Plan area:	1,857 sq.ft *Area from microstation*	X	8.0 ft *Ave. depth below EX Grade*			/ 27	=	**550 cy**
FWD Abutment North Plan:	1,367 sq.ft *Area from microstation*	X	7.0 ft *Ave. depth below EX Grade*			/ 27	=	**354 cy**
505E 11100	**PILE DRIVING EQUIPMENT MOBILIZATION**						**(LS) = 1.0**	0
Super and Sub Removed	**(Assume $10,000, This is a small part of a much larger mobilization)**							
507E 00501	**12" CAST-IN-PLACE REINFORCED CONCRETE PILES, DRIVEN, AS PER PLAN**						**(FT) = 11,025**	0
Rear Abutment	75.0 ft *Estimated Length*	X	60 piles				=	**4,500 lf**
Rear Abut, Left WW	50.0 ft *Estimated Length*	X	16 piles				=	**800 lf**
Rear Abut, Right WW	50.0 ft *Estimated Length*	X	28 piles				=	**1,400 lf**
Forward Abutment	75.0 ft *Estimated Length*	X	47 piles				=	**3,525 lf**
Forward Abut, Left WW	0.0 ft *Estimated Length*	X	0 piles				=	**0 lf**
Forward Abut, Right WW	50.0 ft *Estimated Length*	X	16 piles				=	**800 lf**
507E 00551	**12" CAST-IN-PLACE REINFORCED CONCRETE PILES, FURNISHED, AS PER PLAN**						**(FT) = 11,860**	0
5' Additional Per Pile	11,025 ft *Driven Length*	+	167 piles *Total Piles*	X	5 ft/pile		=	**11,860 lf**
509E 10000	**EPOXY COATED REINFORCING STEEL**						**(LB) = 97,065**	0
Superstructure (	13,313 lb	)					=	**13,313 lb**
Footing (	36,580 lb *Rear*	+	23,516 lb *Forward*	)			=	**60,096 lb**
Abutment (	8,235 lb *Rear*	+	6,132 lb *Forward*	)			=	**14,367 lb**
Wingwall (	5,829 lb *Rear*		3,460 lb *Forward*	)			=	**9,289 lb**
511E 34447	**CLASS QC2 CONCRETE WITH QC/QA, BRIDGE DECK, AS PER PLAN**						**(CY) = 58**	*(CHECK UNIT OF MEASURE)*
Span 1, Typical Deck	67.56 ft *Length*	X	23.50 ft *Width*	X	0.79 ft *Thickness*	/ (27 cf/cy)) =		**46.6** *CU YD*
Span 1, Between Flanges	67.56 ft *Length*	X	0.5156 sf *XS Area Between*	X	5 bays *Bays*	/ (27 cf/cy)) =		**6.5** *CU YD*
Span 1, Overhangs	67.56 ft *Length*	X	0.5443 sf *XS Outside*	X	2 faces *Faces*	/ (27 cf/cy)) =		**2.7** *CU YD*
Span 1, Cross-Slope, Center	67.56 ft *Length*	X	0.2240 sf *XS of Slope*	X	4 Slope *# Typical Slopes*	/ (27 cf/cy)) =		**2.2** *CU YD*
511E 34451	**CLASS QC2 CONCRETE WITH QC/QA, BRIDGE DECK (PARAPET), AS PER PLAN**						**(CY) = 11**	*(CHECK UNIT OF MEASURE)*
Span 1, Curb	67.56 ft *Length*	X 2 X	1.00 ft *Thickness of Curb*	X	2.00 ft *Height of Curb*	/ (27 cf/cy)) =		**10.0** *CU YD*
						(Assumed equal at both edges)		

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Calculation	Stage 3 Quantities: Norfolk Southern Railroad Bridge
Description	*HAM-075-PROSC, NSRR over Prosser avenue (Single Track Deck Girder)*

GF Job No:	Bridge SFN	Calculated	Checked	Updated	Verified	Level 2 Review
•054682	•XXXXXXX	•SNH, 10/31/14	•CTM, 11/10/14	• SNH, 11/17/14 • VDT, 04/18/16	• CTM, 11/19/14 • CTM, 4/25/16	•

This calculation will use ODOT Bid Elements. Reinforcing ratios will be used to determine reinforcing until the Stage 3 reinforcing calculations are complete. Framing is normal to the chord for each span, with end diaphragms being placed along the skew angle at bearing locations.

511E 45513	**CLASS QC1 CONCRETE WITH QC/QA, ABUTMENT, AS PER PLAN**								**(CY) = 32**		*(MISC. NEW CONSTRUCTION)*
Rear, Backwall Between Curbs	23.00 ft *Length*	X	5.56 ft *Height*		X	1.75 ft *Thickness*		/	(27 cf/cy)) =	**8.3**	*CU YD*
Rear, Backwall Shoulders (	6.6 sf *Left*	+	9.2 sf *Right*	)	X	7.43 ft *Height*		/	(27 cf/cy)) =	**4.3**	*CU YD*
Rear, Bckwall Ext. above Stem	21.99 sf *Plan Area*	X	5.56 ft *Height*				/	(27 cf/cy))	=	**4.5**	*CU YD*
FWD, Backwall Between Curbs	23.00 ft *Length*	X	5.33 ft *Height*		X	1.75 ft *Thickness*		/	(27 cf/cy)) =	**7.9**	*CU YD*
FWD, Backwall Shoulders (	5.4 sf *Left*	+	5.1 sf *Right*	)	X	7.21 ft *Height*		/	(27 cf/cy)) =	**2.8**	*CU YD*
FWD, Bckwall Ext. above Stem	16.26 sf *Plan Area*	X	5.33 ft *Height*				/	(27 cf/cy))	=	**3.2**	*CU YD*
511E 45603	**CLASS QC4 MASS CONCRETE, SUBSTRUCTURE WITH QC/QA, AS PER PLAN**								**(CY) = 283**		*()*
Rear, Stem	32.00 ft *Length*	X	20.63 ft *Height*		X	5.75 ft *Thickness*		/	(27 cf/cy)) =	**140.6**	*CU YD*
Rear, Extension	21.99 sf *Plan Area*	X	20.63 ft *Height*				/	(27 cf/cy))	=	**16.8**	*CU YD*
FWD, Stem	29.00 ft *Length*	X	18.42 ft *Height*		X	5.75 ft *Thickness*		/	(27 cf/cy)) =	**113.8**	*CU YD*
FWD, Extension	16.26 sf *Plan Area*	X	18.42 ft *Height*				/	(27 cf/cy))	=	**11.1**	*CU YD*
511E 46013	**CLASS QC1 CONCRETE WITH QC/QA, AS PER PLAN**								**(CY) = 215**		*(RET WALL/WINGWALL, ABVE FOOTN)*
Rear Wingwall, Left Shoulder	15.27 sf *Square Area*	X	28.06 ft *Height*					/	(27 cf/cy)) =	**15.9**	*CU YD*
Rear Wingwall, Left (Above CJ) (	114.2 sf *Left*	+	58.0 sf *Right*	)	X	1.75 ft *Thickenss*		/	(27 cf/cy)) =	**11.2**	*CU YD*
Rear Wingwall, Left (Below CJ) (	144.0 sf *Left*	+	151.1 sf *Right*	)	X	2.58 ft *Thickenss (Ave)*		/	(27 cf/cy)) =	**28.2**	*CU YD*
Rear Wingwall, Right Shoulder	17.97 sf *Square Area*	X	28.06 ft *Height*					/	(27 cf/cy)) =	**18.7**	*CU YD*
Rear Wingwall, Right (Above CJ)	212.8 sf	X	1.75 ft *Thickness*					/	(27 cf/cy)) =	**13.8**	*CU YD*
Rear Wingwall, Right (Below CJ)	646.1 sf *Left*	X	2.74 ft *Thickness (Ave)*					/	(27 cf/cy)) =	**65.6**	*CU YD*
Forward Wingwall, Left Shoulder	7.50 ft *Length*	X	7.21 ft *Height*		X	1.75 ft *Thickness*		/	(27 cf/cy)) =	**3.5**	*CU YD*
Forward Wingwall, Left Variable	116.3 sf *Square Area*	X	1.75 ft *Thickness*					/	(27 cf/cy)) =	**7.5**	*CU YD*
Forward Wingwall, Right Shoulder	10.54 sf *Square Area*	X	25.63 ft *Height*					/	(27 cf/cy)) =	**10.0**	*CU YD*
Forward Wingwall, Right (below CJ)	323.0 sf *Area*	X	2.50 ft *Thickness (Ave)*					/	(27 cf/cy)) =	**29.9**	*CU YD*
Forward Wingwall, Right (above CJ)	164.90 sf *Area*	X	1.75 ft *Thickness*					/	(27 cf/cy)) =	**10.7**	*CU YD*
511E 46511	**CLASS QC1 CONCRETE, FOOTING, AS PER PLAN**								**(CY) = 165**		*()*
Rear Abut., Wingwall Toe (	94.98 sf *Left Wingwall*	+	154.80 sf *Right Wingwall*	)	X	3.50 ft *Thickness*		/	(27 cf/cy)) =	**32.4**	*CU YD*
Rear Abut., Wingwall Heel (	229.55 sf *Left Wingwall*	+	360.01 sf *Right Wingwall*	)	X	3.25 ft *Thickness*		/	(27 cf/cy)) =	**71.0**	*CU YD*
Forward Abut., Wingwall Toe (	0.00 sf *Left Wingwall*	+	98.80 sf *Right Wingwall*	)	X	3.50 ft *Thickness*		/	(27 cf/cy)) =	**12.8**	*CU YD*
Forward Abut., Wingwall Heel (	0.00 sf *Left Wingwall*	+	237.51 sf *Right Wingwall*	)	X	3.25 ft *Thickness*		/	(27 cf/cy)) =	**28.6**	*CU YD*
Forward Abut., Caisson Cap (	125.22 sf *Left Wingwall*	+	0.00 sf *Right Wingwall*	)	X	4.29 ft *Thickness*		/	(27 cf/cy)) =	**19.9**	*CU YD*

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Calculation	Stage 3 Quantities: Norfolk Southern Railroad Bridge
Description	*HAM-075-PROSC, NSRR over Prosser avenue (Single Track Deck Girder)*

GF Job No:	Bridge SFN	Calculated	Checked	Updated	Verified	Level 2 Review
•054682	•XXXXXXX	•SNH, 10/31/14	•CTM, 11/10/14	• SNH, 11/17/14 • VDT, 04/18/16	• CTM, 11/19/14 • CTM, 4/25/16	•

This calculation will use ODOT Bid Elements. Reinforcing ratios will be used to determine reinforcing until the Stage 3 reinforcing calculations are complete. Framing is normal to the chord for each span, with end diaphragms being placed along the skew angle at bearing locations.

511E 53016	**CLASS QC4 CONCRETE, MISC.: FOOTING MASS CONCRETE WITH QC/QA**							**(CY) =**	**293**	*0*
Rear, Toe	240.95 sf *Plan Area*	**X**	5.50 ft *Height*			**/ (27 cf/cy))**		**=**		**49.1** *CU YD*
Rear, Heel	535.76 sf *Plan Area*	**X**	5.25 ft *Thickness*			**/ (27 cf/cy))**		**=**		**104.2** *CU YD*
FWD, Toe	200.57 sf *Plan Area*	**X**	5.50 ft *Height*			**/ (27 cf/cy))**		**=**		**40.9** *CU YD*
FWD, Heel	508.16 sf *Plan Area*	**X**	5.25 ft *Height*			**/ (27 cf/cy))**		**=**		**98.8** *CU YD*
511E 71200	**CONCRETE, MISC.: FACING OF CANTILEVER WALLS**							**(SF) =**	**193**	*0*
Forward	192.30 sf *Plan Area*							**=**		**192.3** *SQ FT*
512E 10100	**SEALING OF CONCRETE SURFACES (EPOXY-URETHANE)**							**(SY) =**	**561**	*0*
Rear Abutment at Deck	27.8 ft *Height (includes seat)*	**X**	23.0 ft *Length*					**/(9 sf/sy)=**		**71.1 SqYd**
Rear Abutment, Left Shoulder	28.0 ft *Height+ T/Shldr*	**X**	9.2 ft *Length*					**/(9 sf/sy)=**		**28.6 SqYd**
Rear Abutment Stem End	18.3 ft *Height*	**X**	4.0 ft *Length*	**X**	2			**/(9 sf/sy)=**		**16.2 SqYd**
Rear Abutment, Right Shoulder	28.0 ft *Height + T/Shldr*	**X**	11.5 ft *Length*					**/(9 sf/sy)=**		**35.7 SqYd**
Rear Abutment, Left Wing (	68.4 sf *Top + 6" Behind*	**+**	325.1 sf *CADD Area*	**)**				**/(9 sf/sy)=**		**43.7 SqYd**
Rear Abutment, Right Wing (	94.7 sf *Top + 6" Behind*	**+**	503.6 sf *CADD Area*	**)**				**/(9 sf/sy)=**		**66.5 SqYd**
Forward Abutment at Deck	24.8 ft *Height (includes seat)*	**X**	23.0 ft *Length*					**/(9 sf/sy)=**		**63.4 SqYd**
Forward Abutment at Shoulder	24.9 ft *Height+ T/Shldr*	**X**	6.0 ft *Length*					**/(9 sf/sy)=**		**16.6 SqYd**
Forward Abutment Stem End	15.5 ft *Height*	**X**	4.0 ft *Length*	**X**	2			**/(9 sf/sy)=**		**13.8 SqYd**
Forward Abutment, WW Shoulder	24.9 ft *Height+ T/Shldr*	**X**	14.3 ft *Length*					**/(9 sf/sy)=**		**39.5 SqYd**
Forward Abutment, Left Wing (	65.0 sf *Top + 6" Behind*	**+**	330.0 sf *CADD Area*	**)**				**/(9 sf/sy)=**		**43.9 SqYd**
Forward Abutment, Right Wing (	65.2 sf *Top + 6" Behind*	**+**	276.8 sf *CADD Area*	**)**				**/(9 sf/sy)=**		**38.0 SqYd**
Parapet	135.13 ft *Length*	**X**	5.56 ft *Perimeter*					**/(9 sf/sy)=**		**83.5 SqYd**
512E 10001	**SEALING OF CONCRETE SURFACES, AS PER PLAN**							**(SY) =**	**477**	*(PERMANENT GRAFFITI PROTECTION)*
Rear	261.9 SqYd							**=**		**261.9 SqYd**
Forward	215.1 SqYd							**=**		**215.1 SqYd**
512E 56100	**SPECIAL - BUTYL RUBBER MEMBRANE WATERPROOFING**							**(SY) =**	**177**	*0*
Span 1 (	21.5 ft *Deck Width*	**+**	2.0 ft *Up the Curb*	**)**	**X**	67.6 ft *Length*	**/(9 sf/sy)**	**=**		**176 SqYd**
512E 56202	**SPECIAL - ASPHALTIC PANEL**							**(SY) =**	**177**	*0*
Span 1 (	21.5 ft *Deck Width*	**+**	2.0 ft *Up the Curb*	**)**	**X**	67.6 ft *Length*	**/(9 sf/sy)**	**=**		**176 SqYd**

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Calculation	Stage 3 Quantities: Norfolk Southern Railroad Bridge
Description	*HAM-075-PROSC, NSRR over Prosser avenue (Single Track Deck Girder)*

GF Job No:	Bridge SFN	Calculated	Checked	Updated	Verified	Level 2 Review
•054682	•XXXXXXX	•SNH, 10/31/14	•CTM, 11/10/14	• SNH, 11/17/14 • VDT, 04/18/16	• CTM, 11/19/14 • CTM, 4/25/16	•

This calculation will use ODOT Bid Elements. Reinforcing ratios will be used to determine reinforcing until the Stage 3 reinforcing calculations are complete. Framing is normal to the chord for each span, with end diaphragms being placed along the skew angle at bearing locations.

512E 67400	SPECIAL - WATERPROOFING, MISC.: DAMPPROOFING OF RAILROAD STRUCTURES				(SF) = 4,534		()
Rear Abutment	25.9 ft (Height)	X	23.0 ft (Length)			=	596 sq.ft
Rear Abutment Shoulder	27.8 ft (Height)	X	20.7 ft (Length)			=	576 sq.ft
Rear Abutment Left Wingwall	17.5 ft (Ave Height)	X	26.3 ft (Length)			=	460 sq.ft
Rear Abutment Right Wingwall	21.0 ft (Ave Height)	X	41.3 ft (Length)			=	867 sq.ft
Forward Abutment	23.5 ft (Height)	X	23.0 ft (Length)			=	540 sq.ft
Forward Abutment Shoulder	25.4 ft (Height)	X	20.3 ft (Length)			=	514 sq.ft
Forward Abutment Left Wingwall	17.6 ft (Ave Height)	X	28.3 ft (Length)			=	497 sq.ft
Forward Abutment Right Wingwall	17.7 ft (Ave Height)	X	27.3 ft (Length)			=	482 sq.ft

513E 10221	STRUCTURAL STEEL MEMBERS, LEVEL 1, AS PER PLAN					(LB) = 10,585		(CHECK UNIT OF MEASURE)
Diaphragm Flanges, Int.	29.0 in (Typical Length)	X (2 X	3.0 sq.in (Sectional Area)	) /	(1728 ci/cf)	0.1007 cf/ea		
End Diaphragm Flanges	28.0 in (Typical Length)	X (2 X	3.0 sq.in (Sectional Area)	) /	(1728 ci/cf)	0.0972 cf/ea		
Diaphragm Web, No Hole, Int.	1,506 sq.in (Elevation Area)	X	0.5 in (Thickness)	) /	(1728 ci/cf)	0.4357 cf/ea		
Diaphragm Web, With Drain Hole, Int.	1,327 sq.in (Elevation Area)	X	0.5 in (Thickness)	) /	(1728 ci/cf)	0.3841 cf/ea		
End Diaphragm Web	834 sq.in (Elevation Area)	X	0.5 in (Thickness)	) /	(1728 ci/cf)	0.2413 cf/ea		
Weight, Diaph, No Holes	0.536 cf/ea (Volume/Each)	X	6/bay (#/Bay, Excludes End)	X	3.33 bays	X 490 lb/CF	=	5,257.1 Pounds
Weight, Diaph, Drain Holes	0.485 cf/ea (Volume/Each)	X	5/bay (#/Bay, Excludes End)	X	2 bays	X 490 lb/CF	=	2,375.4 Pounds
Weight, End Diaphragm	0.338 cf/ea (Volume/Each)	X	2/bay (#/Bay)	X	5 bays	X 490 lb/CF	=	1,769.1 Pounds
Int Diaphragm Flange Connection Plate	0.009 cf/ea (Volume/Each)	X	10/bay (#/face)	X	6 bays	X 490 lb/CF	=	255.2 Pounds
Jacking Stiffener	0.047 cf/ea (Volume/Each)	X	8/bay (#/face)	X	5 bays	X 490 lb/CF	=	924.1 Pounds
End Diaphragm Bent Connection Plate	0.000 cf/ea (Volume/Each)	X	2/bay (#/Bay)	X	5 bays	X 490 lb/CF X 2	=	0.0 Pounds

513E 10321	STRUCTURAL STEEL MEMBERS, LEVEL 6, AS PER PLAN					(LB) = 166,810		(CHECK UNIT OF MEASURE)
Girder Volume (	37.2 sq.in (Web Area)	+ 2 x	41.3 sq.in (Flange Area)	) /	(144 si/sf)	0.8312 sf/ Ft		
Total Girder Weight	66.9 ft (Length)	X	0.83 sf/Ft (Girder Volume)	X	6 girder(s) (Number Girders)	X 490 lb/CF	=	163,518.9 Pounds
Int. Dprm. Conn. Volume (	208.4 si (Elevation Area)	X	0.500 in (Thickness)	) /	(1728 ci/cf)	0.0603 cf/ea		
Int. Dprm. Conn. Weight	0.060 cf/ea (Volume/Each)	X	6/Face	X	10 faces	X 490 lb/CF	=	1,773.2 Pounds
Bearing Stiff. Volume (	296.7 si (Elevation Area)	X	0.750 in (Thickness)	) /	(1728 ci/cf)	0.1288 cf/ea		
Bearing Stiff. Weight	0.129 cf/ea (Volume/Each)	X	4/Girder	X	6 girder(s)	X 490 lb/CF	=	1,514.5 Pounds

513E 20000	WELDED STUD SHEAR CONNECTORS			(EACH) = 1,656		()
Studs	276 (Studs/Girder)	X	6 gird (No Girder)		=	1,656 Each

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Calculation	Stage 3 Quantities: Norfolk Southern Railroad Bridge
Description	*HAM-075-PROSC, NSRR over Prosser avenue (Single Track Deck Girder)*

GF Job No:	Bridge SFN	Calculated	Checked	Updated	Verified	Level 2 Review
•054682	•XXXXXXX	•SNH, 10/31/14	•CTM, 11/10/14	• SNH, 11/17/14 • VDT, 04/18/16	• CTM, 11/19/14 • CTM, 4/25/16	•

This calculation will use ODOT Bid Elements. Reinforcing ratios will be used to determine reinforcing until the Stage 3 reinforcing calculations are complete. Framing is normal to the chord for each span, with end diaphragms being placed along the skew angle at bearing locations.

514E 80020	**SPECIAL - SHOP PAINTING AND FIELD TOUCH-UP OF STRUCTURAL STEEL**								**(SF) = 5,340**	0
Girder Painting	66.9 ft *Length*	X	6 gird	X	(	133.75 in *Perimeter*	/	(12 in/ft)) =	**4,475.1** *Sq. Ft.*	
End Diaphragms	5 /bay *Ends*	X	2 bays *Bays*	X	(	2436.07 si *Average Surface Area*	/	(144 si/sf)) =	**169.2** *Sq. Ft.*	
Int Diaphragms, No Drain	2 /bay *Ends*	X	1 bays *Bays*	X	(	3806.38 si *Surface Area*	/	(144 si/sf)) =	**52.9** *Sq. Ft.*	
Int Diaphragms, With Drain	3.33 /bay *Ends*	X	5 bays *Bays*	X	(	3449.30 si *Surface Area*	/	(144 si/sf)) =	**399.2** *Sq. Ft.*	
Int. Diaphragm Conn Plate	6/Face	X	10 faces	X	(	159.63 si *Surface Area*	/	(144 si/sf)) =	**66.5** *Sq. Ft.*	
Jacking Stiffener	8 /bay *Ends*	X	5 bays *Bays*	X	(	246.88 si *Surface Area*	/	(144 si/sf)) =	**68.6** *Sq. Ft.*	
Bearing Stiffeners	12 each *No. per Bearing Line*	X	2 brg. lines *No. Bearing Lines*	X	(	625.31 si *Ave. Surface Area, 2 sideS*	/	(144 si/sf)) =	**104.2** *Sq. Ft.*	
End Diaphragm Bent Conn Plate	2/bay *#/Bay*	X	5 bays *Girder Faces (None on Exterior)*	X 2 X	(	0.14 sf *Ave. Surface Area, 2 sideS*		) =	**2.7** *Sq. Ft.*	
516E 10501	**STRUCTURAL EXPANSION JOINT INCLUDING ELASTOMERIC COMPRESSION SEAL, AS PER PLAN**								**(FT) = 46**	0
Joint	23.0 ft *Rear*	+	23.0 ft *Forward*					=	**45.99** *Feet*	
516E 13600	**1" PREFORMED EXPANSION JOINT FILLER**								**(SF) = 259**	0
Rear Abutment	72.3 sf *Left*	+	76.9 sf *Right*					=	**149.26** *Sq. Ft.*	
Forward Abutment	44.9 sf *Left*	+	64.0 sf *Right*					=	**108.88** *Sq. Ft.*	
516E 46201	**BEARING DEVICE, ROCKER, AS PER PLAN**								**(EACH) = 6**	0
	6 each								**6.0** *Each*	
516E 46900	**BEARING DEVICE, MISC.: SELF-LUBRICATING CYLINDRICAL BEARING (EXP)**								**(EACH) = 6**	0
	6 each								**6.0** *Each*	
517E 75001	**RAILING, ALUMINUM, AS PER PLAN**								**(FT) = 161**	0
Rear Backwall & Wingwall	35.0 ft *Left*	+	52.0 ft *Right*					=	**87.0** *Feet*	
Forward Backwall & Wingwall	37.5 ft *Left*	+	36.5 ft *Right*					=	**74.0** *Feet*	
517E 76300	**RAILING, MISC.: NSRR ALUMINUM RAILING WITH VANDAL PROTECTION FENCE**								**(FT) = 136**	*(ADD SUPPLEMENTAL DESCRIPTION)*
NSRR Railing	67.6 ft *span1*	X	2 *Sides*					=	**135.1** *Feet*	
518E 21200	**POROUS BACKFILL WITH FILTER FABRIC**								**(CY) = 309**	*(CHECK UNIT OF MEASURE)*
Rear Abutment, between curbs	23.0 ft *Length*	X	25.7 ft *Height*	X		2.0 ft *Thickness*	/	(27 cf/cy)) =	**43.8** *CU YD*	
Rear Abutment, Shoulder	17.6 ft *Length*	X	27.6 ft *Height*	X		2.0 ft *Thickness*	/	(27 cf/cy)) =	**35.9** *CU YD*	
Rear Abutment, Left WW	26.3 ft *Length*	X	17.5 ft *Ave. Height*	X		2.0 ft *Thickness*	/	(27 cf/cy)) =	**34.1** *CU YD*	
Rear Abutment, Right WW	41.3 ft *Length*	X	21.0 ft *Ave. Height*	X		2.0 ft *Thickness*	/	(27 cf/cy)) =	**64.3** *CU YD*	
Forward Abutment, between curbs	23.0 ft *Length*	X	23.3 ft *Height*	X		2.0 ft *Thickness*	/	(27 cf/cy)) =	**39.6** *CU YD*	
Forward Abutment, Shoulder	9.7 ft *Length*	X	25.1 ft *Height*	X		2.0 ft *Thickness*	/	(27 cf/cy)) =	**18.1** *CU YD*	
Forward Abutment, Left WW	28.3 ft *Length*	X	17.6 ft *Ave. Height*	X		2.0 ft *Thickness*	/	(27 cf/cy)) =	**36.8** *CU YD*	
Forward Abutment, Right WW	27.3 ft *Length*	X	17.7 ft *Ave. Height*	X		2.0 ft *Thickness*	/	(27 cf/cy)) =	**35.7** *CU YD*	
518E 42201	**8" PERFORATED CORRUGATED STEEL PIPE, 707.01, AS PER PLAN**								**(FT) = 212**	0
Length (No perf. 5' from ends)	112.0 ft *Rear Abutment*	+	100.0 ft *Forward Abutment*			=			**212.0** *Feet*	
518E 42301	**8" NON-PERFORATED CORRUGATED STEEL PIPE, INCLUDING SPECIALS, 707.01, AS PER PLAN**								**(FT) = 20**	0

This calculation will use ODOT Bid Elements. Reinforcing ratios will be used to determine reinforcing until the Stage 3 reinforcing calculations are complete. Framing is normal to the chord for each span, with end diaphragms being placed along the skew angle at bearing locations.

Length (End 5')	10.0 ft *Rear Abutment*	+	10.0 ft *Forward Abutment*					**20.0** *Feet*
518E 63300	**STRUCTURE DRAINAGE, MISC.: SUPERSTRUCTURE DRAINAGE SYSTEM**					**(LS) = 1**		*0*
Superstructure Drainage	**(Assume $20,000)**							
523E 20000	**DYNAMIC LOAD TESTING**					**(EACH) = 4**		*0*
See Specifications:	**(Spec requires a minimum of two piles, this number is for two piles per substructure)**							**4.0** *Each*
523E 20500	**RESTRIKE**					**(EACH) = 4**		*0*
See Specifications:	**(Spec requires a minimum of two piles, this number is for two piles per substructure)**							**4.0** *Each*
	Length		*Height*		*Thickness*			
524E 94803	**DRILLED SHAFTS, 42" DIAMETER, ABOVE BEDROCK, AS PER PLAN**					**(FT) = 410**		*0*
Forward Wingwall, Left	9 ea *Number*	X	45.5 ft *Length*				=	**409.5** *Feet*
524E 95100	**DRILLED SHAFTS, MISC.: CSL TESTING**					**(EACH) = 5**		*0*
Forward Wingwall, Left	5 ea *Number*						=	**5** *Each*
530E 00200	**SPECIAL - STRUCTURE, MISC.: SURVEY AND MONITORING OF TRACK AND TEMPORARY SHORING**					**(LS) = 1**		*0*
	(Assume $25,000)							
530E 13000	**SPECIAL - FORM LINER**					**(SF) = 4,828**		*0*
Rear Abutment at Deck	20.6 ft *Height*	X	32.0 ft *Length*				=	**660.2** *SQ FT*
Rear Abutment Stem End	20.6 ft *Height*	X	4.0 ft *Length*	X	2		=	**165.0** *SQ FT*
Rear Abutment, WW Shoulder	28.1 ft *Height*	X	20.7 ft *Length*				=	**580.8** *SQ FT*
Rear Abutment, Left Wing	468.4 sf *CADD Area*						=	**468.4** *SQ FT*
Rear Abutment, Right Wing	858.9 sf *CADD Area*						=	**858.9** *SQ FT*
Forward Abutment at Deck	18.4 ft *Height*	X	29.0 ft *Length*				=	**534.2** *SQ FT*
Forward Abutment Stem End	18.4 ft *Height*	X	4.0 ft *Length*	X	2		=	**147.4** *SQ FT*
Forward Abutment, WW Shoulder	25.6 ft *Height*	X	20.3 ft *Length*				=	**519.1** *SQ FT*
Forward Abutment, Left Wing	405.2 sf *CADD Area*						=	**405.2** *SQ FT*
Forward Abutment, Right Wing	487.9 sf *CADD Area*						=	**487.9** *SQ FT*
611E 01907	**8" CONDUIT, TYPE B, DUCTILE IRON PIPE ANSI CLASS 52, MECHANICAL JOINTS AND FITTINGS, AS PER PLAN**					**(FT) = 130**		*(FOR SANITARY)*
Drainage Tie In	60.0 ft *Rear Abutment Tie*	+	70.0 ft *Forward Abutment Tie*				=	**130.0** *Each*
625E 25604	**CONDUIT, 4", 725.051**					**(FT) = 148**		*0*
	73.6 ft *Span length + 6'-0"*	X	2 *Suides*				=	**147.1** *Each*

DESIGN CALCULATION COVER SHEET

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ISO 9001:2008 CERTIFIED

PROJECT: **ODOT/EMH&T: HAM-75-7.85**
BRIDGE NO: **HAM-75-PROS**
SUBJECT: ***Analysis***
Caisson Wingwall
Abut5 Program
FILE NAME(s): *WW_Caisson*

ODOT PID NO: *77889*
EMH&T JOB NO:
GF JOB NO: 54682
CALC NO.: **HAM075_PROS_15**
REV NO.: **00**
SHEET: 1 OF 8

SUPERSEDES CALC NO: N/A

CALCULATION DESCRIPTION:

This will analyze the proposed caisson wingwalls stem over the cap of the shafts. The caisson wall serves as permanent wing wall post construction. During the construction the same wall will be used for temporary shoring support. This design is only for the stem reinforcement.

DESIGN BASIS OR REFERENCES:

NS Underpass Design Criteria
AREMA (2008) Chapter 8, Section 5
Input Load Calculation (HAM075_PROS_06)

MISCELLANEOUS NOTES:

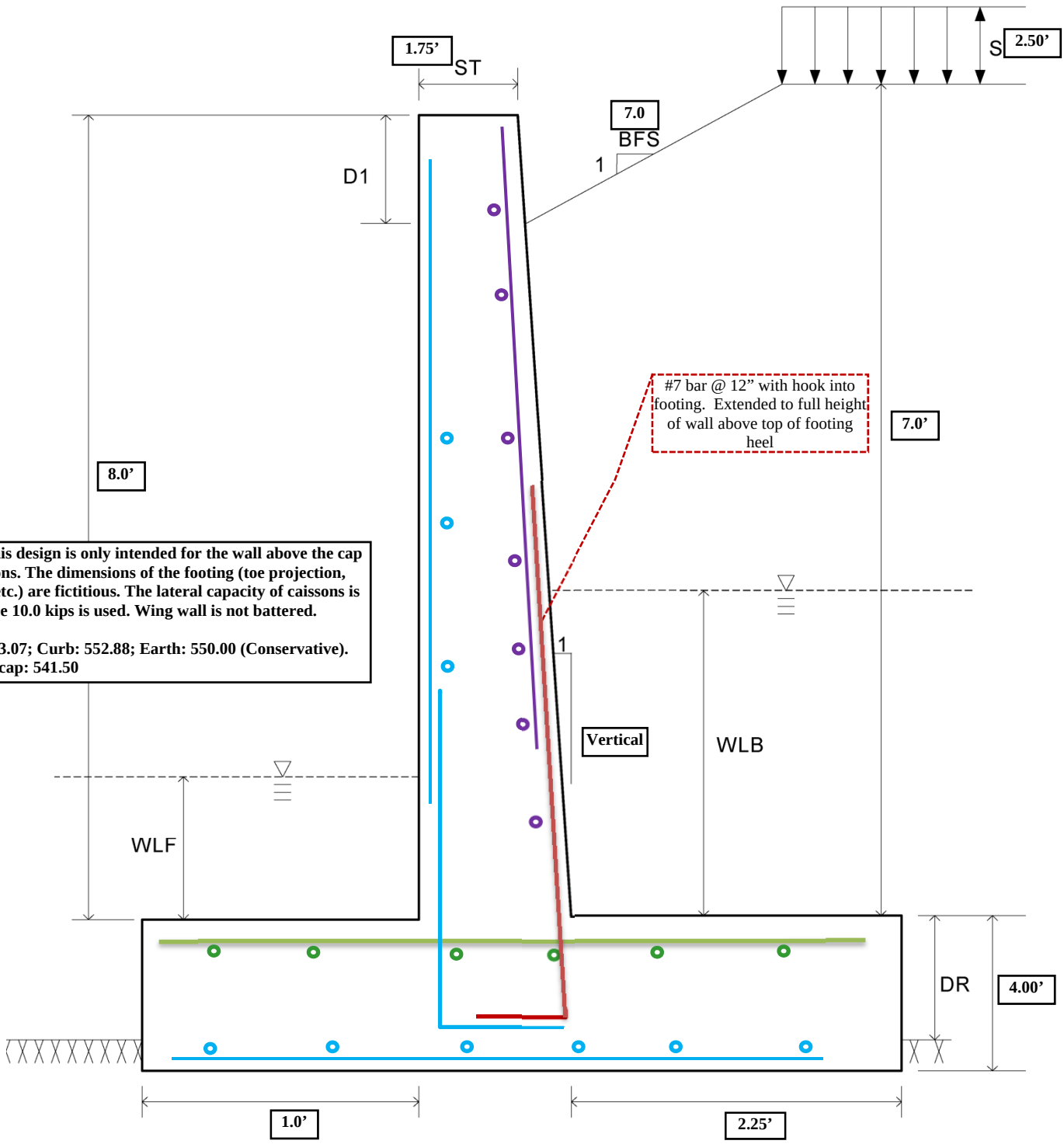
Abut5 Calculates the wingwall self-weight and earth loads automatically.

RECORD OF ISSUES:

REV	DESCRIPTION	BY	DATE	VERIF'D	DATE
00	Stage 1 Calculation	VDK	07/09/14	VDT	11/19/14

RECORD OF CHECK OF CURRENT CALCULATION VERSION

BY	DATE	CHKD	DATE	UPD.	DATE	VERIF'D	DATE
VDK	07/09/14	VDT	10/20/14	SNH	11/19/14	VDT	11/19/14



NOTE: This design is only intended for the wall above the cap over caissons. The dimensions of the footing (toe projection, thickness etc.) are fictitious. The lateral capacity of caissons is high. Hence 10.0 kips is used. Wing wall is not battered.

Profile: 553.07; Curb: 552.88; Earth: 550.00 (Conservative). Bottom of cap: 541.50

NOTE: Heights "H" are increased (approximately 6") in this analysis to ensure conservative design will still apply subject to any potential minor elevation changes during final detailing.

Reinforcing Note: All footing bars will be provided based on engineering judgment to account for the torsional moment in the cap. The bars calculated in this design for the footing are not used.

```
LFD Abutment and Retaining Wall                    333067

PROGRAM P4354040                                          11/19/2014 11:32
VERSION 5.4.0.0           LAST UPDATED 05/05/2005        DOCUMENTATION 05/2005

INPUT: WW_CAI~1.DAT

**PROJECT: ODOT/EMH&T: HAM-75-7.85, GF JOB # 054682
**NSRR OVER PROSSER: CAISSON WINGWALL ADJACENT TO THE BEND AT TOP OF WA
STATION 159+10.79

                              ANALYSIS OF
                            RETAINING WALL

                       TYPE OF FOOTING : ON PILES

                                                                  EQUIV FLUID
DESIGN                    FTG    PILE            PILE              PRESSURE
METHOD   A OR D   TYPE   TYPE     D   EMBEDDED  ROWS   R OR S     DRY   SAT
  SL       A       3      2     42.0     Y       1               40.1

                       ALLOW SOIL   WATER  TOP FTG   WATER    TOP WALL   LIVE
COEFF OF  BACKFILL  PRESS OR AXIAL LEVEL  TO TOP    LEVEL     TO TOP    LOAD
FRICTION    SLOPE     PILE CAPAC    BACK  EMBANK    FRONT    BACKFILL   SURCH
             7.0        190.00             6.50                          2.50

TOP FTG  F'C                                   LATERAL                     PILE
TO ROCK  BACK   F'C    F'C  REBAR   PILE  PILE  PILE      REBAR  OVR  80%   ROW
SURFACE  WALL   STEM   FTG  GRADE  BATTER  OPT  CAPAC  Kv  DES   STR  RULE  OPT
        4000.  4000.  4000.   60.               10.0   5.7  Y     Y

  TOP OF
(BACK)WALL                                          MAX
TO TOP OF     TOP      PROJECTION          MAX    FTG
 FOOTING   THICKNESS   TOE   HEEL  T OR H  PROJ  WIDTH   H1     H2    H3    H4
   8.00      1.75      1.00  2.25

                            FRONT           HEIGHT
                            FACE   BRIDGE     OF
                            TO DL   SEAT   BACKWALL  BACK   FOOTING  PILE  FTG
 W1   W2   W3   BW1   BW2   REACT   WIDTH   BATTER  BATTER THICKNESS COST  COST
                                                                4.00

     TRIAL
 REBAR SPACING  BAR   BW    STEM    FTG    MIN
 1  2  3  4  5  LOC  EXPOS  EXPOS  EXPOS   As
 3  6  8  9 12   Y     S      S      S

     STEM REBAR COVER           FOOTING REBAR COVER
 VERT  HORIZ   VERT  HORIZ    LONG  TRANS   LONG  TRANS
 2.00   3.00   2.00   3.00    3.00   2.00   4.00   3.00

                                          LONG
                      WIND   WIND         FORCE
 DL     LL    WIND    ON     ON   UPWARD  FROM   CENTR  TEMP
REACT  REACT  ON LL  SUPER   SUB   WIND    LL    FORCE  FORCE


                              BACKWALL            ALLOW
    PARAPET OR EXTERNAL       LIVE LOAD  SEISMIC   PILE
 HORZ   DIST   VERT   DIST   VERT   HORZ   LOAD   UPLIFT
                                                  19.00

             DISTANCE           PERCENT
ROW   PILE   BETWEEN    PILE      ROW
NO   BATTER    ROWS    SPACING  BATTERED
 1     0.0      0.25     3.50      0.0
```

42" Concrete Caissons with ODOT allowable 95T per pile (conservative)
Caissons are chosen for their use as temporary shoring and permanent wing wall post construction. Higher capacity and Lower displacement is desirable due to the proximity of underground utilities and profile grade.

```
                    REINFORCEMENT SUMMARY
                    ---------------------
                     REBAR SPACINGS (in):
                              3.
                              6.
                              8.
                              9.
                             12.

          REBAR COVERS (in):   (C.G. OF BAR TO OUTER FACE)
                  STEM                      FOOTING
          BACK          FRONT          TOP         BOTTOM
       VERT  HORIZ   VERT  HORIZ  LONG   TRANS  LONG   TRANS
       2.00   3.00   2.00   3.00  3.00    2.00  4.00    3.00

                     BACK WALL -- SEVERE
       EXPOSURES:    STEM ------- SEVERE
                     FOOTING ---- SEVERE

       MINIMUM AREA OF STEEL PER FOOT: 0.125 SQUARE INCHES


***  NOTE  ***
 - THE REBARS IN THE BOTTOM OF THE FOOTING ARE BELOW THE TOP OF THE PILES.



                         PILE FOOTING ANALYSIS
                         ---------------------

***  NOTE  ***
 - FOR THE TEMPORARY CONSTRUCTION LOADING CONDITION (GROUP T) - FILL HEIGHT
   TO ABUTMENT SEAT, LIVE LOAD SURCHARGE, BUOYANCY AND WIND ON SUBSTRUCTURE
   (IF APPLICABLE):
    > PILE CAPACITY HAS BEEN INCREASED BY A FACTOR OF 1.25.

 - FOR PILE ROWS WITH BOTH VERTICAL AND BATTERED PILES, PILE LOADS ARE
   FOR THE MAXIMUM CONDITION (AXIAL LOAD IN BATTERED PILES).
```

NOTE: Max Pile Reaction under Primary Loads is negligible compared to capacity of 42" caissons.

STABILITY ANALYSIS (SERVICE LOADS) - WITH VERTICAL COMPONENT OF LL SURCHARGE

AASHTO GROUP	SUM V	BAL. MOMENT	SUM H	O.T. MOMENT	FRONT ROW TO RESULT	PILE LOAD FRONT	PILE LOAD BACK	PILE LOAD ALLOW	LAT. RESIST	F.S. O.T.
1	8.29	22.11	3.26	13.26	1.07	0.00	0.00	190.00	5.71	1.67
2	7.62	19.66	2.21	7.74	1.57	0.00	0.00	237.50	7.14	2.54
3	8.29	22.11	3.26	13.26	1.07	0.00	0.00	237.50	7.14	1.67
4	8.29	22.11	3.26	13.26	1.07	0.00	0.00	237.50	7.14	1.67
5	7.62	19.66	2.21	7.74	1.57	0.00	0.00	266.00	8.00	2.54
6	8.29	22.11	3.26	13.26	1.07	0.00	0.00	266.00	8.00	1.67
T	8.25	21.93	3.26	13.26	1.05	0.00	0.00	237.50	7.14	1.65

STABILITY ANALYSIS (SERVICE LOADS) - WITHOUT VERTICAL COMPONENT OF LL SURCHARGE

AASHTO GROUP	SUM V	BAL. MOMENT	SUM H	O.T. MOMENT	FRONT ROW TO RESULT	PILE LOAD FRONT	PILE LOAD BACK	PILE LOAD ALLOW	LAT. RESIST	F.S. O.T.
1	7.62	19.66	3.26	13.26	0.84	0.00	0.00	190.00	5.71	1.48
2	7.62	19.66	2.21	7.74	1.57	0.00	0.00	237.50	7.14	2.54
3	7.62	19.66	3.26	13.26	0.84	0.00	0.00	237.50	7.14	1.48
4	7.62	19.66	3.26	13.26	0.84	0.00	0.00	237.50	7.14	1.48
5	7.62	19.66	2.21	7.74	1.57	0.00	0.00	266.00	8.00	2.54
6	7.62	19.66	3.26	13.26	0.84	0.00	0.00	266.00	8.00	1.48
T	7.57	19.49	3.26	13.26	0.82	0.00	0.00	237.50	7.14	1.47

FOOTING ANALYSIS (SERVICE LOADS) - WITH VERTICAL COMPONENT OF LL SURCHARGE

AASHTO GROUP	TOE MOMENT	TOE SHEAR @	HEEL MOMENT	HEEL SHEAR @
1	-0.30	-0.60 F	4.25	3.78 F
2	-0.30	-0.60 F	3.49	3.10 F
3	-0.30	-0.60 F	4.25	3.78 F
4	-0.30	-0.60 F	4.25	3.78 F
5	-0.30	-0.60 F	3.49	3.10 F
6	-0.30	-0.60 F	4.25	3.78 F
T	-0.30	-0.60 F	4.25	3.78 F

FOOTING ANALYSIS (SERVICE LOADS) - WITHOUT VERTICAL COMPONENT OF LL SURCHARGE

AASHTO GROUP	TOE MOMENT	TOE SHEAR @	HEEL MOMENT	HEEL SHEAR @
1	-0.30	-0.60 F	3.49	3.10 F
2	-0.30	-0.60 F	3.49	3.10 F
3	-0.30	-0.60 F	3.49	3.10 F
4	-0.30	-0.60 F	3.49	3.10 F
5	-0.30	-0.60 F	3.49	3.10 F
6	-0.30	-0.60 F	3.49	3.10 F
T	-0.30	-0.60 F	3.49	3.10 F

FOOTING DESIGN - WITH AND WITHOUT VERTICAL COMPONENT OF LL SURCHARGE

FOOTING WIDTH	FOOTING THICKNESS	TOE PROJ	HEEL PROJ	EFFECTIVE DEPTH TOE	EFFECTIVE DEPTH HEEL
5.00	4.00	1.00	2.25	3.750	3.833

SHEAR STR TOE	ALLOWABLE SHEAR TOE	SHEAR STR HEEL	ALLOWABLE SHEAR HEEL	TRANS REINFORCEMENT TOE-BOT	TRANS REINFORCEMENT HEEL-TOP	LONGITUDINAL REINFORCEMENT
0.000	0.060	0.007	0.060	0.12	0.31	0.12

LONGITUDINAL MOMENT CALCULATIONS - WITH VERTICAL COMPONENT OF LL SURCHARGE
GROUP 1 GOVERNS

MAX UNIFORM LOAD (w) = 1.659 K/FT
MAX UNIFORM SERVICE LOAD (w) = 1.659 K/FT
MAX PILE SPACING = 3.50 FEET
MAX LONGITUDINAL MOMENT = 1.02 K-FT
MAX LONG. MOMENT (SERVICE) = 1.02 K-FT

PILE PATTERN

ROW	BATTER	DISTANCE	SPACING	% BATTERED
1	VERT	0.25	3.50	0.0

C.G. OF PILES 0.25 FT. FROM TOE I OF PILES PER FOOT 0.000
PILE DENSITY 0.2857 PILES PER FOOT

STEM ANALYSIS

DESIGN OF STEM SECTION AT 2.00 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	T
MOMENT	0.25	0.05	0.25	0.25	0.05	0.25	0.90
AXIAL FORCE	0.53	0.53	0.53	0.53	0.53	0.53	0.53
SHEAR	0.28	0.08	0.28	0.28	0.08	0.28	0.60

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	BACK FACE REINF
1.750	0.003	0.060	0.125

DESIGN OF STEM SECTION AT 4.00 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	T
MOMENT	1.23	0.43	1.23	1.23	0.43	1.23	2.63
AXIAL FORCE	1.05	1.05	1.05	1.05	1.05	1.05	1.05
SHEAR	0.72	0.32	0.72	0.72	0.32	0.72	1.16

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	BACK FACE REINF
1.750	0.005	0.060	0.125

DESIGN OF STEM SECTION AT 6.00 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	T
MOMENT	3.25	1.44	3.25	3.25	1.44	3.25	5.64
AXIAL FORCE	1.58	1.58	1.58	1.58	1.58	1.58	1.58
SHEAR	1.32	0.72	1.32	1.32	0.72	1.32	1.88

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	BACK FACE REINF
1.750	0.008	0.060	0.126

DESIGN OF STEM SECTION AT 8.00 FT FROM TOP

AASHTO GROUP (SERVICE)	1	2	3	4	5	6	T
MOMENT	6.63	3.42	6.63	6.63	3.42	6.63	10.25
AXIAL FORCE	2.10	2.10	2.10	2.10	2.10	2.10	2.10
SHEAR	2.09	1.28	2.09	2.09	1.28	2.09	2.76

SECTION THICKNESS	SHEAR STRESS	ALLOWABLE SHEAR	BACK FACE REINF
1.750	0.012	0.060	0.251

SUMMARY OF STEEL DESIGN

FOOTING DESIGN

Top longitudinal reinforcement:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
3	5	1.240
6	5	0.620
8	5	0.465
9	5	0.413
12	5	0.310

Bottom longitudinal reinforcement:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
3	5	1.240
6	5	0.620
8	5	0.465
9	5	0.413
12	5	0.310

Top transverse reinforcement:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
3	5	1.240
6	5	0.620
8	5	0.465
9	5	0.413
12	5	0.310

Bottom transverse reinforcement:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
3	5	1.240
6	5	0.620
8	5	0.465
9	5	0.413
12	5	0.310

STEM DESIGN

Vertical reinforcement on back face:

REBAR SPACING (in)	REBAR SIZE	ACTUAL As (sq in/ft)
At Section #3, 2.00 feet from the top:		
3	4	0.800
6	4	0.400
8	4	0.300
9	4	0.267
12	4	0.200
At Section #4, 4.00 feet from the top:		
3	4	0.800
6	4	0.400
8	4	0.300
9	4	0.267
12	4	0.200
At Section #5, 6.00 feet from the top:		
3	4	0.800
6	4	0.400
8	4	0.300
9	4	0.267
12	4	0.200
At Section #6, 8.00 feet from the top:		
3	5	1.240
6	5	0.620
8	5	0.465
9	5	0.413
12	5	0.310

OPTIMUM REBAR DESIGN

FOOTING:

```
Top longitudinal reinforcement:
     Spacing       Rebar

       12.             5
        9.             5
        6.             5
        8.             5
        3.             5
Bottom longitudinal reinforcement:
     Spacing       Rebar

       12.             5
        9.             5
        6.             5
        8.             5
        3.             5
  Top transverse reinforcement:
     Spacing       Rebar

       12.             5
        9.             5
        6.             5
        8.             5
        3.             5
 Bottom transverse reinforcement:
     Spacing       Rebar

       12.             5
        9.             5
        6.             5
        8.             5
        3.             5
```

Proposed Footing Reinforcing

Top Mat:
Not designed using this sheet.
Bottom Mat:
Not designed using this sheet.

STEM:

```
  Here is the design of the vertical
reinforcement
  in the back face of the stem.   For this
design,
  the  spacing and rebar size are the same for
the
  entire height of the wall.

  Vertical reinforcement on back of stem:
     Spacing       Rebar

       12.             5
        9.             5
        8.             5
        6.             5
        3.             5
```

STEEL DESIGNS LISTED FROM MOST DESIRABLE TO LEAST OPTIMAL

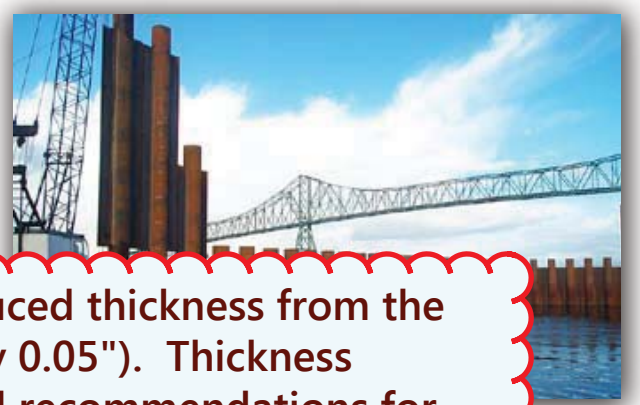
Proposed Primary Stem Reinforcing

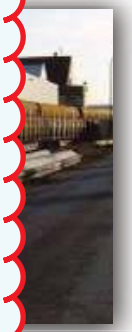
Rear Mat Vertical Bars:
#7's @ 12" hooked into foundation and extending full height of wall above top of footing.

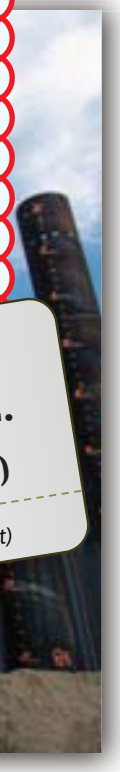
Similar to H-Piles, Pipe Piles are also designed to transfer structural loads through the foundation to soils below. Where H-Piles are typically classified as poi
effi
sub
wit
gre

Pip
wit
and
exc
alo

L.B
DS
We
size
req
pro
coa
complete pipe pile solutions.







Calculations will assume 0.05" of reduced thickness from the outside of the pile (O.D. is reduced by 0.05"). Thickness reduction is from SkylineSteel manual recommendations for 100 years.

.

3/8" and 7/16" minimum pile wall thicknesses are being specified.

.

12.6ksi allowable stress per AREMA 8-4.4.4.6(b)

.

Capacity = 12.6 * Pi/4 * { (OD-2*0.05)^2 - (OD - 2*Twall)^2 }

.

12" Capacity (0.375" wall): 148 kips
14" Capacity (0.437" wall): 207 kips
16" Capacity (0.437" wall): 237 kips

Gannett Fleming Login
edues
Calculated
By: Eric Dues, P.E., S.E.
Date: 07/17/2014 (Thu, am)
(Note Result) (Note Input)

Gannett Fleming Login
vtraini
Check
By: Vincent Traini
Date: 07/17/2014 (Thu, am)
(Agree) (Disagree) (Discuss)

LBFoster Piling

Electric Resistance Weld

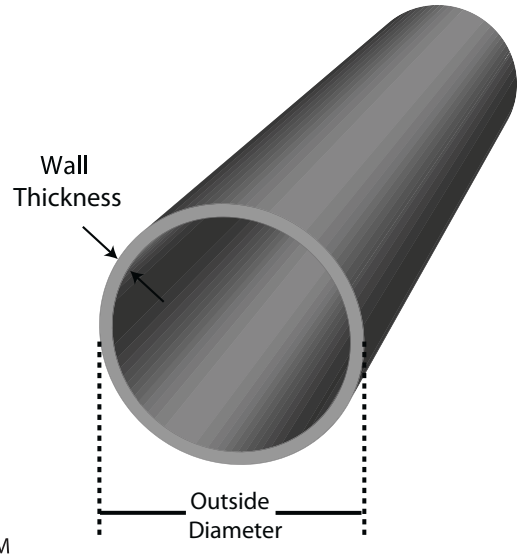
3/8" wall

7/16" wall

ERW	Wall Thickness (in.)							
Outside Diameter (in.) / lbs / ft	0.219	0.250	0.312	0.375	0.438	0.500	0.625	0.750
8.625	19.68	22.38	27.73	33.07	38.33	43.43		
10.75	24.65	28.06	34.81	41.59	48.28	54.79		
12	27.58	31.40	38.98	46.60	54.14	61.47	76.00	
12.75	29.34	33.41	41.48	49.61	57.65	65.48	81.01	
14	32.26	36.75	45.65	54.62	63.50	72.16	89.36	
16	36.95	42.09	52.32	62.64	72.86	82.85	102.72	
18	41.63	47.44	58.99	70.65	82.23	93.54	116.09	
20		52.78	65.66	78.67	91.59	104.23	129.45	
24		63.47	79.01	94.71	110.32	125.61	156.17	186.41

ERW is able to be produced to pipe specifications: A252, A500, A53 and API 5L

Pipe can be produced to the above specifications utilizing other grades of steel (A36, A572, A588 etc.)



Pipe Weight Calculation:

LB / FT = (OD-WT) WT × 10.69

OD = Outside Diameter
WT = Wall Thickness

TRU12™
TRUE 12" PIPE PILING

(800) 848-6249
www.lbfoster.com

skylinesteel
a NUCOR company

Pipe

we are driving through undisturbed natural soils

Loss of Thickness Due to Corrosion for Piles in Soil with or without Groundwater

Required design working life	5 Years	25 Years	50 Years	75 Years	100 Years
	in / mm				
Undisturbed natural soils (sand, clay, schist, ...)	0.000 0.00	0.012 0.30	0.024 0.60	0.035 0.90	0.047 1.20
Polluted natural soils and industrial grounds	0.006 0.15	0.030 0.75	0.059 1.50	0.089 2.25	0.118 3.00
Aggressive natural soils (swamp, marsh, peat, ...)	0.008 0.20	0.039 1.00	0.069 1.75	0.098 2.50	0.128 3.25
Non-compacted and non-aggressive fills (clay, schist, sand, silt, ...)	0.007 0.18	0.028 0.70	0.047 1.20	0.067 1.70	0.087 2.20
Non-compacted and aggressive fills (ashes, slag, ...)	0.020 0.50	0.079 2.00	0.128 3.25	0.177 4.50	0.226 5.75

Notes:
1. Corrosion rates in compacted fills are lower than those in non-compacted ones. In compacted fills, the figures in the table should be divided by two.
2. The values given are only for guidance. Local conditions should be considered because they may affect the actual corrosion rate, which can be lower or higher than the average value given in the table.
3. The values given for 5 and 25 years are based on measurements, whereas the other values are extrapolated.

Loss of Thickness Due to Corrosion for Piles in Fresh Water or in Sea Water

Required design working life	5 Years	25 Years	50 Years	75 Years	100 Years
	in / mm				
Common fresh water (river, ship canal, ...) in the zone of high attack (water line)	0.006 0.15	0.022 0.55	0.035 0.90	0.045 1.15	0.055 1.40
Very polluted fresh water (sewage, industrial effluent, ...) in the zone of high attack (water line)	0.012 0.30	0.051 1.30	0.091 2.30	0.130 3.30	0.169 4.30
Sea water in temperate climate in the zone of high attack (low water and splash zones)	0.022 0.55	0.074 1.90	0.148 3.75	0.220 5.60	0.295 7.50
Sea water in temperate climate in the zone of permanent immersion or in the intertidal zone	0.010 0.25	0.035 0.90	0.069 1.75	0.102 2.60	0.138 3.50

Notes:
1. The highest corrosion rate is usually found at the splash zone or at the low water level in tidal waters. However, in most cases, the highest stresses are in the permanent immersion zone.
2. The values given are only for guidance. Local conditions should be considered because they may affect the actual corrosion rate, which can be lower or higher than the average value given in the table.
3. The values given for 5 and 25 years are based on measurements, whereas the other values are extrapolated.