Structural Design Calculations

ABLE CANAL PATHWAY

FPID NO: 435351-2

PREPARED FOR:

Lee County
Southwest Florida

LUCIO MARTINEZ
LICENSE
No. 82380
STATE OF
FLORIDA
PROFESSIONAL ENGINEER

This item has been digitally signed and sealed by:

Lucio Martinez
2024.11.06
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on the date adjacent to the seal. Printed copies of this document are not considered signed and sealed and the signature must be verified on any electronic copies.

PREPARED BY:

KISINGER CAMPO & ASSOCIATES CORP.
13461 PARKER COMMONS BOULEVARD, SUITE 104
FORT MYERS, FL 33912

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1. Geometry

1.1. Horizontal Alignment

LM 03/2022

```
Degree       =      57° 17' 44.81"
Tangent      =            12.5863
Length       =            25.0409
Radius       =           100.0000
External     =             0.7890
Long Chord   =            24.9755
Mid. Ord.    =             0.7828
P.C.  Station          446+88.56  N        837,719.1245  E        760,468.2754
P.T.  Station          447+13.60  N        837,701.5852  E        760,486.0559
C.C.                              N        837,639.7203  E        760,407.4892
Back         = S  52° 33' 53.75" E
Ahead        = S  38° 13' 03.26" E
Chord Bear   = S  45° 23' 28.50" E

Course from PT BL_ABLE_31 to 387 S 38° 13' 03.26" E Dist 436.0296

Point 387             N     837,359.0110 E     760,755.8053 Sta     451+49.63
```

Bridge limits within these bounds

```
Course from 387 to 388 S 36° 58' 09.17" E Dist 570.0065

Point 388             N     836,903.5993 E     761,098.5992 Sta     457+19.63

Course from 388 to PC BL_ABLE_38 S 36° 26' 04.02" E Dist 73.0679

                                  Curve Data
                                 *----------*
Curve BL_ABLE_38
P.I.  Station          458+17.04  N        836,825.2354  E        761,156.4468
Delta        =       5° 34' 21.59" (LT)
Degree       =      11° 27' 32.96"
Tangent      =            24.3345
Length       =            48.6307
Radius       =           500.0000
External     =             0.5918
Long Chord   =            48.6115
Mid. Ord.    =             0.5911
P.C.  Station          457+92.70  N        836,844.8135  E        761,141.9944
P.T.  Station          458+41.33  N        836,807.1534  E        761,172.7320
C.C.                              N        837,141.7648  E        761,544.2629
Back         = S  36° 26' 04.02" E
Ahead        = S  42° 00' 25.61" E
Chord Bear   = S  39° 13' 14.82" E

                                  Curve Data
                                 *----------*
Curve BL_ABLE_39
P.I.  Station          458+90.26  N        836,770.7969  E        761,205.4757
Delta        =       5° 36' 08.17" (RT)
Degree       =       5° 43' 46.48"
Tangent      =            48.9280
Length       =            97.7780
Radius       =         1,000.0000
External     =             1.1963
Long Chord   =            97.7391
Mid. Ord.    =             1.1948
P.C.  Station          458+41.33  N        836,807.1534  E        761,172.7320
P.T.  Station          459+39.11  N        836,731.4175  E        761,234.5139
C.C.                              N        836,137.9305  E        760,429.6703
Back         = S  42° 00' 25.61" E
Ahead        = S  36° 24' 17.44" E
Chord Bear   = S  39° 12' 21.53" E

Course from PT BL_ABLE_39 to PC BL_ABLE_42 S 36° 24' 17.44" E Dist 1,816.9848
```

1.2. Vertical Profile

VPC 445+66.00 19.1317 -0.1996 K = 17.4
Low Point 445+69.47 19.1283
VPI 31 445+96.00 19.0719 60.0000 30.0000 30.0000
VPT 446+26.00 20.0469 3.2500

VPC 446+75.00 21.6394 3.2500 K = 19.7 SSD = 383.8
VPI 32 447+05.00 22.6144 60.0000 30.0000 30.0000
VPT 447+35.00 22.6744 0.2000
Begin Bridge Sta.: 447+38.67
VPI 33 448+44.75 22.8939 0.2000
End Bridge Sta.: 450+03.83
VPC 450+10.00 22.7298 -0.0993 K = 29.3 SSD = 556.2
VPI 34 450+40.00 22.7000 60.0000 30.0000 30.0000
VPT 450+70.00 22.0550 -2.1500

VPC 451+35.00 20.6575 -2.1500 K = 27.9
VPI 35 451+65.00 20.0125 60.0000 30.0000 30.0000
VPT 451+95.00 20.0125 0.0000

VPC 453+10.00 20.0125 0.0000 K = 43.5 SSD = 812.0
VPI 36 453+40.00 20.0125 60.0000 30.0000 30.0000
VPT 453+70.00 19.5986 -1.3798

VPC 454+87.00 17.9842 -1.3798 K = 43.5
VPI 37 455+17.00 17.5702 60.0000 30.0000 30.0000
VPT 455+47.00 17.5702 0.0000

VPI 38 456+53.00 17.5702 0.0000

VPI 39 457+90.73 18.1900 0.4500

VPC 458+70.28 19.4503 1.5841 K = 17.8 SSD = 352.6
High Point 458+98.45 19.6734
VPI 40 459+00.00 19.9210 59.4344 29.7172 29.7172
VPT 459+29.72 19.3986 -1.7580

VPI 41 460+00.00 18.1630 -1.7580

VPI 42 461+00.00 17.9383 -0.2247

VPC 462+70.00 19.1623 0.7200 K = 31.2 SSD = 592.0
High Point 462+92.50 19.2433
VPI 43 463+00.00 19.3783 60.0000 30.0000 30.0000
VPT 463+30.00 19.0183 -1.2000

VPC 463+70.00 18.5383 -1.2000 K = 50.0
VPI 44 464+00.00 18.1783 60.0000 30.0000 30.0000
VPT 464+30.00 18.1783 0.0000

1.3. Vertical Clearance

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LEE COUNTY
ABLE CANAL PATHWAY

DESIGNED BY: JHJ 08/22
CHECKED BY: LM 08/2022

Miminum Vertical Clearance

ABLE CANAL OVER CHARLIE DIVERSION CANAL

Minimum Vertical Clearance

Location of LME	Begin Bridge LME*	End Bridge LME**	Station at LME (Begin Bridge)	LME
Begin Bridge, Beam 1	22.666	22.720	447+38.67	20.824

	Elevation *(ft.)*	M.V.C. *(ft.)*	Plan Dimension
100 Yr.	19.770	1.054	1'-0 5/8"
NW	14.680	6.144	6'-1 3/4"
Begin Bridge	19.239	1.585	1'-7"

* 22.6817 - 9.5" x 0.02

** 22.7359 - 9.5" x 0.02

NOTE: The proposed minimum vertical clearances meet the LA-MSID requirements of 1ft above 100-year elevation and 6ft above the NW elevation.

1.4. Finish Grade Elevations

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CHECKED BY: LM 03/2022

Finished Grade Elevations

ABLE CANAL PATHWAY OVER CHARLIE DIVERSION CANAL

Vertical Alignment Along PBL_ABLE				
	Station	Elevation	Grade 1 =	3.250%
PVC	446+75.00	21.639	Grade 2 =	0.200%
PVI	447+05.00	22.614	G2-G1 =	-3.050%
PVT	447+35.00	22.674	L =	60.00

Vertical Alignment Along PBL_ABLE				
	Station	Elevation	Grade 1 =	0.200%
PVT	447+35.00	22.674		
PI	448+44.75	22.894	Grade 2 =	-0.099%
PVC	450+10.00	22.730		

Vertical Alignment Along PBL_ABLE				
	Station	Elevation	Grade 1 =	-0.099%
PVC	450+10.00	22.730	Grade 2 =	-3.250%
PVI	450+40.00	22.700	G2-G1 =	-3.151%
PVT	450+70.00	21.725	L =	60.00

Information Along T-Lines				
	Begin A.S. 1	T-1	T-2	Begin Bridge
Station	447+28.67	447+32.00	447+35.33	447+38.67
FGE	22.652	22.666	22.675	22.682

Horizontal Alignment - BL_ABLE_31	
X-Slope	2.00%
BL Radius	100.00
Begin Curve	446+88.56
End Curve	447+13.60

Approach Slab 1						
Location	Offset	X-Slope	Begin A.S. 1	T-1	T-2	Begin Bridge
Left Coping	-0.792	0.02	22.636	22.650	22.659	22.666
BL Const. Able Canal/ Left Gutter Line	0.000	0.02	22.652	22.666	22.675	22.682
Right Gutter Line	12.000	0.02	22.892	22.906	22.915	22.922
Right Coping	12.792	0.02	22.907	22.922	22.931	22.938

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Vertical Alignment Along PBL_ABLE				
	Station	Elevation	Grade 1 =	3.250%
PVC	446+75.00	21.639	Grade 2 =	0.200%
PVI	447+05.00	22.614	G2-G1 =	-3.050%
PVT	447+35.00	22.674	L =	60.00

Vertical Alignment Along PBL_ABLE				
	Station	Elevation	Grade 1 =	0.200%
PVT	447+35.00	22.674		
PI	448+44.75	22.894	Grade 2 =	-0.099%
PVC	450+10.00	22.730		

Vertical Alignment Along PBL_ABLE				
	Station	Elevation	Grade 1 =	-0.099%
PVC	450+10.00	22.730	Grade 2 =	-3.250%
PVI	450+40.00	22.700	G2-G1 =	-3.151%
PVT	450+70.00	21.725	L =	60.00

Information Along T-Lines				
	End Bridge	T-1	T-2	End A.S. 2
Station	450+03.83	450+07.17	450+10.50	450+13.83
FGE	22.736	22.733	22.729	22.722

Approach Slab 2						
Location	Offset	X-Slope	End Bridge	T-1	T-2	End A.S. 2
Left Coping	-0.792	0.02	22.720	22.717	22.713	22.706
BL Const. Able Canal/ Left Gutter Line	0.000	0.02	22.736	22.733	22.729	22.722
Right Gutter Line	12.000	0.02	22.976	22.973	22.969	22.962
Right Coping	12.792	0.02	22.992	22.988	22.985	22.978

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Vertical Alignment Along PBL_ABLE				
	Station	Elevation	Grade 1 =	3.250%
PVC	446+75.00	21.639	Grade 2 =	0.200%
PVI	447+05.00	22.614	G2-G1 =	-3.050%
PVT	447+35.00	22.674	L =	60.00

Vertical Alignment Along PBL_ABLE				
	Station	Elevation	Grade 1 =	0.200%
PVT	447+35.00	22.674		
PI	448+44.75	22.894	Grade 2 =	-0.099%
PVC	450+10.00	22.730		

Vertical Alignment Along PBL_ABLE				
	Station	Elevation	Grade 1 =	-0.099%
PVC	450+10.00	22.730	Grade 2 =	-3.250%
PVI	450+40.00	22.700	G2-G1 =	-3.151%
PVT	450+70.00	21.725	L =	60.00

Information Along T-Lines								
	Begin Bridge	CL Exp. Joint	T-1	T-2	T-3	T-4	T-5	CL Int. Bent 2
Station	447+38.67	447+38.75	447+47.58	447+56.42	447+65.25	447+74.08	447+82.92	447+91.75
FGE	22.682	22.682	22.700	22.717	22.735	22.753	22.770	22.788

Span 1										
Location	Offset	X-Slope	Begin Bridge	CL Exp. Joint	T-1	T-2	T-3	T-4	T-5	CL Int. Bent 2
Left Coping	-0.792	0.02	22.666	22.666	22.684	22.701	22.719	22.737	22.754	22.772
BL Const. Able Canal/ Left Gutter Line	0.000	0.02	22.682	22.682	22.700	22.717	22.735	22.753	22.770	22.788
CL Beam 1	1.417	0.02	22.710	22.710	22.728	22.746	22.763	22.781	22.799	22.816
CL Beam 2	6.000	0.02	22.802	22.802	22.820	22.837	22.855	22.873	22.890	22.908
CL Beam 3	10.583	0.02	22.893	22.894	22.911	22.929	22.947	22.964	22.982	23.000
Right Gutter Line	12.000	0.02	22.922	22.922	22.940	22.957	22.975	22.993	23.010	23.028
Right Coping	12.792	0.02	22.938	22.938	22.955	22.973	22.991	23.008	23.026	23.044

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Finished Grade Elevations

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Vertical Alignment Along PBL_ABLE				
	Station	Elevation	Grade 1 =	3.250%
PVC	446+75.00	21.639	Grade 2 =	0.200%
PVI	447+05.00	22.614	G2-G1 =	-3.050%
PVT	447+35.00	22.674	L =	60.00

Vertical Alignment Along PBL_ABLE				
	Station	Elevation	Grade 1 =	0.200%
PVT	447+35.00	22.674		
PI	448+44.75	22.894	Grade 2 =	-0.099%
PVC	450+10.00	22.730		

Vertical Alignment Along PBL_ABLE				
	Station	Elevation	Grade 1 =	-0.099%
PVC	450+10.00	22.730	Grade 2 =	-3.250%
PVI	450+40.00	22.700	G2-G1 =	-3.151%
PVT	450+70.00	21.725	L =	60.00

Information Along T-Lines							
	CL Int. Bent 2	T-1	T-2	T-3	T-4	T-5	CL Int. Bent 3
Station	447+91.75	448+00.58	448+09.42	448+18.25	448+27.08	448+35.92	448+44.75
FGE	22.788	22.806	22.823	22.841	22.859	22.876	22.894

Span 2									
Location	Offset	X-Slope	CL Int. Bent 2	T-1	T-2	T-3	T-4	T-5	CL Int. Bent 3
Left Coping	-0.792	0.02	22.772	22.790	22.807	22.825	22.843	22.860	22.878
BL Const. Able Canal/ Left Gutter Line	0.000	0.02	22.788	22.806	22.823	22.841	22.859	22.876	22.894
CL Beam 1	1.417	0.02	22.816	22.834	22.852	22.869	22.887	22.905	22.922
CL Beam 2	6.000	0.02	22.908	22.926	22.943	22.961	22.979	22.996	23.014
CL Beam 3	10.583	0.02	23.000	23.017	23.035	23.053	23.070	23.088	23.106
Right Gutter Line	12.000	0.02	23.028	23.046	23.063	23.081	23.099	23.116	23.134
Right Coping	12.792	0.02	23.044	23.061	23.079	23.097	23.114	23.132	23.150

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Finished Grade Elevations

ABLE CANAL PATHWAY OVER CHARLIE DIVERSION CANAL

Vertical Alignment Along PBL_ABLE				
	Station	Elevation	Grade 1 =	3.250%
PVC	446+75.00	21.639	Grade 2 =	0.200%
PVI	447+05.00	22.614	G2-G1 =	-3.050%
PVT	447+35.00	22.674	L =	60.00

Vertical Alignment Along PBL_ABLE				
	Station	Elevation	Grade 1 =	0.200%
PVT	447+35.00	22.674		
PI	448+44.75	22.894	Grade 2 =	-0.099%
PVC	450+10.00	22.730		

Vertical Alignment Along PBL_ABLE				
	Station	Elevation	Grade 1 =	-0.099%
PVC	450+10.00	22.730	Grade 2 =	-3.250%
PVI	450+40.00	22.700	G2-G1 =	-3.151%
PVT	450+70.00	21.725	L =	60.00

Information Along T-Lines							
	CL Int. Bent 3	T-1	T-2	T-3	T-4	T-5	CL Int. Bent 4
Station	448+44.75	448+53.58	448+62.42	448+71.25	448+80.08	448+88.92	448+97.75
FGE	22.894	22.885	22.876	22.868	22.859	22.850	22.841

Span 3									
Location	Offset	X-Slope	CL Int. Bent 3	T-1	T-2	T-3	T-4	T-5	CL Int. Bent 4
Left Coping	-0.792	0.02	22.878	22.869	22.861	22.852	22.843	22.834	22.825
BL Const. Able Canal/ Left Gutter Line	0.000	0.02	22.894	22.885	22.876	22.868	22.859	22.850	22.841
CL Beam 1	1.417	0.02	22.922	22.913	22.905	22.896	22.887	22.878	22.870
CL Beam 2	6.000	0.02	23.014	23.005	22.996	22.988	22.979	22.970	22.961
CL Beam 3	10.583	0.02	23.106	23.097	23.088	23.079	23.070	23.062	23.053
Right Gutter Line	12.000	0.02	23.134	23.125	23.116	23.108	23.099	23.090	23.081
Right Coping	12.792	0.02	23.150	23.141	23.132	23.123	23.115	23.106	23.097

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Finished Grade Elevations

ABLE CANAL PATHWAY OVER CHARLIE DIVERSION CANAL

Vertical Alignment Along PBL_ABLE				
	Station	Elevation	Grade 1 =	3.250%
PVC	446+75.00	21.639	Grade 2 =	0.200%
PVI	447+05.00	22.614	G2-G1 =	-3.050%
PVT	447+35.00	22.674	L =	60.00

Vertical Alignment Along PBL_ABLE				
	Station	Elevation	Grade 1 =	0.200%
PVT	447+35.00	22.674		
PI	448+44.75	22.894	Grade 2 =	-0.099%
PVC	450+10.00	22.730		

Vertical Alignment Along PBL_ABLE				
	Station	Elevation	Grade 1 =	-0.099%
PVC	450+10.00	22.730	Grade 2 =	-3.250%
PVI	450+40.00	22.700	G2-G1 =	-3.151%
PVT	450+70.00	21.725	L =	60.00

Information Along T-Lines							
	CL Int. Bent 4	T-1	T-2	T-3	T-4	T-5	CL Int. Bent 5
Station	448+97.75	449+06.58	449+15.42	449+24.25	449+33.08	449+41.92	449+50.75
FGE	22.841	22.832	22.824	22.815	22.806	22.797	22.789

Span 4									
Location	Offset	X-Slope	CL Int. Bent 4	T-1	T-2	T-3	T-4	T-5	CL Int. Bent 5
Left Coping	-0.792	0.02	22.825	22.817	22.808	22.799	22.790	22.782	22.773
BL Const. Able Canal/ Left Gutter Line	0.000	0.02	22.841	22.832	22.824	22.815	22.806	22.797	22.789
CL Beam 1	1.417	0.02	22.870	22.861	22.852	22.843	22.835	22.826	22.817
CL Beam 2	6.000	0.02	22.961	22.952	22.944	22.935	22.926	22.917	22.909
CL Beam 3	10.583	0.02	23.053	23.044	23.035	23.027	23.018	23.009	23.000
Right Gutter Line	12.000	0.02	23.081	23.072	23.064	23.055	23.046	23.037	23.029
Right Coping	12.792	0.02	23.097	23.088	23.080	23.071	23.062	23.053	23.044

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Finished Grade Elevations

ABLE CANAL PATHWAY OVER CHARLIE DIVERSION CANAL

Vertical Alignment Along PBL_ABLE				
	Station	Elevation	Grade 1 =	3.250%
PVC	446+75.00	21.639	Grade 2 =	0.200%
PVI	447+05.00	22.614	G2-G1 =	-3.050%
PVT	447+35.00	22.674	L =	60.00

Vertical Alignment Along PBL_ABLE				
	Station	Elevation	Grade 1 =	0.200%
PVT	447+35.00	22.674		
PI	448+44.75	22.894	Grade 2 =	-0.099%
PVC	450+10.00	22.730		

Vertical Alignment Along PBL_ABLE				
	Station	Elevation	Grade 1 =	-0.099%
PVC	450+10.00	22.730	Grade 2 =	-3.250%
PVI	450+40.00	22.700	G2-G1 =	-3.151%
PVT	450+70.00	21.725	L =	60.00

Information Along T-Lines								
	CL Int. Bent 5	T-1	T-2	T-3	T-4	T-5	CL Exp. Joint	End Bridge
Station	449+50.75	449+59.58	449+68.42	449+77.25	449+86.08	449+94.92	450+03.75	450+03.83
FGE	22.789	22.780	22.771	22.762	22.754	22.745	22.736	22.736

Span 5										
Location	Offset	X-Slope	CL Int. Bent 5	T-1	T-2	T-3	T-4	T-5	CL Exp. Joint	End Bridge
Left Coping	-0.792	0.02	22.773	22.764	22.755	22.746	22.738	22.729	22.720	22.720
BL Const. Able Canal/ Left Gutter Line	0.000	0.02	22.789	22.780	22.771	22.762	22.754	22.745	22.736	22.736
CL Beam 1	1.417	0.02	22.817	22.808	22.799	22.791	22.782	22.773	22.764	22.764
CL Beam 2	6.000	0.02	22.909	22.900	22.891	22.882	22.874	22.865	22.856	22.856
CL Beam 3	10.583	0.02	23.000	22.992	22.983	22.974	22.965	22.956	22.948	22.948
Right Gutter Line	12.000	0.02	23.029	23.020	23.011	23.002	22.994	22.985	22.976	22.976
Right Coping	12.792	0.02	23.044	23.036	23.027	23.018	23.009	23.001	22.992	22.992

1.5. Substructure Elevations

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LEE COUNTY
ABLE CANAL PATHWAY

DESIGNED BY: JHJ 03/22
CHECKED BY: LM 03/2022

Substructure Elevations - FGE at Centerlines of Bearing and Mid-Span

ABLE CANAL PATHWAY OVER CHARLIE DIVERSION CANAL

Vertical Alignment

Location	Station	Elevation		
PVT	447+35.00	22.674	Grade 1 =	0.200%
PI	448+44.75	22.894	Grade 2 =	-0.099%
PVC	450+10.00	22.730		

Legend

Data Manual Input:
Data Reference:

Finish Grade Elevations

Location		Station	PGL EL. (ft.)	Offset (ft.)	Cross Slope (%)	FGE (ft.)	Span Brg-Brg (ft.)	Beam Grade (%)
Span 1	E.B. 1 Brg.	447+39.88	22.684	0.000	0.000%	22.684	50.75	0.20%
	Mid-Span	447+65.25	22.735	0.000	0.000%	22.735		
	Pier 2 Brg.	447+90.63	22.786	0.000	0.000%	22.786		
Span 2	Pier 2 Brg.	447+92.88	22.790	0.000	0.000%	22.790	50.75	0.20%
	Mid-Span	448+18.25	22.841	0.000	0.000%	22.841		
	Pier 3 Brg.	448+43.63	22.892	0.000	0.000%	22.892		
Span 3	Pier 3 Brg.	448+45.88	22.893	0.000	0.000%	22.893	50.75	-0.10%
	Mid-Span	448+71.25	22.868	0.000	0.000%	22.868		
	Pier 4 Brg.	448+96.63	22.842	0.000	0.000%	22.842		
Span 4	Pier 4 Brg.	448+98.88	22.840	0.000	0.000%	22.840	50.75	-0.10%
	Mid-Span	449+24.25	22.815	0.000	0.000%	22.815		
	Pier 5 Brg.	449+49.63	22.790	0.000	0.000%	22.790		
Span 5	Pier 5 Brg.	449+51.88	22.788	0.000	0.000%	22.788	50.75	-0.10%
	Mid-Span	449+77.25	22.762	0.000	0.000%	22.762		
	Pier 6 Brg.	450+02.63	22.737	0.000	0.000%	22.737		

KISINGER CAMPO & ASSOCIATES

LEE COUNTY
ABLE CANAL PATHWAY

DESIGNED BY: JHJ 03/22
CHECKED BY: LM 03/2022

Substructure Elevations

ABLE CANAL PATHWAY OVER CHARLIE CANAL

End Bent Geometry								
Location	FGE @ FFBW	Cap Height (ft)	Cap Slope (%)	Backwall Ht. (ft.)	Average Wingwall Ht. (ft.)	Pile 1 Cut-off El. (ft.)	Pile 2 Cut-off El. (ft.)	Chk A.S. Depth @ FFBW (in.)
End Bent 1	22.682	2.50	2.00%	0.50	1.92	19.277	19.477	17.098
End Bent 6	22.736	2.50	2.00%	0.50	1.92	19.331	19.531	17.098

*Set wingwall at deck Finish Grade Elevation.

End Bent Elevations			
Description	Location	End Bent 1	End Bent 6
Left Top of Cap	EL. A	21.014	20.794
Left Top of Backwall	EL. B	21.514	21.294
Left Bottom of Cap	EL. C	18.534	18.274
Left Top of Wingwall at Begin/End Bridge	EL. D	22.944	22.721
Left Top of Wingwall at End of Wingwall	EL. E	22.930	22.714
Right Top of Cap	EL. F	20.739	21.069
Right Top of Backwall	EL. G	21.239	21.569
Right Bottom of Cap	EL. H	18.219	18.589
Right Top of Wingwall at Begin/End Bridge	EL. J	22.669	22.996
Right Top of Wingwall at End of Wingwall	EL. K	22.655	22.989

Note: View is looking towards FFBW.

Intermediate Bent Elevations							
Location	FGE @ PGL	Cap Height (ft.)	Cap Length (ft.)	Cap Slope (ft.)	Pile 1 Cutoff EL. (ft.)	Pile 2 Cutoff EL. (ft.)	Pile 3 Cutoff EL. (ft.)
Int. Bent 2	22.788	2.50	13.67	2.00%	19.381	19.483	19.585
Int. Bent 3	22.894	2.50	13.67	2.00%	19.487	19.589	19.691
Int. Bent 4	22.841	2.50	13.67	2.00%	19.435	19.536	19.638
Int. Bent 5	22.789	2.50	13.67	2.00%	19.382	19.484	19.585

Intermediate Bent Elevations					
Description	Location	Int. Bent 2	Int. Bent 3	Int. Bent 4	Int. Bent 5
Left Top of Cap	EL. A	20.846	20.952	20.900	20.847
Right Top of Cap	EL. B	21.120	21.226	21.173	21.120
Left Cap Underside	EL. C	18.346	18.452	18.400	18.347
Right Cap Underside	EL. D	18.620	18.726	18.673	18.620

Note: View is looking upstation.

2. Superstructure Design

2.1. Deck Design

SUPERSTRUCTURE DESIGN

CIP TOPPING REINFORCEMENT

CALCULATION QUALITY CONTROL		
RESPONSIBLE PROFESSIONAL / ORIGINATOR	INITIALS: JHJ	DATE: 01/22
QUALITY REVIEWER	INITIALS: LM	DATE: 02/2022

SUMMARY OF SECTION

The reinforcing layout of the ***CIP Topping*** is completed in accordance with FDOT SPI 450-450 and FDOT Structures Design Guidelines (SDG). The CIP Topping thickness is determined by the bridge length. Typically, Long Bridges require a 6.5" thick CIP Topping, but pedestrian bridges are exempt from this requirement per SDG 4.2.2. A 6.0" thick CIP Topping will be used. The reinforcing layout for the CIP Topping will be as follows:

- 6.0" CIP Topping with shrinkage reducing admixture per Specification 346 and 924
- 4500 psi Compressive Strength
- 2.0" Clear Cover
- Longitudinal #5 bars (5S01) at 9" spacing on center
- Transverse #5 bars (5S02) at 6" spacing on center
- Closed #5 stirrups (5S03) matching all 5E Bars extending from FSBs within the pockets
- Four Longitudinal #5 bars (5S04) tied by #5 closed stirrups (5S03) within each pocket
- Bent #4 Diaphragm Bars (4S05) at 9" spacing, paired with Longitudinal #5 bars (5S01) at beam ends
- Bent #4 Coping Bars (4S06) at 6" spacing, paired with Transverse #5 bars (5S02) at bridge coping

2.2. Beam Design

SUPERSTRUCTURE DESIGN

BEAM DESIGN

CALCULATION QUALITY CONTROL		
RESPONSIBLE PROFESSIONAL / ORIGINATOR	INITIALS: JHJ	DATE: 03/22
QUALITY REVIEWER	INITIALS: LM	DATE: 03/2022

SUMMARY OF SECTION

And consists of the following design cases:

- Spans 1 through 5: Exterior beam design (H10 & Pedestrian Load)
- Spans 1 through 5: Interior beam design (H10 Load)

15" Florida Slab Beams are used for the design of the bridge. Design of the beam is completed in accordance with FDOT Indexes 450-450 and 450-451 and analyzed with the FDOT LRFD Prestressed Beam Program. All loads are generated within the program. The H10 maintenance truck live load was utilized for all beam designs. Adjustments to the program calculations were made to represent the H10 axial loads.

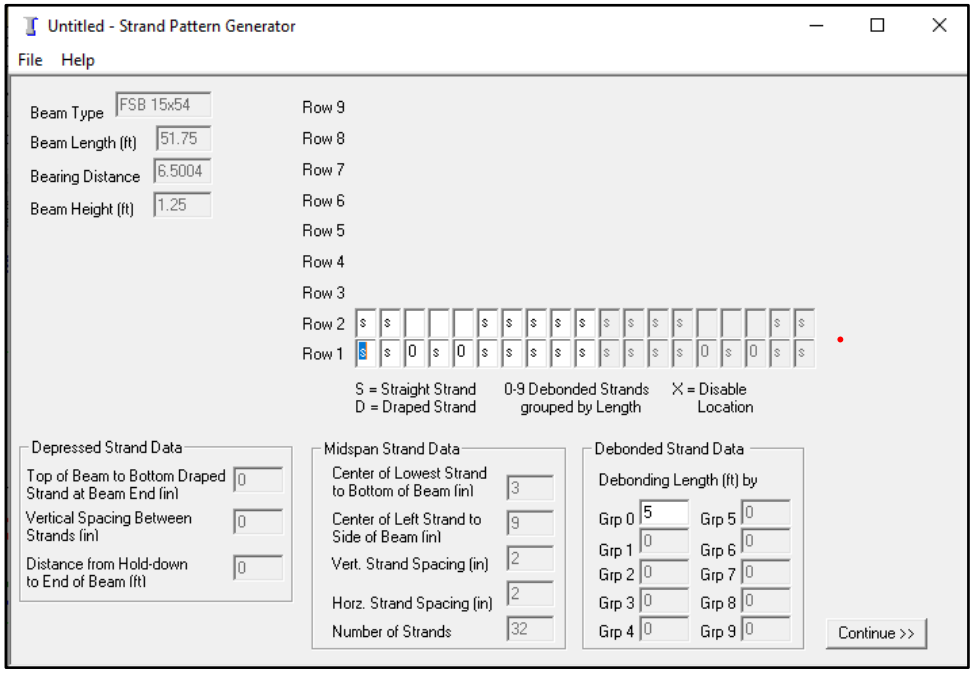
Live Load Distribution Factors for the interior and exterior beams were calculated independently and modified within the Mathcad program. The lever rule was implemented because the number of beams used does not meet the minimum allowed number of beams per the SDG 2.9.4.b.

Build-up varies, with a minimum height of 6" which includes the CIP Concrete Topping. The difference in build-up from the maximum to the minimum is entered in the program as $t_{deck.delta}$.

Beam Strand Pattern

The same strand patterns was used for all beams.

Strand Pattern:



Beam Stirrup Spacing:

Shear reinforcement inputted into FDOT LRFD Prestressed Beam Program is consistent with Standard Plans Index 450-450. The reinforcement considered for design were the 4D and 4K stirrups. The spacing input for A1 was taken from the 4D pairs bar spacing, and the area input was obtained from the combination of the first four 4D pairs and the first 4K pair per 4D pair location. The spacing input for A2 was the difference from the prescribed V1 spacing and the distance from the first 4K pair to the last 4D pair of the A1 input. The area input for A2 was obtained by the combination of the V1 spacing and the 4D pair spacing.

4K Variable Bar Spacing (Calculation)		
Location	Number of Spaces	Spacing (in.)
S1	4	12
S2	10	24

4K Variable Bar Spacing (Plans)		
Location	Number of Spaces	Spacing (in.)
S1	5	12
S2	10	24

Bearing Pad Design Data:

Beam Line:	All Spans
Live Load Reaction (kip)	17.4
Dead Load Reaction (kip)	32.6
Temperature Load Reaction (kip)	3.64
Pad Width (in.)	42.0

FSB Dimension Data:

Spans 1 through 5:

Interior/Exterior	*Change in L @ Top @ 120 days (in)*	*Change in L @ Bot. @ 120 days (in)*	*Dimension "R"*
Exterior	0.3610	0.9603	0-5/8"
Interior	0.3610	0.9603	0-5/8"

Elastic and time dependent shortening effects (Dimension "R") at mid-height of beam @ 120 days may be taken as the average of the calculated values for the top and bottom of the beam (Index 450-450).

Build-Up & Deflection Design Data:

	Dead Load Deflection @ 120 days (in)	*Camber @ 120 days (in)*
Spans 1 through 5 Ext.	1.071	2.169
Spans 1 through 5 Int.	1.124	2.169

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LEE COUNTY
ABLE CANAL PATHWAY

DESIGNED BY: JHJ 03/22
CHECKED BY: LM 02/2022

Beam Design Data

Bearing Dimensions

Span No.	Expansion Joint Width	Bearing pad width	Bearing pad length	Beam skew Begin	K1	K2	Min Bent Width	Bent Width Used
	(in)	(in)	(in)	(deg)	(ft)	(ft)	(ft)	(ft)
1	2.00	42.00	8.00	0.000	1.208	1.125	2.542	3.500
2	2.00	42.00	8.00	0.000	1.125	1.125	2.917	3.667
3	2.00	42.00	8.00	0.000	1.125	1.125	2.917	3.667
4	2.00	42.00	8.00	0.000	1.125	1.125	2.917	3.667
5	2.00	42.00	8.00	0.000	1.125	1.208	2.542	3.500

Beam Geometry

Span No.	Bridge Span	Beam Length	Design Span	Beam Grade	Bearing 1	Bearing 2
	ft	ft	ft	%	ft	ft
1	53.08	51.83	50.75	**0.20%**	1.21	1.13
2	53.00	51.83	50.75	**0.20%**	1.13	1.13
3	53.00	51.83	50.75	**-0.10%**	1.13	1.13
4	53.00	51.83	50.75	**-0.10%**	1.13	1.13
5	53.08	51.83	50.75	**-0.10%**	1.13	1.21

Beam Data Table

Location		Beam Type	Concrete Properties			Stnd Ptrn Type	Plan View Case		Angle		Beam Dimensions					
Span No.	Beam No.		Class	Strengths (ksi)			End 1	End 2	End 1	End 2	DIM W		DIM L		DIM R	
				28 Day	Release						*(ft)*		*(ft)*		*(in)*	
ALL	1 & 3	FSB 15x54	VI	8.5	6	1	1	1	90	90	4.50	4'-6"	52.49	52'-5 7/8"	0.6607	0'-0 5/8"
	2	FSB 15x54	VI	8.5	6	1	1	1	90	90	4.50	4'-6"	52.49	52'-5 7/8"	0.6607	0'-0 5/8"

Beam Data Table

Location		Reinforcing Steel																					
Span No.	Beam No.	Length Beam End to S1	3C		4D1	4D2	4D3		5E1		5E2		6Y1	6Y2	4K	No. of Bar Spaces		Bar Spacing*		Check Bar Spacing		
																				5E Spacing ≥4" & ≤10"	4D Spacing ≤12"	4K Spacing ≤V2/2
		(in)	No.	DIM C	DIM D	DIM D	No.	DIM D	No.	DIM E	No.	DIM E	DIM Y	DIM Y	No.	S1	S2	V1	V2	*(in)*		*(in)*
ALL	1 & 3	52.5	53	49.5	27.0	27.0	48	27.0	-	-	103	50.5	37.5	37.5	132	5	10	12	24	8.96	GOOD	8.46
	2	52.5	53	49.5	27.0	27.0	48	27.0	103	52.0	-	-	37.5	37.5	132	5	10	12	24	8.96	GOOD	8.46

* Spacing at center of beam

Beam Design - Live Load Check

AASHTO LRFD Guide Specifications for the Design of Pedestrian Bridges (AASHTO Ped.), AASHTO LRFD Bridge Design Specifications 9th Ed. (LRFD), and FDOT Structures Design Guidelines (SDG) 2022 are referenced.

A quick check is performed of the pedestrian live load beam moments and deflections. The values calculated are conservative estimates, considering the composite section with the CIP deck topping, but omitting additional build-up to accomodate beam camber. This demonstrates that the pedestrian deflection requirements are met and that the design considering standard bridge truck/traffic is conservative.

Geometry

Span Length: $L := 53.00 \cdot ft$

Tributary Width of Pedestrian Load per Beam: $b_{beam} := 4 \cdot ft + 7 \cdot in$

Material Properties

Deck Concrete Strength: $f_{c.d} := 4.5$

CIP Topping Concrete Elastic Modulus: $E_{c.d} := 120000 \cdot (0.145)^2 \cdot f_{c.d}^{0.33} \cdot ksi$ — $E_{c.d} = 4145 \cdot ksi$

Beam Concrete Strength: $f_c := 8.5$

Beam Concrete Elastic Modulus: (LRFD 5.4.2.4, SDG 1.4.1) $E_c := 120000 \cdot (0.145)^2 \cdot f_c^{0.33} \cdot ksi$ — $E_c = 5112 \cdot ksi$

Modular Ratio: $n := \frac{E_{c.d}}{E_c}$ — $n = 0.811$

CIP Topping Thickness: $t_d := 6 \cdot in$

CIP Topping Area: $A_d := n \cdot b_{beam} \cdot t_d$ — $A_d = 268 \cdot in^2$

CIP Distance to N.A. from top of deck: $Y_d := \frac{t_d}{2}$ — $Y_d = 3 \cdot in$

Deck Moment of Inertia (I_{xx}): $I_{xx.d} := n \cdot b_{beam} \cdot (t_d)^3 \div 12$ — $I_{xx.d} = 803 \cdot in^4$

Beam Area: $A_b := 671.76 \cdot in^2$

Beam Centroid fron Top: $Y_b := 7.91 \cdot in$

Beam Moment of Inertia (I_{xx}): $I_{xx.b} := 12950 \cdot in^4$

Composite Section Moment of Inertia (I_{xx}): $I_{xx} := I_{xx.b} + A_b \cdot Y_b^2 + I_{xx.d} + A_d \cdot Y_d^2$ — $I_{xx} = 58191 \cdot in^4$

Loads

Pedestrian Load: (AASHTO Ped. 3.1) $p_{ped} := 90 \cdot psf$

Pedestrian Truck: (AASHTO Ped. 3.2) $P_{truck} := 20 \cdot kip$
Full weight of truck is conservatively assumed as a point load.

Live Load Distribution (Lever Rule)

The number of beams does not meet the minimum number of beams to utilize AASHTO's LLDF tables. The lever rule will be used to determine the live load distribution factor. Point loads will be placed at the location of the H10 maintence truck. The left axle will start 2 feet from the curb and the right axle will be placed 6 feet from the left axle. The anaylsis will move both point loads to the right until the axles are within 2 feet of the right curb.

Width of bridge: $b_{bridge} := 13 \cdot ft + 7 \cdot in$

Distance to Curb: $dist_{curb} := 9.5 \cdot in$

Distance from Left Coping to Starting Axle Location: $loc_{start} := dist_{curb} + 2 \cdot ft$ — $loc_{start} = 2.792 \cdot ft$

Distance from Left Coping to Ending Axle Location: $loc_{end} := b_{bridge} - (dist_{curb} + 2 \cdot ft)$ — $loc_{end} = 10.792 \cdot ft$

Axle Spacing: $S_{axle} := 6 \cdot ft$

Number of Beams: $N_b := 3$

Beam Location from Left Coping: $loc_{BM1} := FIF(2, 2, .5)$

Beam Spacing: $S := 4 \cdot ft + 7 \cdot in$

Number of Load Cases: $n_{LC} := \frac{(loc_{end} - loc_{start}) - S_{axle}}{0.5 \cdot ft} + 1$ — $n_{LC} = 5$

Left Axle Location:

$$loc_{A1} := \text{for } i \in 1..n_{LC}$$
$$loc_{A1_i} \leftarrow loc_{start} + (i-1) \cdot .5 \cdot ft$$
$$loc_{A1}^T = (2.792 \quad 3.292 \quad 3.792 \quad 4.292 \quad 4.792)\, ft$$

Right Axle Location:

$$loc_{A2} := loc_{A1} + 6 \cdot ft$$
$$loc_{A2}^T = (8.792 \quad 9.292 \quad 9.792 \quad 10.292 \quad 10.792)\, ft$$

Beam Locations:

$$loc_{BM} := \text{for } i \in 1..N_b$$
$$loc_{BM_i} \leftarrow loc_{BM1} + (i-1) \cdot S$$
$$loc_{BM}^T = (2.208 \quad 6.792 \quad 11.375)\, ft$$

Live Load Distribution (Lever Rule) - Cont'd

Live Load Distribution Factor:
(Lever Rule)

$$LLDF := \text{for } i \in 1..N_b \quad \text{for } j \in 1..n_{LC} \quad \begin{cases} LLDF_{1,j} \leftarrow \left(loc_{BM_2} - loc_{A1_j}\right) \div S \div 2 \\ LLDF_{2,j} \leftarrow \left[\left(loc_{A1_j} - loc_{BM_1}\right) + \left(loc_{BM_3} - loc_{A2_j}\right)\right] \div S \\ LLDF_{3,j} \leftarrow \left(loc_{A2_j} - loc_{BM_2}\right) \div S \div 2 \end{cases}$$

$$LLDF = \begin{pmatrix} 0.4364 & 0.3818 & 0.3273 & 0.2727 & 0.2182 \\ 0.6909 & 0.6909 & 0.6909 & 0.6909 & 0.6909 \\ 0.2182 & 0.2727 & 0.3273 & 0.3818 & 0.4364 \end{pmatrix}$$

Multiple Preasence Factor:
(LRFD Table 3.6.1.1.2-1)

$$m := 1.00$$

Distribution Factor for each Beam:
(Lever Rule)

$$g := \text{for } i \in 1..N_b \quad \begin{cases} g_1 \leftarrow m \cdot \max\left[\left(LLDF^T\right)^{\langle 1 \rangle}\right] \\ g_2 \leftarrow m \cdot \max\left[\left(LLDF^T\right)^{\langle 2 \rangle}\right] \\ g_3 \leftarrow m \cdot \max\left[\left(LLDF^T\right)^{\langle 3 \rangle}\right] \end{cases}$$

$$g^T = \left(0.4364 \quad 0.6909 \quad 0.4364\right)$$

Live Load Shears

Uniform Pedestrian Load per Beam: $w_{ped} := p_{ped} \cdot b_{beam}$ $\quad w_{ped} = 0.413 \cdot \frac{kip}{ft}$

Max. Shear under Pedestrian Load: $V_{ped} := \frac{w_{ped} \cdot L}{2}$

$V_{ped} = 10.9 \cdot kip$

Max. Shear under Pedestrian Truck: $V_{truck} := g \cdot P_{truck}$

$V_{truck}^T = \left(8.7 \quad 13.8 \quad 8.7\right) \cdot kip$

Live Load Moments

Uniform Pedestrian Load per Beam: $w_{ped} = 0.413 \cdot \frac{kip}{ft}$

Max. Moment under Pedestrian Load: $M_{ped} := \frac{1}{8} \cdot \left(w_{ped}\right) \cdot L^2$

$M_{ped} = 144.8 \cdot kip \cdot ft$

Max. Moment under Pedestrian Truck: $M_{truck} := \frac{1}{4} \cdot \left(g \cdot P_{truck}\right) \cdot L$

$M_{truck}^T = \left(115.6 \quad 183.1 \quad 115.6\right) \cdot kip \cdot ft$

Live Load Deflections

Allowable Deflection: (SDG 10.4G) $\Delta_{allow} := \frac{L}{500}$ $\quad \Delta_{allow} = 1.27 \cdot in$

Beam Pedestrian Load Deflection: $\Delta_{ped} := \frac{5 \cdot \left(g \cdot w_{ped}\right) \cdot L^4}{384 \cdot E_c \cdot I_{xx}}$ $\quad \max\left(\Delta_{ped}\right) = 0.17 \cdot in$

Beam Pedestrian Truck Deflection: $\Delta_{truck} := \frac{\left(g \cdot P_{truck}\right) \cdot L^3}{48 \cdot E_c \cdot I_{xx}}$ $\quad \max\left(\Delta_{truck}\right) = 0.25 \cdot in$

FDOT LRFD Prestressed Beam Program v6.1

State Structures Design Office

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Select Beam Data Folder

- I-Beam
- FSB
- FUB
- FlatSlab
- InvertedT
- DoubleT

Category and Beams

I-Beams: *Florida I-Beams (FIBs) and formerly used Florida Bulb Tee and AASHTO Beams*
Slab Beams: *Florida Slab Beams (FSBs)*
U-Beams: *Florida U-Beams (FUBs)*
Flat Slab Previously used Florida Flat Slab Beams, superceded by FSBs
Inverted Tee: *Former FDOT Standard Inverted Tee Beams*
Double Tee: *Former FDOT Standard Double Tee Beams from the 1990's*

Data Files Folder

C:\Users\jjohnson2\Desktop\Able Canal\Superstructure\PrestressedBeamV6.1\DataFiles-FSB

Change Folder

Open Existing Data File (required). To save New Data Files, enter new data name at the end of Section IX.

- Able Canal - Span 1 _6 - Exterior.dat
- Able Canal - Spans 2 though 4 - Interior.dat
- Able Canal - Spans 2 through 4 - Exterior.dat
- Able Canal - Spans 2 through 4 - Interior.dat
- Able Canal - Spans 2-4 - Exterior - H10 Load.dat
- Able Canal - Spans 2-4 - Exterior - Ped. Load.dat
- Able Canal - Spans 2-4 - Interior - H10 Load.dat
- Able Canal - Spans 2-4 - Interior - Ped. Load.dat
- FSB-30ftSpan-12inx53inBeam.dat

Refresh List

Open File

Reset All Data

InputDataFile = "Able Canal - Spans 2-4 - Exterior - H10 Load.dat"

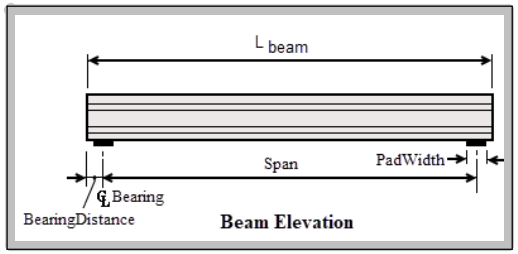
Project Information

Project Information

Project Name	Able Canal Pathway - ~~Spans 2 through 4 -~~ Exterior - H10 Load		
Project No.	435351-2-38-01		
Designed by	JHJ	*Date*	02/2022
Checked by		*Date*	

Structure Name	53ft Span - 15"x54"
Structure No.	XXXXXX

Plan, Elevation, and Cross Section Data



Note: All dimensions shown in Beam Elevation measured along centerline of beam (requiring adjustment for a skew)

Note: The top of the precast beam is the location of the origin for the coordinate system.

Superstructure Data

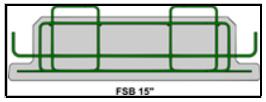
InputDataFile = "Able Canal - Spans 2-4 - Exterior - H10 Load.dat"

Beam Type: FSB12, FSB15, FSB18

Beam Types are the designations found in FDOT standards. The user can also create a coordinate file for a custom shape. Top of the beam is at the y=0 ordinate.

BMPfile := concat(substr(inpBeamType, 0, strlen(inpBeamType)), ".bmp")

BMPfile = "FSB15.bmp"



L_{beam}	51.833	ft	*See Beam Elevation above.*
Bearing Distance	6.5	in	*See Beam Elevation above.*
Pad Width	8	in	*Width of the bearing pad, see Beam Elevation above.*
Beam Spacing	4.583	ft	*Measured from beam centerline to centerline.*
Overhang	2.25	ft	*Measured from centerline of exterior beam.*
Deck Thickness	6	in	*Not including sacrificial thickness.*
$t_{sacrificial}$	0	in	*Sacrificial thickness cast with the deck (used for DL only, not section properties).* *(0" for* BridgeLengths ≤ 100 ft, *SDG 4.2.2)*
d_e	1.4167	ft	*Front face of barrier to centerline of exterior beam (3 ft max).* *(LRFD 4.6.2.2.1)*
Beam Position	Interior / Exterior		*Use either* "interior" *or* "exterior".
$Weight_{future.ws}$	0.00	kip/ft^2	*Future wearing surface.* (0.015ksf *for* BridgeLengths ≤ 100 ft, *SDG Table 2.1)*
$Weight_{barrier}$	0.080	kip/ft	*Weight of single barrier.* *(0.030 klf for Pedestrian Railing, SDG Table 2-2.1 & 0.050 klf for 8" x 6" Concrete Curb)*
Number of Barriers	2		*Number of barriers in x-section (multiplies single barrier weight).*
Number of Beams	3		*Number of beams in the span cross section.* *(LRFD 4.6.2.2.1)*
Skew	0	deg	*measured from the perpendicular to the longitudinal axis in bridge plan view.*

Dimensions Specific to Certain Beam Categories

I-Beams, U-Beams and Inverted-T Beams

$h_{buildup}$		in	*Buildup or haunch is the concrete between the bottom of deck and top of beam.*

Slab-Beams (Florida Slab Beams, Flat Slab Beams)

Slab Beam Width	54	in	*Width of the slab unit (not including Gap width)*
Slab Beam Thickness	15	in	*Thickness of the slab unit.*
$t_{deck.delta}$	1.0980	in	*maximum additional deck thickness over support to accommodate camber, used for additional DL only.*
Gap	1.016	in	*Gap distance between slab beams.(Beam Spacing - Slab Beam Width)*

Double Tee Beams

Width Double-T		ft	*Width of the Double-T unit.*
$Depth_{flange}$		in	*Depth of the flange of the Double-T unit, See Beam Cross Section above.*

Concrete Material Properties

FDOT Environmental Classification: Slightly / Moderately / Extremely — *Environmental Classification determines the Allowable Tension Stress*

f`c of deck or WS	4.5	ksi	*Strength of deck or wearing surface concrete.*
f`c of beam or slab	8.5	ksi	*28 day concrete strength of beam or slab.*
f`ci of beam or slab	6	ksi	*Release concrete strength of beam or slab.*

Help - Conc. Strength

Permit Truck Axle Loads and Spacings

Default value for the permit truck is the FL-120 truck

Reset Permit Truck Data

Number of Permit Axles 3 — *Number of wheel loads that comprise the permit truck*

Permit Uniform LL 0 kip/ft — *Permit Uniform Live Load coincident with truck load*

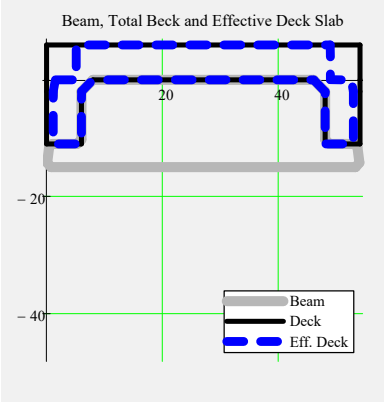
Help - Permit Truck

	Axle Number										
	1	2	3	4	5	6	7	8	9	10	11
Load (kips)	0.01	0.01	0.01	0	0	0	0	0	0	0	0
Spacing (ft)	n/a	14	14	0	0	0	0	0	0	0	0

Custom Beam Coordinates, Initialize Data, Section Properties and Dead Loads

LRFD Live Load Distribution Factors, User Defined Live Load Distribution Factors and Section Properties

Section View: Beam, Deck and Effective Deck Slab



$g_{mom} = 0.44$

$g_{shear} = 0.44$

BeamType = "FSB15"

$t_{deck} = 6 \cdot in$

$h_{buildup} = 0 \cdot in$

$h_{beam} = 15 \cdot in$

BeamSpacing = $54 \cdot in$

$b_e = 54.01 \cdot in$ *effective slab width*

Additional Dead Load Data:

Uniform Dead Loads

Additional Uniform Noncomposite DL	0.0	$\frac{kip}{ft}$	$w_{deck} = 0.48 \cdot \frac{kip}{ft}$ $w_{beam} = 0.7 \cdot \frac{kip}{ft}$ $w_{forms} = 0.00 \cdot \frac{kip}{ft}$
Additional Uniform Composite DL	0	$\frac{kip}{ft}$	$w_{future.ws} = 0 \cdot \frac{kip}{ft}$ $w_{barrier} = 0.05 \cdot \frac{kip}{ft}$

Diaphragms (Point) Dead Loads $L_{beam} = 51.83$ ft

End Diaphragms or Point Loads over bearing included in bearing reaction calculation only.
Intermediate Diaphragms or Point Loads included in shear, moment, and bearing reaction calculations

Diaphragm (Point) Dead Loads and Location

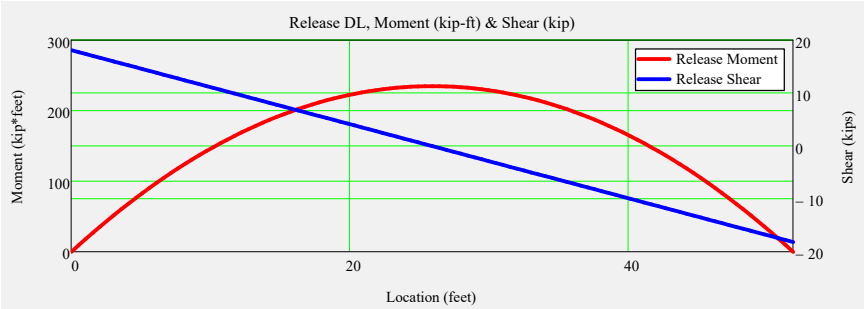
	Begin	Int. 1	Int. 2	Int. 3	End
Load (kips)	0	0	0	0	0
Distance (ft)	0	0	0	0	0

Distance measured from CL Bearing at begin bridge

Dead Loads, DL Moment, DL Shears

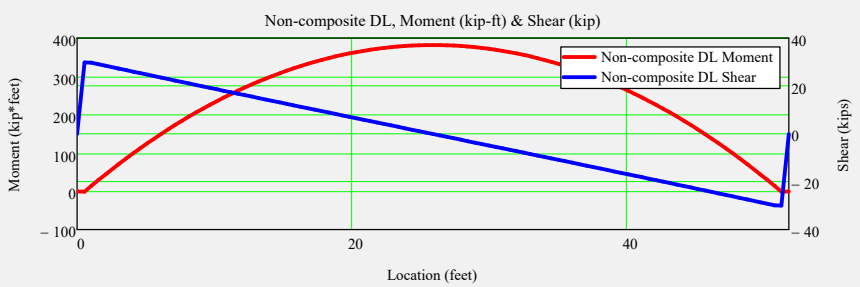
Release Dead Load Moments and Shears

$L_{span} = 50.75\ ft$



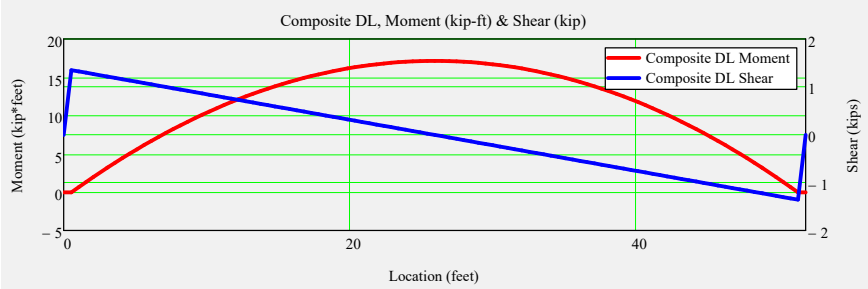
$\max(V_{release}) = 18.1 \cdot kip$ $\max(M_{release}) = 235.0 \cdot kip \cdot ft$ $\min(V_{release}) = -18.1 \cdot kip$

Noncomposite Dead Load Moments and Shears



$\max(V_{dl.non.comp}) = 29.9 \cdot kip$ $\max(M_{dl.non.comp}) = 382.3 \cdot kip \cdot ft$ $\min(V_{dl.non.comp}) = -29.9 \cdot kip$

Composite Dead Load Moments and Shear

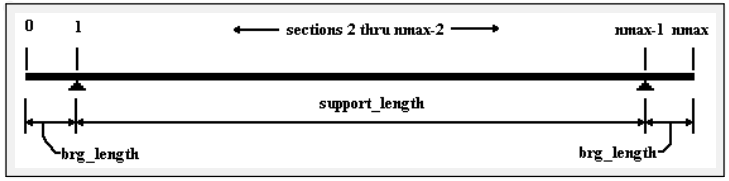


$\max(V_{dl.comp}) = 1.4 \cdot kip$ $\max(M_{dl.comp}) = 17.2 \cdot kip \cdot ft$ $\min(V_{dl.comp}) = -1.4 \cdot kip$

Live Load Moments and Shears

$WRITEPRN(\text{"coord.dat"}) := \frac{Coord_{beam}}{ft}$

Live Load Moments and Shears



$L_{c.c.bearings} := L_{beam} - 2 \cdot BearingDistance$ *Distance between the center line of bearings*

$Location_1 := BearingDistance$ $Location_0 := 0 \cdot ft$ *Location vector defines the x location of the sections investigated relative to the start of the beam*

$Location_{ns} := \frac{L_{c.c.bearings}}{nmax - 2} \cdot (ns - 1) + Location_1$ $Location_{nmax} := L_{beam}$ *The Location vector need not be in even increments*

$a_n := Location_n - BearingDistance$ *a vector gives the section x coordinate relative to the center line of the first bearing*

$b_n := L_{c.c.bearings} - a_n$ *b vector gives the distance from the section to the center line of the last bearing*

$P := \begin{pmatrix} 4 & 16 & 0 \\ 16 & 4 & 0 \\ 0 & 0 & 0 \end{pmatrix} \cdot kip$ *P vector set contains the magnitudes of the system of concentrated loads (HS20 is default)*

$d := \begin{pmatrix} 0 & 0 & 0 \\ 14 & 14 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{pmatrix} \cdot ft$ *d vector set contains the spacings between the concentrated loads. The first and last values are place holders and should always be zero*

$PermitAxleLoad^T = (0.01 \quad 0.01 \quad 0.01) \cdot kip$

$PermitAxleSpacing^T = (0 \quad 14 \quad 14 \quad 0)\ ft$

$FatigueAxleLoad := \begin{pmatrix} 0 \\ 0 \\ 0 \end{pmatrix} \cdot kip$ $FatigueAxleSpacing := \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot ft$

$P^{\langle 3 \rangle} := \text{PermitAxleLoad}$ $\quad P^{\langle 4 \rangle} := \text{reverse(PermitAxleLoad)}$

$d^{\langle 3 \rangle} := \text{PermitAxleSpacing}$ $\quad d^{\langle 4 \rangle} := \text{reverse(PermitAxleSpacing)}$

$P^{\langle 5 \rangle} := \text{FatigueAxleLoad}$ $\quad P^{\langle 6 \rangle} := \text{reverse(FatigueAxleLoad)}$

$d^{\langle 5 \rangle} := \text{FatigueAxleSpacing}$ $\quad d^{\langle 6 \rangle} := \text{reverse(FatigueAxleSpacing)}$

$$P = \begin{pmatrix} 4 & 16 & 0 & 0.01 & 0.01 & 0 & 0 \\ 16 & 4 & 0 & 0.01 & 0.01 & 0 & 0 \\ 0 & 0 & 0 & 0.01 & 0.01 & 0 & 0 \end{pmatrix} \cdot kip \qquad d = \begin{pmatrix} 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 14 & 14 & 0 & 14 & 14 & 0 & 0 \\ 0 & 0 & 0 & 14 & 14 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 \end{pmatrix} ft$$

$NumberOfTrucks := cols(P) = 7$ *number of trucks*

$nt := 0 .. (NumberOfTrucks - 1)$ *nt vector of length equal to number of Trucks*

$$TruckLength_{nt} := \sum_{qt} \left(d^{\langle nt \rangle}\right)_{qt}$$ *total length of the concentrated load system*

$$inc_{nt} := floor\left[\frac{300}{\left(\frac{TruckLength_{nt}}{L_{beam}}\right) + 1}\right]$$ *number of even increments that the load system will use to advance across the span*

$$add_inc_{nt} := ceil\left(\frac{TruckLength_{nt} \cdot inc_{nt}}{L_{beam}}\right)$$ *additional increments required to get the load system off of the span*

$lastp_{nt} := 199$ *value of the last increment*

$$step_{nt} := \frac{L_{beam} + TruckLength_{nt} - 0.0000001 \cdot ft}{lastp_{nt}}$$ *distance between increments*

Remove units for calculations

$a := \frac{a}{ft} \qquad b := \frac{b}{ft} \qquad P := \frac{P}{kip} \qquad d := \frac{d}{ft}$

Functions used to determine moments and shears

$$fM_{int}(P, a, b, loc, brg, sl, M, n, z) := \begin{cases} Ans \leftarrow \frac{P_z \cdot a_n \cdot \left[sl - \left(loc_z - brg\right)\right]}{sl} + M_n & \text{if } a_n < \left(loc_z - brg\right) \\ Ans \leftarrow \frac{P_z \cdot b_n \cdot \left(loc_z - brg\right)}{sl} + M_n & \text{otherwise} \end{cases}$$

$$fV_{int}(P, loc, brg, sl, SZ, n, z) := \begin{cases} Ans \leftarrow \frac{P_z \cdot \left[sl - \left(loc_z - brg\right)\right]}{sl} + SZ_n & \text{if } a_n < \left(loc_z - brg\right) \\ Ans \leftarrow \frac{P_z \cdot \left(brg - loc_z\right)}{sl} + SZ_n & \text{otherwise} \end{cases}$$

$$fM_{start}(P, a, b, loc, brg, sl, M, n, z) := \begin{cases} Ans \leftarrow M_n + \left(loc_z - brg\right) \cdot P_z & \text{if } (n > 0) \cdot (n < 2) \\ Ans \leftarrow \left(loc_z - brg\right) \cdot P_z \cdot \frac{b_n}{sl} + M_n & \text{otherwise} \end{cases}$$

$$fV_{start}(P, loc, brg, sl, SZ, n, z) := Ans \leftarrow \left[P_z \cdot \frac{\left(brg - loc_z\right) + sl}{sl} - P_z\right] + SZ_n$$

$$fM_{end}(P, a, b, loc, brg, sl, M, n, z, k) := \begin{cases} Ans \leftarrow P_z \cdot \left[(sl + brg) - loc_z\right] + M_n & \text{if } [n > (k - 2)] \cdot (n < k) \\ Ans \leftarrow P_z \cdot \left[(sl + brg) - loc_z\right] \cdot \frac{sl - b_n}{sl} + M_n & \text{otherwise} \end{cases}$$

$$f_{end}(P, loc, brg, sl, SZ, n, z, k) := Ans \leftarrow P_z \cdot \left(1 - \frac{loc_z - brg}{sl}\right) + SZ_n$$

```
M_ll(a, b, L, k, brg, P, d, lp, nl, st) :=
  sl ← L − 2·brg
  for n ∈ 0..k
    for j ∈ 0..5
      R_{n,j} ← 0
  for q ∈ 0..lp
    for n ∈ 0..k
      M_n ← 0
      SZ_n ← 0
    al ← q·st
    for z ∈ 0..(nl − 1)
      loc_z ← al − Σ_{t=0}^{z} d_t
      for n ∈ 1..(k − 1)                                  if (loc_z < brg)·(loc_z ≥ 0)
        M_n ← fM_start(P, a, b, loc, brg, sl, M, n, z)
        SZ_n ← fV_start(P, loc, brg, sl, SZ, n, z)
      for n ∈ 1..(k − 1)                                  if (loc_z > brg)·[loc_z ≤ (sl + brg)]
        M_n ← fM_int(P, a, b, loc, brg, sl, M, n, z)
        SZ_n ← fV_int(P, loc, brg, sl, SZ, n, z)
      for n ∈ 2..(k − 1)                                  if ⌈loc_z > (sl + brg)⌉·(loc_z ≤ L)
```

```
M_n ← fM_end(P, a, b, loc, brg, sl, M, n, z, k)
SZ_n ← f_end(P, loc, brg, sl, SZ, n, z, k)
for n ∈ 1 .. k − 1
  R_n,0 ← M_n  if R_n,0 < M_n
  R_n,1 ← M_n  if R_n,1 > M_n
  if R_n,2 < SZ_n
    R_n,2 ← SZ_n
    R_n,3 ← M_n
  if R_n,4 > SZ_n
    R_n,4 ← SZ_n
    R_n,5 ← M_n
R
```

$$M_{truck_{nt}} := M_{ll}\left[a, b, \left(\frac{L_{beam}}{ft}\right), nmax, \left(\frac{BearingDistance}{ft}\right), P^{\langle nt \rangle}, d^{\langle nt \rangle}, lastp_{nt}, rows(P), \left(\frac{step_{nt}}{ft}\right)\right]$$

Graphs of the Lane, Truck, Permit Truck, Strength I, and Strength II Live Loads

The results are contained in a matrix with each row associated with the same numbered section and the columns containing the following data: *Column 0 --- the maximum positive moment*

Column 1 --- the maximum negative moment (if any depending on the Bearing Distance)
Column 2 --- maximum positive shear
Column 3 --- moment associated with the maximum positive shear
Column 4 --- maximum negative shear
Column 5 --- moment associated with the maximum negative shear

```
compl(A, ns, ne, col) :=
  res ← (A_ns)^⟨col⟩
  for j ∈ (ns + 1) .. ne
    res ← augment[res, (A_j)^⟨col⟩]
  res
```

```
sift(AA, ns, ne, nax) :=
  for ik ∈ 0 .. nax
    test ← 0
    res_ik ← 0
    for j ∈ ns .. ne
      if AA_ik,j > test
        test ← AA_ik,j
        res_ik ← j
  res
```

Design Truck Maximum Moments, Maximum Shears, and Moments associated with Maximum Shears

$pos_m_tot := compl(M_{truck}, 0, 2, 0)$

$neg_m_tot := compl(M_{truck}, 0, 2, 1)$

$M_{truck.pos_n} := max\left[\left(pos_m_tot^T\right)^{\langle n \rangle}\right] \cdot kip \cdot ft$

$M_{truck.neg_n} := min\left[\left(neg_m_tot^T\right)^{\langle n \rangle}\right] \cdot kip \cdot ft$

$pos_v_tot := compl(M_{truck}, 0, 2, 2)$

$pos_vm_tot := compl(M_{truck}, 0, 2, 3)$

$pos_v_index := sift[(pos_v_tot), 0, 2, nmax]$

$V_{truck.pos_n} := pos_v_tot_{n, pos_v_index_n} \cdot kip$

$Mshr_{truck.pos_n} := pos_vm_tot_{n, pos_v_index_n} \cdot kip \cdot ft$

$neg_v_tot := compl(M_{truck}, 0, 2, 4)$

$neg_vm_tot := compl(M_{truck}, 0, 2, 5)$

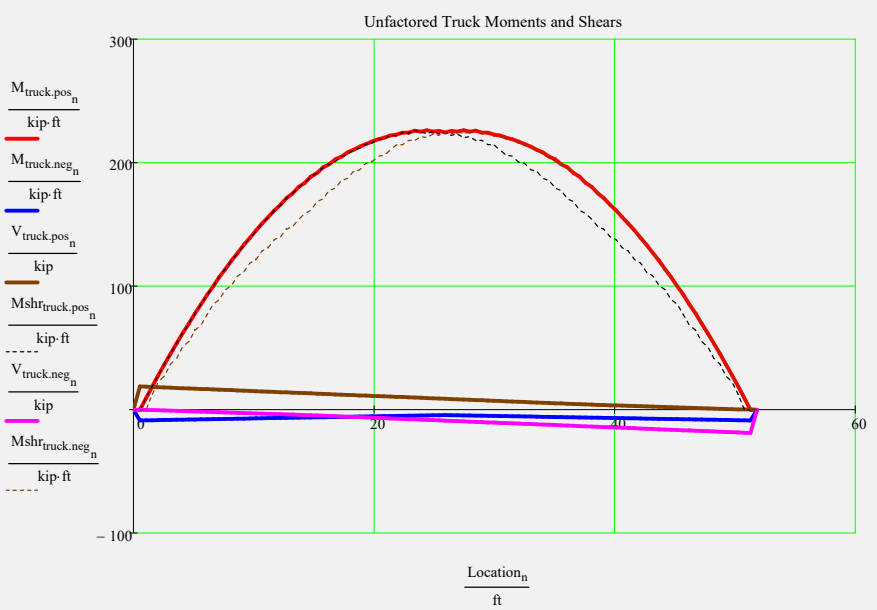
$neg_v_index := sift[neg_v_tot \cdot (-1), 0, 2, nmax]$

$V_{truck.neg_n} := neg_v_tot_{n, neg_v_index_n} \cdot kip$

$Mshr_{truck.neg_n} := neg_vm_tot_{n, neg_v_index_n} \cdot kip \cdot ft$

$pos_m_fatigue := compl(M_{truck}, 5, 6, 0)$

$M_{fatiguetruck.pos_n} := max\left[\left(pos_m_fatigue^T\right)^{\langle n \rangle}\right] \cdot kip \cdot ft$



$\max(M_{truck.pos}) = 226.4 \cdot kip \cdot ft$ $\quad$ $\min(M_{truck.neg}) = -8.7 \cdot kip \cdot ft$

$\max(V_{truck.pos}) = 18.9 \cdot kip$ $\quad$ $\max(Mshr_{truck.pos}) = 226 \cdot kip \cdot ft$

Lane Load Maximum Moments, Maximum Shears, and Moments associated with Maximum Shears

$$M_{lane.pos_n} := \frac{0 \cdot \frac{kip}{ft} \cdot a_n \cdot ft}{2} \cdot \left(L_{c.c.bearings} - a_n \cdot ft\right)$$

$M_{lane.pos_0} := 0 \cdot kip \cdot ft$ $\quad$ $M_{lane.pos_{nmax}} := 0 \cdot kip \cdot ft$

$$M_{lane.neg_n} := \frac{-\left(0 \cdot \frac{kip}{ft}\right) \cdot BearingDistance^2}{2}$$

$M_{lane.neg_0} := 0 \cdot kip \cdot ft$ $\quad$ $M_{lane.neg_{nmax}} := 0 \cdot kip \cdot ft$

$$V_{lane.neg_n} := \left[\frac{-0 \cdot \frac{kip}{ft} \cdot \left(a_n \cdot ft\right)^2}{2 \cdot L_{c.c.bearings}}\right]$$

$$V_{lane.pos_n} := \frac{0 \cdot \frac{kip}{ft} \cdot \left(b_n \cdot ft\right)^2}{2 \cdot L_{c.c.bearings}}$$

$V_{lane.neg_0} := 0 \cdot kip$ $\quad$ $V_{lane.neg_{nmax}} := 0 \cdot kip$ $\quad$ $V_{lane.pos_0} := 0 \cdot kip$ $\quad$ $V_{lane.pos_{nmax}} := 0 \cdot kip$

$Mshr_{lane.neg_n} := \left(-V_{lane.neg}\right)_n \cdot b_n \cdot ft$ $\quad$ $Mshr_{lane.pos_n} := V_{lane.pos_n} \cdot a_n \cdot ft$

$PermitUniformLoad := 0.0 \cdot klf$

$$M_{lane.permit.pos_n} := \frac{\left(PermitUniformLoad + 0.1 \cdot \frac{lbf}{ft}\right) \cdot a_n \cdot ft}{2} \cdot \left(L_{c.c.bearings} - a_n \cdot ft\right)$$

$M_{lane.permit.pos_0} := 0 \cdot kip \cdot ft$

$M_{lane.permit.pos_{nmax}} := 0 \cdot kip \cdot ft$

$$M_{lane.permit.neg_n} := \frac{-\left(PermitUniformLoad + 0.1 \cdot \frac{lbf}{ft}\right) \cdot BearingDistance^2}{2}$$

$M_{lane.permit.neg_0} := 0 \cdot kip \cdot ft$

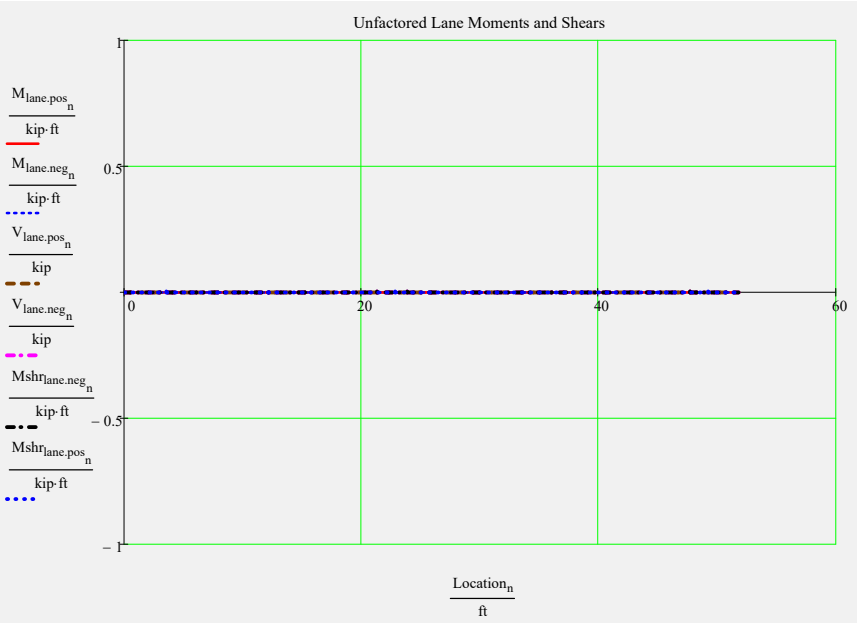
$M_{lane.permit.neg_{nmax}} := 0 \cdot kip \cdot ft$

$$V_{lane.permit.neg_n} := \left[\frac{-\left(PermitUniformLoad + 0.1 \cdot \frac{lbf}{ft}\right) \cdot \left(a_n \cdot ft\right)^2}{2 \cdot L_{c.c.bearings}}\right]$$

$$V_{lane.permit.pos_n} := \frac{\left(PermitUniformLoad + 0.1 \cdot \frac{lbf}{ft}\right) \cdot \left(b_n \cdot ft\right)^2}{2 \cdot L_{c.c.bearings}}$$

$V_{lane.permit.neg_0} := 0 \cdot kip$ $\quad$ $V_{lane.permit.neg_{nmax}} := 0 \cdot kip$ $\quad$ $V_{lane.permit.pos_0} := 0 \cdot kip$ $\quad$ $V_{lane.permit.pos_{nmax}} := 0 \cdot kip$

$Mshr_{lane.permit.neg_n} := \left(-V_{lane.permit.neg}\right)_n \cdot b_n \cdot ft$ $\quad$ $Mshr_{lane.permit.pos_n} := V_{lane.permit.pos_n} \cdot a_n \cdot ft$



$\max(M_{lane.pos}) = 0 \cdot kip \cdot ft$ $\qquad$ $\min(M_{lane.neg}) = 0 \cdot kip \cdot ft$

$\max(V_{lane.pos}) = 0 \cdot kip$ $\qquad$ $\max(Mshr_{lane.pos}) = 0 \cdot kip \cdot ft$

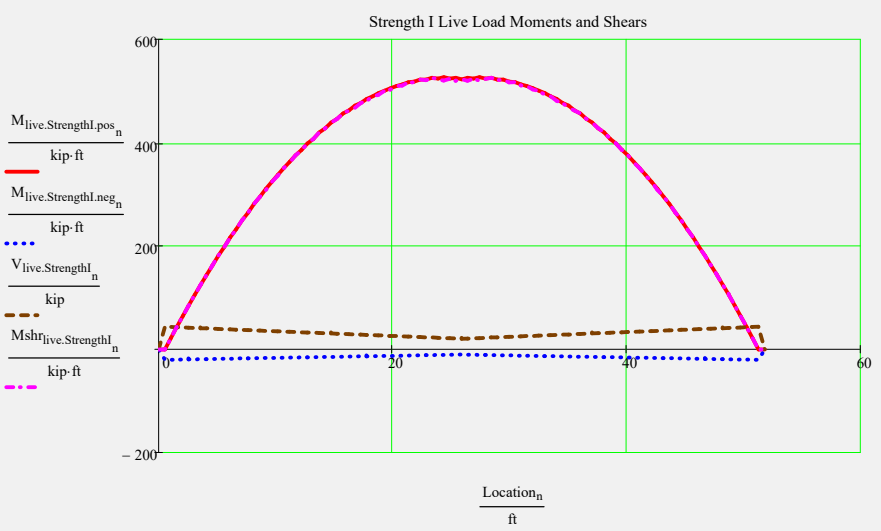
Strength I Live Load Maximum Moments, Maximum Shears, and Moments associated with Maximum Shears (Undistributed)

$M_{live.StrengthI.pos_n} := \left[M_{truck.pos_n} \cdot (1.33) + M_{lane.pos_n}\right] \cdot 1.75$ $\qquad$ $M_{live.StrengthI.neg_n} := \left[M_{truck.neg_n} \cdot (1.33) + M_{lane.neg_n}\right] \cdot 1.75$

$V_{live.StrengthIa_n} := \left[V_{truck.pos_n} \cdot (1.33) + V_{lane.pos_n}\right] \cdot 1.75$ $\qquad$ $V_{live.StrengthIb_n} := \left[V_{truck.neg_n} \cdot (1.33) + V_{lane.neg_n}\right] \cdot 1.75$

$V_{live.StrengthI_n} := \left|\text{if}\left(\left|V_{live.StrengthIa_n}\right| > \left|V_{live.StrengthIb_n}\right|, V_{live.StrengthIa_n}, V_{live.StrengthIb_n}\right)\right|$

$Mshr_{live.StrengthI_n} := \text{if}\left[\left|V_{live.StrengthIa_n}\right| > \left|V_{live.StrengthIb_n}\right|, \left(Mshr_{truck.pos_n} \cdot 1.33 + Mshr_{lane.pos_n}\right), \left(\left(Mshr_{truck.neg_n} \cdot 1.33 \ldots + Mshr_{lane.neg_n}\right)\right)\right] \cdot 1.75$



$\max(M_{live.StrengthI.pos}) = 527 \cdot kip \cdot ft$ $\qquad$ $\min(M_{live.StrengthI.neg}) = -20 \cdot kip \cdot ft$

$\max(V_{live.StrengthI}) = 44 \cdot kip$ $\qquad$ $\max(Mshr_{live.StrengthI}) = 526 \cdot kip \cdot ft$

Permit Truck Maximum Moments, Maximum Shears, and Moments associated with Maximum Shears

$ppos_m_tot := compl(M_{truck}, 3, 4, 0)$ $\qquad$ $M_{permit.pos_n} := \max\left[\left(ppos_m_tot^T\right)^{\langle n \rangle}\right] \cdot kip \cdot ft$

$pneg_m_tot := compl(M_{truck}, 3, 4, 1)$ $\qquad$ $M_{permit.neg_n} := \min\left[\left(pneg_m_tot^T\right)^{\langle n \rangle}\right] \cdot kip \cdot ft$

$ppos_v_tot := compl(M_{truck}, 3, 4, 2)$ $\qquad$ $ppos_vm_tot := compl(M_{truck}, 3, 4, 3)$

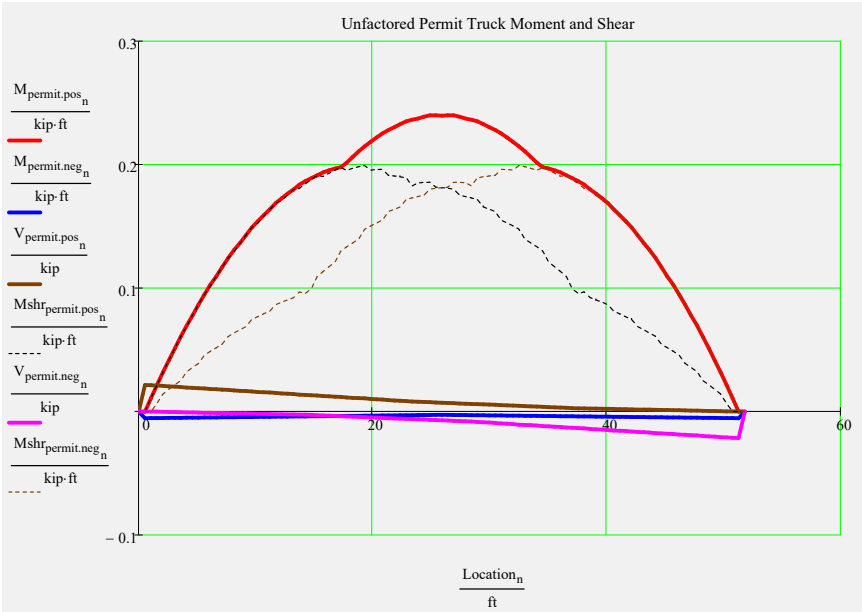
$ppos_v_index := sift[(ppos_v_tot), 0, 1, nmax]$ $\qquad$ $V_{permit.pos_n} := ppos_v_tot_{n, ppos_v_index_n} \cdot kip$

$Mshr_{permit.pos_n} := ppos_vm_tot_{n, ppos_v_index_n} \cdot kip \cdot ft$

$pneg_v_tot := compl(M_{truck}, 3, 4, 4)$ $\qquad$ $pneg_vm_tot := compl(M_{truck}, 3, 4, 5)$

$pneg_v_index := sift[pneg_v_tot \cdot (-1), 0, 1, nmax]$ $\qquad$ $V_{permit.neg_n} := pneg_v_tot_{n, pneg_v_index_n} \cdot kip$

$Mshr_{permit.neg_n} := pneg_vm_tot_{n, pneg_v_index_n} \cdot kip \cdot ft$



$\max(M_{permit.pos}) = 0 \cdot kip \cdot ft$ $\qquad$ $\min(M_{permit.neg}) = -0 \cdot kip \cdot ft$

$\max(V_{permit.pos}) = 0 \cdot kip$ $\qquad$ $\max(Mshr_{permit.pos}) = 0 \cdot kip \cdot ft$

Strength II Live Load Maximum Moments, Maximum Shears, and Moments associated with Maximum Shears (undistributed)

$M_{live.StrengthII.pos_n} := \left(1.33\, M_{permit.pos_n} + M_{lane.permit.pos_n}\right)\cdot 1.35$ *(LRFD C3.6.2.1)*

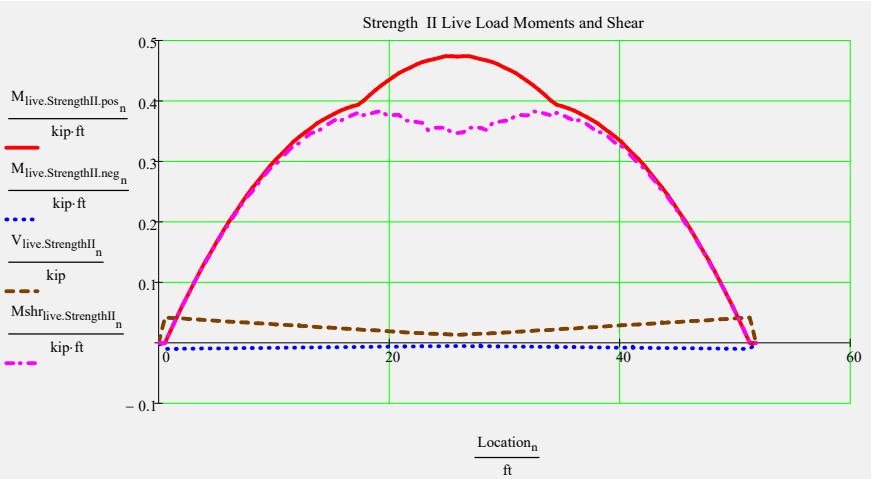
$M_{live.StrengthII.neg_n} := \left(1.33\, M_{permit.neg_n} + M_{lane.permit.neg_n}\right)\cdot 1.35$

$V_{live.StrengthIIa_n} := \left(1.33\cdot V_{permit.pos_n} + V_{lane.permit.pos_n}\right)\cdot 1.35$

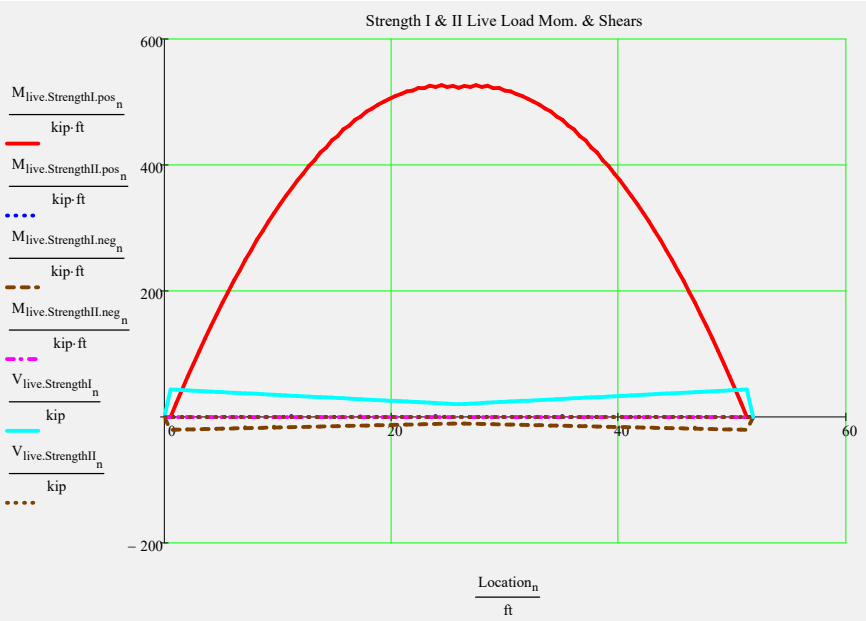
$V_{live.StrengthIIb_n} := \left(1.33\cdot V_{permit.neg_n} + V_{lane.permit.neg_n}\right)\cdot 1.35$

$V_{live.StrengthII_n} := \left|\text{if}\left(\left|V_{live.StrengthIIa_n}\right| > \left|V_{live.StrengthIIb_n}\right|, V_{live.StrengthIIa_n}, V_{live.StrengthIIb_n}\right)\right|$

$Mshr_{live.StrengthII_n} := \text{if}\left(\left|V_{live.StrengthIIa_n}\right| > \left|V_{live.StrengthIIb_n}\right|, Mshr_{permit.pos_n}\cdot 1.33 \ldots + Mshr_{lane.permit.pos_n}, Mshr_{permit.neg_n}\cdot 1.33 \ldots + Mshr_{lane.permit.neg_n}\right)\cdot 1.35$



$\max\left(M_{live.StrengthII.pos}\right) = 0\cdot kip\cdot ft$



$\max\left(M_{live.StrengthI.pos}\right) = 527\cdot kip\cdot ft$

Distributed Live Load Maximum Moments, Maximum Shears, and Moments associated with Maximum Shears

$M_{dist.live.pos_n} := \left(1.33\cdot M_{truck.pos_n} + M_{lane.pos_n}\right)\cdot g_{mom}$

$M_{dist.live.neg_n} := \left(1.33\cdot M_{truck.neg_n} + M_{lane.neg_n}\right)\cdot g_{mom}$

$V_{dist.live.pos_n} := \left(1.33\cdot V_{truck.pos_n} + V_{lane.pos_n}\right)\cdot g_{shear}$

$V_{dist.live.neg_n} := \left(1.33\cdot V_{truck.neg_n} + V_{lane.neg_n}\right)\cdot g_{shear}$

$Mshr_{dist.live.pos_n} := \left(1.33\cdot Mshr_{truck.pos_n} + Mshr_{lane.pos_n}\right)\cdot g_{mom}$

$Mshr_{dist.live.neg_n} := \left(1.33\cdot Mshr_{truck.neg_n} + Mshr_{lane.neg_n}\right)\cdot g_{mom}$

$\max\left(M_{dist.live.pos}\right) = 131.4\cdot kip\cdot ft$ $\max\left(V_{dist.live.pos}\right) = 11\cdot kip$ $\max\left(Mshr_{dist.live.pos}\right) = 131.2\cdot kip\cdot ft$

$\min(M_{dist.live.neg}) = -5 \cdot kip \cdot ft$ $\min(V_{dist.live.neg}) = -11 \cdot kip$ $\max(Mshr_{dist.live.neg}) = 131.2 \cdot kip \cdot ft$

Distributed Live Load - Permit Loading

$M_{dist.live.permit.pos_n} := \left(1.33 \cdot M_{permit.pos_n} + M_{lane.permit.pos_n}\right) \cdot g_{mom}$

$M_{dist.live.permit.neg_n} := \left(1.33 \cdot M_{permit.neg_n} + M_{lane.permit.neg_n}\right) \cdot g_{mom}$

$V_{dist.live.permit.pos_n} := \left(1.33 \cdot V_{permit.pos_n} + V_{lane.permit.pos_n}\right) \cdot g_{shear}$

$V_{dist.live.permit.neg_n} := \left(1.33 \cdot V_{permit.neg_n} + V_{lane.permit.neg_n}\right) \cdot g_{shear}$

$Mshr_{dist.live.permit.pos_n} := \left(1.33 \cdot Mshr_{permit.pos_n} + Mshr_{lane.permit.pos_n}\right) \cdot g_{mom}$

$Mshr_{dist.live.permit.neg_n} := \left(1.33 \cdot Mshr_{permit.neg_n} + Mshr_{lane.permit.neg_n}\right) \cdot g_{mom}$

$\max(M_{dist.live.permit.pos}) = 0.2 \cdot kip \cdot ft$ $\max(V_{dist.live.permit.pos}) = 0 \cdot kip$

$\min(M_{dist.live.permit.neg}) = -0 \cdot kip \cdot ft$ $\min(V_{dist.live.permit.neg}) = -0 \cdot kip$

$\max(Mshr_{dist.live.permit.pos}) = 0.1 \cdot kip \cdot ft$

$\max(Mshr_{dist.live.permit.neg}) = 0.1 \cdot kip \cdot ft$

Distributed Live Load - Fatigue Truck

$M_{dist.fatigue.pos_n} := \left(1.15 \cdot M_{fatiguetruck.pos_n}\right) \cdot g_{mom.fatigue}$

Bearing Reactions

$$f_{trk.reaction}(trk, asp, bd, lgth) := \left| \begin{array}{l} stop \leftarrow last(trk) \\ react \leftarrow 0 \\ \text{for } i \in 0..stop \\ \quad \left| \begin{array}{l} sum \leftarrow 0 \\ \text{for } j \in i..stop \\ \quad \left| \begin{array}{l} fact \leftarrow \dfrac{lgth - bd}{lgth - 2 \cdot bd} \ \text{ if } j = i \\ fact \leftarrow \dfrac{(lgth - 2 \cdot bd) - \left[\left(\sum_{n=i+1}^{j} asp_n\right) - bd\right]}{lgth - 2 \cdot bd} \ \text{ otherwise} \\ sum \leftarrow sum + fact \cdot trk_j \\ chk_{i,j} \leftarrow fact \end{array} \right. \\ react \leftarrow sum \ \text{ if } sum > react \end{array} \right. \\ react \end{array} \right.$$

$$Reaction_{truck} := \left| \begin{array}{l} ans \leftarrow 0 \\ \text{for } i \in 0..2 \\ \quad \left| \begin{array}{l} trkreact \leftarrow f_{trk.reaction}\left(P^{\langle i \rangle}, d^{\langle i \rangle}, \dfrac{BearingDistance}{ft}, \dfrac{L_{beam}}{ft}\right) \\ ans \leftarrow trkreact \ \text{ if } trkreact > ans \end{array} \right. \\ ans \end{array} \right. \cdot kip$$

$Reaction_{truck} = 19.1 \cdot kip$

$$Reaction_{permit.trk} := \left| \begin{array}{l} ans \leftarrow 0 \\ \text{for } i \in 3..4 \\ \quad \left| \begin{array}{l} trkreact \leftarrow f_{trk.reaction}\left(P^{\langle i \rangle}, d^{\langle i \rangle}, \dfrac{BearingDistance}{ft}, \dfrac{L_{beam}}{ft}\right) \\ ans \leftarrow trkreact \ \text{ if } trkreact > ans \end{array} \right. \\ ans \end{array} \right. \cdot kip$$

$Reaction_{permit.trk} = 0 \cdot kip$

$Reaction_{lane} := 0 \cdot \dfrac{kip}{ft} \cdot \dfrac{L_{beam}}{2} = 0 \cdot kip$

$Reaction_{lane.permit} := \left(PermitUniformLoad + 0.1 \dfrac{lbf}{ft}\right) \cdot \dfrac{L_{beam}}{2} = 0 \cdot kip$

$Reaction_{composite.dl} := w_{composite} \cdot \dfrac{L_{beam}}{2} = 1.4 \cdot kip$

$$Reaction_{noncomposite.dl} := w_{noncomposite} \cdot \frac{L_{beam}}{2} \ldots + \max\left[\left(\left|EndDiaphragmA\right| + \left|V_{point_1}\right|\right), \left(\left|EndDiaphragmE\right| + \left|V_{point_{nmax-1}}\right|\right)\right] = 30.6 \cdot kip$$

$$Reaction_{HL93} := \left(Reaction_{truck} \cdot 1.33 + Reaction_{lane}\right) \cdot g_{shear} = 11.1 \cdot kip$$

$$Reaction_{permit} := \left(Reaction_{permit.trk} \cdot 1.33 + Reaction_{lane.permit}\right) \cdot g_{shear} = 0 \cdot kip$$

$$Reaction_{LL} := Reaction_{HL93} = 11.1 \cdot kip$$

$$Reaction_{DL} := Reaction_{composite.dl} + Reaction_{noncomposite.dl} = 31.9 \cdot kip$$

```
WRITEPRN("beam.dat") := | r0 ← 0  if BeamType = "TypeII"
                        | r0 ← 1  if BeamType = "TypeIII"
                        | r0 ← 2  if BeamType = "TypeIV"
                        | r0 ← 3  if BeamType = "TypeV"
                        | r0 ← 4  if BeamType = "TypeVI"
                        | r0 ← 5  if BeamType = "FBT54"
                        | r0 ← 6  if BeamType = "FBT63"
                        | r0 ← 7  if BeamType = "FBT72"
                        | r0 ← 8  if BeamType = "FBT78"
                        | r0 ← 10  if BeamType = "FIB36"
                        | r0 ← 11  if BeamType = "FIB45"
                        | r0 ← 12  if BeamType = "FIB54"
                        | r0 ← 13  if BeamType = "FIB63"
                        | r0 ← 14  if BeamType = "FIB72"
                        | r0 ← 15  if BeamType = "FIB78"
                        | r0 ← 16  if BeamType = "FIB84"
                        | r0 ← 17  if BeamType = "FIB96"
                        | r0 ← 18  if BeamType = "FIB120"
                        | r0 ← 30  if substr(BeamType, 0, 3) = "FUB"
```



```
                        | r0 ← 50  if substr(BeamType, 0, 3) = "FUB"
                        | r0 ← 50  if BeamType = "FDT18"
                        | r0 ← 51  if BeamType = "FDT24"
                        | r0 ← 52  if BeamType = "FDT30"
                        | r0 ← 60  if (FSBBeamName = "FSB12x48")·(BeamType = "FSB12")
                        | r0 ← 61  if (FSBBeamName = "FSB15x48")·(BeamType = "FSB15")
                        | r0 ← 62  if (FSBBeamName = "FSB18x48")·(BeamType = "FSB18")
                        | r0 ← 63  if (FSBBeamName = "FSB12x49")·(BeamType = "FSB12")
                        | r0 ← 64  if (FSBBeamName = "FSB15x49")·(BeamType = "FSB15")
                        | r0 ← 65  if (FSBBeamName = "FSB18x49")·(BeamType = "FSB18")
                        | r0 ← 66  if (FSBBeamName = "FSB12x50")·(BeamType = "FSB12")
                        | r0 ← 67  if (FSBBeamName = "FSB15x50")·(BeamType = "FSB15")
                        | r0 ← 68  if (FSBBeamName = "FSB18x50")·(BeamType = "FSB18")
                        | r0 ← 69  if (FSBBeamName = "FSB12x51")·(BeamType = "FSB12")
                        | r0 ← 70  if (FSBBeamName = "FSB15x51")·(BeamType = "FSB15")
                        | r0 ← 71  if (FSBBeamName = "FSB18x51")·(BeamType = "FSB18")
                        | r0 ← 72  if (FSBBeamName = "FSB12x52")·(BeamType = "FSB12")
                        | r0 ← 73  if (FSBBeamName = "FSB15x52")·(BeamType = "FSB15")
                        | r0 ← 74  if (FSBBeamName = "FSB18x52")·(BeamType = "FSB18")
                        | r0 ← 75  if (FSBBeamName = "FSB12x53")·(BeamType = "FSB12")
                        | r0 ← 76  if (FSBBeamName = "FSB15x53")·(BeamType = "FSB15")
                        | r0 ← 77  if (FSBBeamName = "FSB18x53")·(BeamType = "FSB18")
                        | r0 ← 78  if (FSBBeamName = "FSB12x54")·(BeamType = "FSB12")
                        | r0 ← 79  if (FSBBeamName = "FSB15x54")·(BeamType = "FSB15")
                        | r0 ← 80  if (FSBBeamName = "FSB18x54")·(BeamType = "FSB18")
                        | r0 ← 81  if (FSBBeamName = "FSB12x55")·(BeamType = "FSB12")
                        | r0 ← 82  if (FSBBeamName = "FSB15x55")·(BeamType = "FSB15")
                        | r0 ← 83  if (FSBBeamName = "FSB18x55")·(BeamType = "FSB18")
                        | r0 ← 84  if (FSBBeamName = "FSB12x56")·(BeamType = "FSB12")
                        | r0 ← 85  if (FSBBeamName = "FSB15x56")·(BeamType = "FSB15")
                        | r0 ← 86  if (FSBBeamName = "FSB18x56")·(BeamType = "FSB18")
                        | r0 ← 87  if (FSBBeamName = "FSB12x57")·(BeamType = "FSB12")
                        | r0 ← 88  if (FSBBeamName = "FSB15x57")·(BeamType = "FSB15")
```

$r_0 \leftarrow 89$ if $(FSBBeamName = "FSB18x57") \cdot (BeamType = "FSB18")$

$r_0 \leftarrow 90$ if $(FSBBeamName = "FSB12x58") \cdot (BeamType = "FSB12")$

$r_0 \leftarrow 91$ if $(FSBBeamName = "FSB15x58") \cdot (BeamType = "FSB15")$

$r_0 \leftarrow 92$ if $(FSBBeamName = "FSB18x58") \cdot (BeamType = "FSB18")$

$r_0 \leftarrow 93$ if $(FSBBeamName = "FSB12x59") \cdot (BeamType = "FSB12")$

$r_0 \leftarrow 94$ if $(FSBBeamName = "FSB15x59") \cdot (BeamType = "FSB15")$

$r_0 \leftarrow 95$ if $(FSBBeamName = "FSB18x59") \cdot (BeamType = "FSB18")$

$r_0 \leftarrow 96$ if $(FSBBeamName = "FSB12x60") \cdot (BeamType = "FSB12")$

$r_0 \leftarrow 97$ if $(FSBBeamName = "FSB15x60") \cdot (BeamType = "FSB15")$

$r_0 \leftarrow 98$ if $(FSBBeamName = "FSB18x60") \cdot (BeamType = "FSB18")$

$r_0 \leftarrow 99$ if $substr(BeamType, 0, 3) = "Inv"$

$r_0 \leftarrow 99$ if $BeamType = "Custom"$

$r_0 \leftarrow 99$ otherwise

$r_1 \leftarrow "e"$

$r_2 \leftarrow \frac{h_{beam}}{ft}$

r

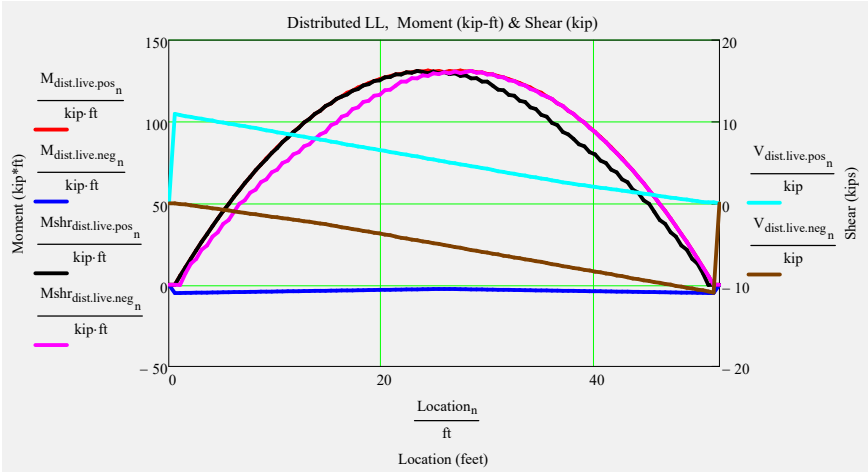
$BeamType = "FSB15"$ $FSBBeamName = "FSB15x54"$ $h_{beam} = 15 \cdot in$

Live Load Moments and Shears

Distributed Live Load Moments and Shears

$L_{span} = 50.75\ ft$

Live Load Distribution Factors $g_{mom} = 0.4364$ $g_{mom.fatigue} = 0.4364$ $g_{shear} = 0.4364$



$\max(V_{dist.live.pos}) = 11.0 \cdot kip$ $\max(M_{dist.live.pos}) = 131.4 \cdot kip \cdot ft$ $\min(V_{dist.live.neg}) = -11.0 \cdot kip$

$Reaction_{LL} = 11.1 \cdot kip$ $\min(M_{dist.live.neg}) = -5.0 \cdot kip \cdot ft$

$Reaction_{DL} = 31.9 \cdot kip$ $\max(Mshr_{dist.live.pos}) = 131 \cdot kip \cdot ft$

Beam End Reactions...
with IM factor only

Reinforcement Properties

InputDataFile = "Able Canal - Spans 2-4 - Exterior - H10 Load.dat"

Mild Reinforcement Type:

- Carbon Steel - Grade 60
- Stainless Steel - Grade 60
- Glass Fiber Reinforced Polymer
- Carbon Steel - Grade 80
- Stainless Steel - Grade 75

Help on Mild Reinforcement Types

Longitudinal Mild and Partial PS Reinforcement:

Excel Table of Standard FDOT Prestressed Beam Mild and Partial PS Longitudinal Reinforcement

Help on Longitudinal Mild and Partial PS Reinf

Mild Reinforcement

	Deck	Bottom of Beam	Beam End: Top	Beam End: Bottom
Location				
Area (in^2)	0.4108 *1	0	0	0
Distance (in)	2.3125 *2	0 *3		
Length (ft)			0	0
Bar Size		5 *4	0 *4	0 *4

Partial PS (Dormant) Strand

Location	Top of Beam
Diameter(in)	3/8 inch / 1/2 inch
Distance (in)	2
# Strands	4
Force per Strand (kip)	10

**1 - Area of longitudinal deck reinf. per unit width of deck, both layers combined. Typically one #5 Bar top & bottom for 8" decks.*
**2 - Distance measured from top of deck to centroid of deck longitudinal reinforcement, positive value.*
**3 - Distance measured from top of beam, positive value.*
**4 - Size of bars used to create $A_{s.long}$ needed to calculate development length.*

Prestressing Tendons:

Humidity	75 %	*% relative humidity (75% typical)*
Time: jacking to transfer	0.75 days	*Time in days between jacking and transfer.*

Prestress Strand Type

- Carbon Steel - Low Lax
- Carbon Steel - Stress Relieved
- Stainless Steel
- Carbon Fiber Reinforced Polymer

Help on Prestress Reinforcement Typ

Prestress Strand Size

- 0.6 in.
- 0.5 in.
- 0.5 in. Special
- 9/16 in.
- 0.7 in.

Help on Strand Generator | Help on Strand Debonding

Strand Pattern Generator for Entering Prestressing Strand Layout

Double click the icon to open the 'Strand Pattern Generator'. Specify the location and debonding of strands. When finished, press the 'Continue' button. Then press 'Read Strand Data' button below.

Press 'Read Strand Data' Button.to Read in Strand Generator Data

Read Strand Data

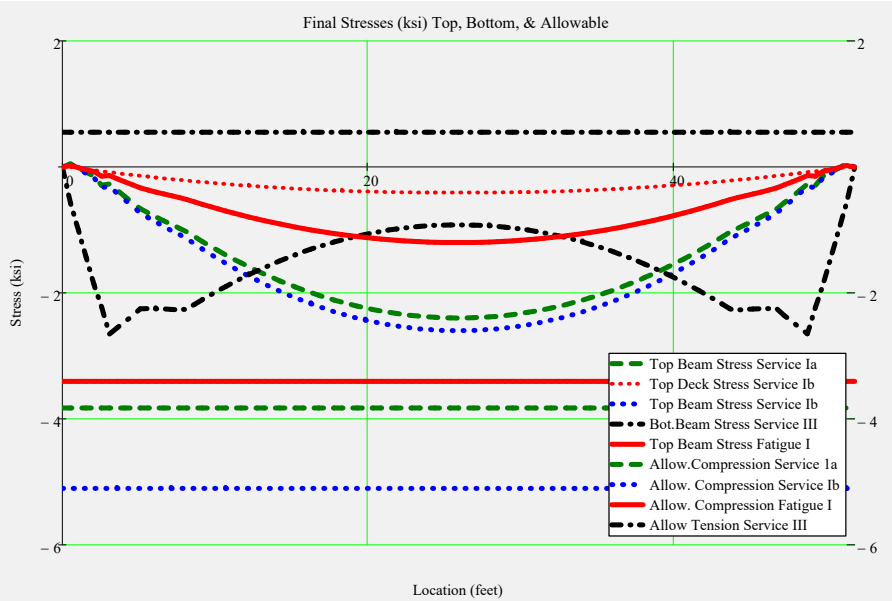
Press 'Read Strand Data' button to Read in Strand Generator Data.

Material Properties
Strand Data, Pattern, and Properties
Section Properties, Strand Profile Properties, and Service I and III Beam Stresses and Limits

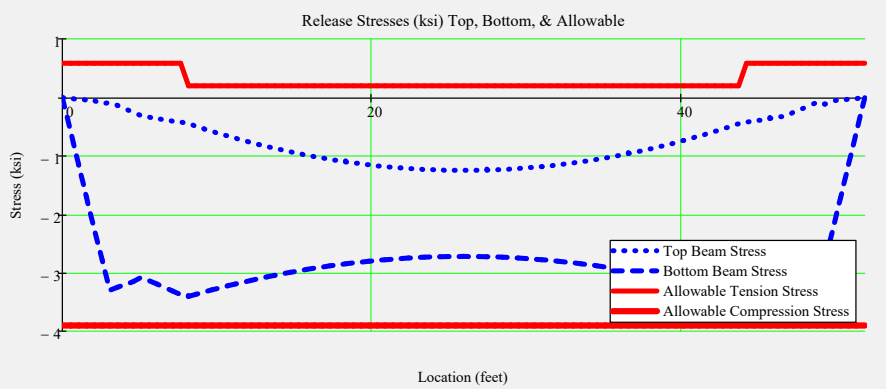
Service I and III Stresses and Limits and Tendon Layout Summary

tension (+), compression (--)

Final Stresses



Release Stresses



Stress Checks at Service Limit States

Release tension	$Check_f_{tension.rel}$ = "OK"	$\max(DC_f_{tension.rel}) = 0.68$
Release compression	$Check_f_{comp.rel}$ = "OK"	$\max(DC_f_{comp.rel}) = 0.87$
*Service III , PS+DL+0.8*LL*	$Check_f_{tension.ServiceIII}$ = "OK"	$\max(DC_f_{tension.ServiceIII}) = 0.83$
Service Ia , PS+DL	$Check_f_{comp.ServiceIa}$ = "OK"	$\max(DC_f_{comp.ServiceIa}) = 0.63$
Service Ib , PS+DL+LL	$Check_f_{comp.ServiceIb}$ = "OK"	$\max(DC_f_{comp.ServiceIb}) = 0.51$
*Fatigue I , (PS + DL)*0.5 +1.5 Fatigue Truck*	$Check_f_{comp.FatigueI}$ = "OK"	$\max(DC_f_{comp.FatigueI}) = 0.35$

StressChecks =

"StressCheck"	"Location(ft)"	"Stress(ksi)"	"AllowableStress(ksi)"
"Tension @ Release"	0.00	0.00	0.59
"Compression @ Release"	8.15	−3.40	−3.90
"Tension @ ServiceIII"	0.00	0.00	0.55
"CompressionIa @ ServiceI"	25.92	−2.40	−3.83
"CompressionIb @ ServiceI"	26.42	−2.59	−5.10
"Compression @ FatigueI"	25.92	−1.20	−3.40

CheckBeamEndTopLongReinf = "OK"

Strand Pattern Checks

1. *The number of strands debonded per row shall not exceed 45 percent of the strands provided in that row.*
2. *Debonding shall not be terminated for more than six strands in any given section. When a total of ten or fewer strands are debonded, debonding shall not be terminated for more than four strands in any given section*
3. *Longitudinal spacing of debonding termination locations shall be at least 60db apart.*
4. *Alternate bonded and debonded strand locations both horizontally and vertically.*
5. *For simple span precast, pretensioned girders, debonding length from the beam end should be limited to 20 percent of the span length or one half the span length minus the development length, whichever is less.*

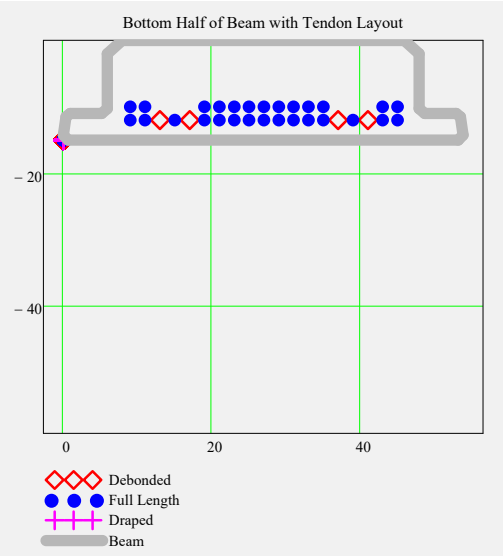
CheckPattern = ("OK", "OK", "OK", "OK", "OK")

For single-web flanged sections (I-beams, bulbtees, and inverted-tees):
1. *Bond all strands within the horizontal limits of the web*
2. *Bond the outer-most strands in all rows located within the full-width section of the flange.*
3. *Position debonded strands furthest from the vertical centerline.*

For multi-web sections having bottom flanges (voided slab, box beams and U-beams):
1. *Strands shall be bonded within 1.0 times the web width projection.*
2. *Bond the outer-most strands within the section.*
3. *Uniformly distribute debonded strands between webs.*

BeamType = "FSB15" CheckWebPattern = ("OK", "OK")

Tendon Layout



BeamType = "FSB15"

$h_{beam} = 15 \cdot in$

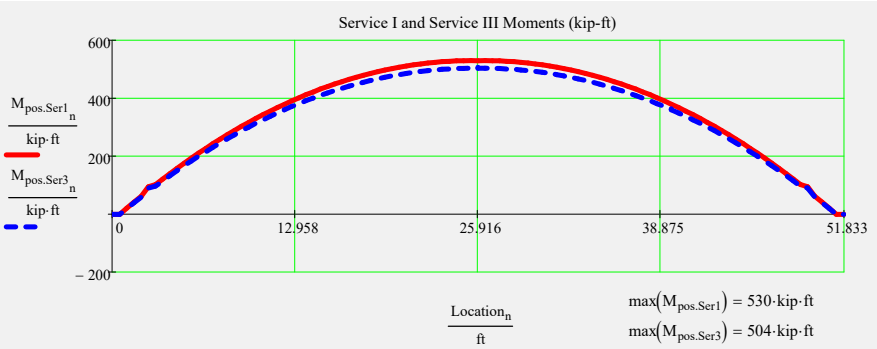
$A_{strand} = 0.217 \cdot in^2$

#Strands = 32

Prestress Losses Summary

$f_{pj} = 202.5 \cdot ksi$	$Check_f_{pt}$ = "OK"
$\Delta f_{pES} = 0 \cdot ksi$	*Note: Elastic shortening losses are zero in concrete stress calculations when using transformed section properties per* *LRFD 5.9.3.2.3*
$\Delta f_{pT} = -25 \cdot ksi$	$\frac{\Delta f_{pT}}{f_{pj}} = -12.22 \cdot \%$
$f_{pe} = 178 \cdot ksi$	$\frac{f_{pe}}{f_{pj}} = 87.78 \cdot \%$
$0.8 \cdot f_{py} = 194 \cdot ksi$	$Check_f_{pe}$ = "OK"

Moments at Service Limit States



Summary of Values at Midspan

Stresses =

Stage	Top of Beam (ksi)	Bottom of Beam (ksi)
"1"	–1.25	–2.72
"2"	–1.30	–2.21
"4"	–1.23	–2.27
"6"	–2.37	–1.29
"8"	–2.59	–0.92

PrestressForce =

Time	Axial (kip)	Moment (kip*ft)
"@Release"	–1446	–368
"@Final (about composite centroid)"	–1269	–305

Properties =

Section	Area (in^2)	Inertia (in^4)	Top of Beam to Centroid (in)
"Net Beam"	664.6	12861	–7.9
"Transformed Beam (initial)"	709.3	13394	–8.1
"Transformed Beam (final)"	704.4	13339	–8.1
"Composite Section"	1086.1	39332	–5

ServiceMoments =

Load Type	Moment (kip*ft)
"Release Dead Load"	235
"Non-Composite DL (incl. beam wt.)"	382
"Composite Dead Load"	17
"Distributed Live Load"	130

Stage 1 ---> At release with span length equal to length of the beam. Prestress losses are elastic shortening and overnight relax

Stage 2 ---> At final, starting with Stage 1 and adding the remaining prestress losses applied to the transformed beam

Stage 4 ---> Starting with Stage 2 and changing the support locations from the end of the beam to the bearing locations

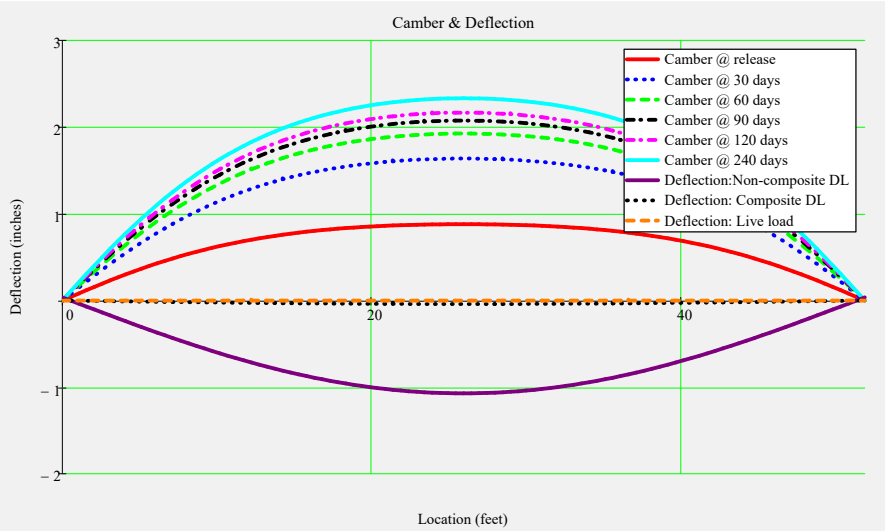
Stage 6 ---> Starting with Stage 4 and adding non-composite dead load (excluding beam wt included in Stage 1)

Stage 8 ---> Starting with Stage 6 and adding composite dead load and distributed live loads applied to the composite section

Camber at Release, Short Term Camber, & Summary of Slope Data

Camber at Different Stages and Deflection due to Dead Load and Live Load

$L_{span} = 50.75\,ft$



Slope&Defl =

Stage	ΔL @ Beam Top (in)	ΔL @ Beam Bot. (in)	Slope at End (deg)	Midspan Defl (in)
"Release"	–0.1000	–0.3936	0.3771	0.8815
"30 Days"	–0.2547	–0.7282	0.6822	1.6389
"60 Days"	–0.3126	–0.8547	0.7986	1.9277
"90 Days"	–0.3426	–0.9202	0.8588	2.0772
"120 Days"	–0.3610	–0.9603	0.8956	2.1685
"240 Days"	–0.3944	–1.0331	0.9624	2.3341
"Non-comp DL"	–0.0908	0.0783	–0.3230	–1.0705
"Comp DL"	–0.0021	0.0042	–0.0119	–0.0396
"LL"	0.0000	0.0000	0.0000	0.0000

Check net camber at end of construction (120 day) *SDG 4.3.2.D & 4.4.3.A*

$NetCamber := \max(Defl_{120day}) + \min(Defl_{dl.nc}) + \min(Defl_{dl.c}) = 1.06 \cdot in$

$CheckNetCamber := if(NetCamber > 0, "OK", "No Good")$ CheckNetCamber = "OK"

Moment Resistance

Moments and Moment Resistance at StrengthI and StrengthII Limit State

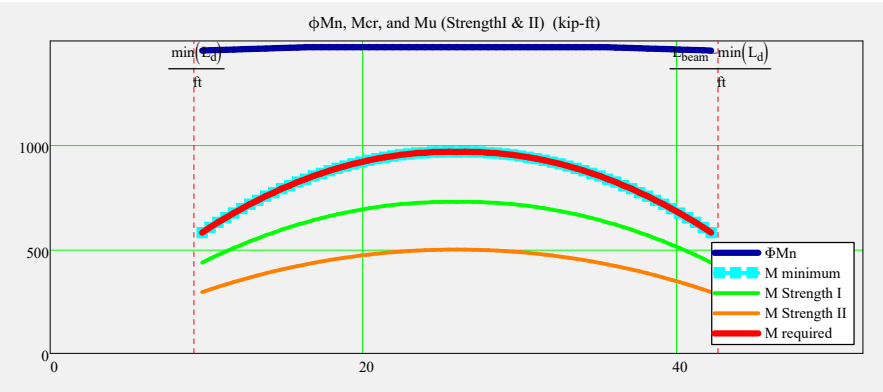
Values at midspan

$LimitStateForMoment_{51}$ = "Strength I" $\max(M_{Strength}) = 729 \cdot kip \cdot ft$

$IsSectionNonCompControlled?_{51}$ = "Yes" $M_{minimum_{51}} = 968 \cdot kip \cdot ft$ $M_{reqd_{51}} = 968 \cdot kip \cdot ft$

StrandType = "Carbon Steel - Low Lax" $\phi_{mom_{51}} = 0.84$ $\phi_{mom_{51}} \cdot (M_{n_{51}}) = 1471 \cdot kip \cdot ft$

$FailureMode_{51}$ = "N.A." CheckMinSSLimit = "N.A."



$\min(L_d) = 9.2\,ft$ $L_{beam} - \min(L_d) = 42.6\,ft$

$DC_{moment_{mn}} := \dfrac{M_{reqd_{mn}}}{\phi_{mom_{mn}} \cdot (M_{n_{mn}})}$ *Demand/Capacity Ratio at locations where all strands are fully developed* $\max(DC_{moment}) = 0.66$

CheckMomentCapacity := $\text{if}(\max(DC_{moment}) < 1.0, \text{"OK"}, \text{"No Good!"}) = \text{"OK"}$

FSB only - Design Check of Transverse Reinforcing Bars E

FSB only - Design Check of Bottom Transverse Reinforcing Bars E

Spacing of Bars E is checked per Control of Cracking by Distribution of Reinforcement [LRFD 5.6.7 or AASHTO GFRP 2.6.7].
The Service I moment in transverse direction is calculated based on the r ratio to the longitudinal moment per LRFD 9.7.3.2.

$s \le \dfrac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$ *The maximum spacing of the mild steel reinforcement for control of cracking at the service limit state shall satisfy* *[LRFD 5.6.7]*

$\gamma_e := 1.00$

$s \le \min\left(1.15 \cdot \dfrac{C_b \cdot E_f \cdot w}{f_{fs}} - 2.5c_c, 0.92 \cdot \dfrac{C_b \cdot E_f \cdot w}{f_{fs}}\right)$ *[AASHTO GFRP 2.6.7-1]*

$d_c \le \dfrac{C_b E_f w}{2 f_{fs} \xi}$ (2.6.7-2) *[AASHTO GFRP Eq. 2.6.7-2]*

$C_b := 0.83$ *GFRP bars bond coefficient*

$w_{max} := 0.028in$ *limiting crack width*

$A_{BarsE} := 0.31in^2$ *Area of Bars E*

$s_{BarsE} := 6in$ *Typical Spacing of Bars E*

$s_{BarsE.NearEnds} := 12in$ *Spacing of Bars E near ends of span*

$n_{BarsE.NearEnds} := 6$ *Number of spaces of Bars E near ends of span*

BarsESize := 5 *Size of Bars E*

$c_c := 1.5in$ *clear cover*

Area and spacing of Bars E per location (up to half span)

$hs := 0 .. \dfrac{last(Location)}{2} - 1$

$A_{BarsE.prov_{hs}} := \begin{cases} \begin{cases} \dfrac{A_{BarsE}}{s_{BarsE.NearEnds}} & \text{if } Location_{hs} \le n_{BarsE.NearEnds} \cdot s_{BarsE.NearEnds} \\ \dfrac{A_{BarsE}}{s_{BarsE}} & \text{otherwise} \end{cases} & \text{if substr(BeamType, 0, 3) = "FSB"} \\ 0 & \text{otherwise} \end{cases}$

$s_{BarsE.prov_{hs}} := \begin{cases} \begin{cases} s_{BarsE.NearEnds} & \text{if } Location_{hs} \le n_{BarsE.NearEnds} \cdot s_{BarsE.NearEnds} \\ s_{BarsE} & \text{otherwise} \end{cases} & \text{if substr(BeamType, 0, 3) = "FSB"} \\ 0 & \text{otherwise} \end{cases}$

$ratio := \dfrac{100}{\sqrt{\dfrac{L_{span}}{ft}}} \cdot \dfrac{1}{100}$ *LRFD 9.7.3.2*

$ratio := \text{if}(ratio \le 0.5, ratio, 0.5)$ $ratio = 0.14$

$M_{transverse.pos.Ser1} := ratio \cdot \frac{M_{dist.live.pos} + M_{dl.comp}}{b_e} \cdot 1ft$ $\quad \max(M_{transverse.pos.Ser1}) = 4.63 \cdot kip \cdot ft$ *per foot width*

Note: there is no stress induced in Bars E from the self-weight of the FSB's or the CIP deck overlay.

$d_c := c_c + 0.5 \cdot \frac{BarsESize}{8} \cdot in = 1.81 \cdot in$ *Distance from extreme tension fiber to center of closest bar*

$h := t_{deck} + Height_{pocket} = 17 \cdot in$ *Overall thickness or depth of the component*

$d_s := h - d_c = 15.19 \cdot in$

$\beta_s := 1 + \frac{d_c}{0.7(h - d_c)} = 1.17$

$E_{c.deck} := 120000 \cdot K_1 \cdot \left(w_{c.deck} \cdot \frac{ft^3}{kip}\right)^2 \cdot \left(\frac{f_{c.deck}}{ksi}\right)^{0.33} \cdot ksi = 4145 \cdot ksi$

$n_s := \frac{E_s}{E_{c.deck}} = 7$ *Modular ratio*

$\rho_{hs} := \frac{A_{BarsE.prov_{hs}}}{d_s}$ *Reinforcement ratio*

$kk_{hs} := \sqrt{2 \cdot \rho_{hs} \cdot n_s + (\rho_{hs} \cdot n_s)^2} - \rho_{hs} \cdot n_s$

$j_{hs} := 1 - \frac{kk_{hs}}{3}$

$f_{r.pocket} := 0.24 \cdot \sqrt{f_{c.deck} \cdot ksi}$ $\quad f_{r.pocket} = 0.51 \cdot ksi$

$S_{pocket} := \frac{1}{6} \cdot 12 \cdot in \cdot h^2$ $\quad S_{pocket} = 578 \cdot in^3$

$\gamma_1 := 1.6$ *Flexural cracking variability factor*

$\gamma_3 := 0.67$ *Ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement (0.67 for ASTM A615, Grade 60 reinforcing steel per SDG 1.4.1)*

$M_{cr.pocket} := \gamma_1 \cdot \gamma_3 \cdot f_{r.pocket} \cdot S_{pocket}$ $\quad M_{cr.pocket} = 26.29 \cdot kip \cdot ft$ *per feet width*

$$f_{ss.Ser1_{hs}} := \begin{cases} \text{if substr(BeamType, 0, 3) = "FSB"} \\ \quad \begin{cases} 0 & \text{if } M_{transverse.pos.Ser1_{hs}} \le 0 \\ \frac{M_{transverse.pos.Ser1_{hs}}}{A_{BarsE.prov_{hs}} \cdot 1ft \cdot j_{hs} \cdot d_s} & \text{otherwise} \end{cases} \\ 0 \quad \text{otherwise} \end{cases}$$

Stress in Bars E (if moment is negative, stress in bottom steel is set to 0)

Required reinforcement spacing

$$Spa_{BarsE.reqd_{hs}} := \begin{cases} \text{if substr(BeamType, 0, 3) = "FSB"} \\ \quad \begin{cases} 10000in & \text{if } f_{ss.Ser1_{hs}} \le 0 \\ \text{otherwise} \\ \quad \begin{cases} \left(\frac{700 \cdot \gamma_e \cdot \frac{kip}{in}}{\beta_s \cdot f_{ss.Ser1_{hs}}} - 2 \cdot d_c\right) & \text{if (MildReinfType = "Carbon Steel") + (MildReinfType = "Stainless Steel")} \\ \min\left(1.15 \cdot \frac{C_b \cdot E_f \cdot w_{max}}{f_{ss.Ser1_{hs}}} - 2.5 \cdot c_c, 0.92 \cdot \frac{C_b \cdot E_f \cdot w_{max}}{f_{ss.Ser1_{hs}}}\right) & \text{if MildReinfType = "Glass Fiber Reinforced Polymer"} \end{cases} \end{cases} \\ 0 \quad \text{otherwise} \end{cases}$$

$CriticalSpa_{BarsE.reqd} := \min(Spa_{BarsE.reqd}) = 91.09 \cdot in$ *The most critical spacing along cap sections*

$DC_{BarsE_{hs}} := \frac{s_{BarsE.prov_{hs}}}{Spa_{BarsE.reqd_{hs}}}$ $\quad \max(DC_{BarsE}) = 0.1$

$$d_{c.max_{hs}} := \begin{cases} \text{if substr(BeamType, 0, 3) = "FSB"} \\ \quad \begin{cases} dcmax \leftarrow 10000in & \text{if } f_{ss.Ser1_{hs}} \le 0 \\ \text{otherwise} \\ \quad \begin{cases} \zeta_{hs} \leftarrow \frac{h - kk_{hs} \cdot d_s}{d_s - kk_{hs} \cdot d_s} \\ dcmax \leftarrow \frac{C_b \cdot E_f \cdot w_{max}}{2 \cdot f_{ss.Ser1_{hs}} \cdot \zeta_{hs}} \end{cases} \end{cases} \\ dcmax \leftarrow 0 \quad \text{otherwise} \\ dcmax \end{cases}$$

limiting d.c value

[AASHTO GFRP Eq. 2.6.7-2]

$\min(d_{c.max}) = 10.42 \cdot in$

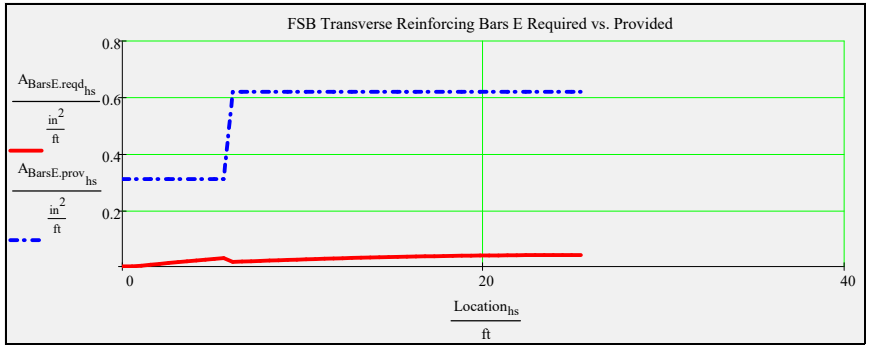
$d_c = 1.81 \cdot in$

$f_{s.max} := \text{if}(f_y < 75ksi, 0.8 \cdot f_y, 0.6 \cdot f_y) = 48 \cdot ksi$ *[SDG 4.1.8]*

$$Check_{FSB.BarsE} := \begin{cases} \text{if substr(BeamType, 0, 3) = "FSB"} \\ \quad \begin{cases} \text{"OK"} & \text{if } \max(DC_{BarsE}) \le 1 \wedge \max(f_{ss.Ser1}) \le f_{s.max} \wedge \text{(MildReinfType = "Carbon Steel") + (MildReinfType} \\ \text{"OK"} & \text{if } \max(DC_{BarsE}) \le 1 \wedge d_c \le \min(d_{c.max}) \wedge \text{MildReinfType = "Glass Fiber Reinforced Polymer"} \\ \text{"No Good"} & \text{otherwise} \end{cases} \\ \text{"N/A"} \quad \text{otherwise} \end{cases}$$

$Check_{FSB.BarsE} = \text{"OK"}$

$$A_{BarsE.reqd_{hs}} := \text{if}\left(\text{substr}(BeamType,0,3) = \text{"FSB"}, \frac{A_{BarsE}}{Spa_{BarsE.reqd_{hs}}}, 0\right)$$



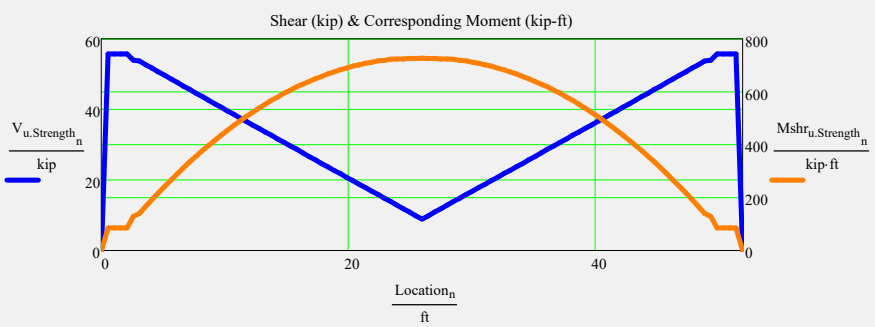
FSB only - Design Check of Transverse Reinforcing Bars E

Shear Loads

Shear and Corresponding Moments:

Calculations use the greater of Strength I or Strength II Limit State

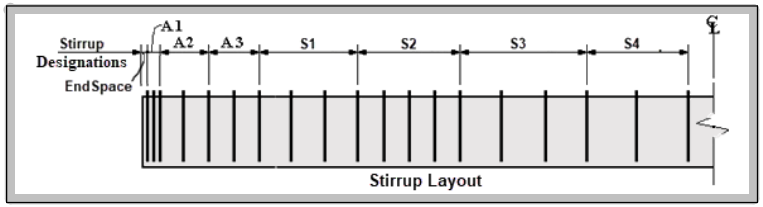
$$\begin{pmatrix} \max(V_{pos.StrengthI}) \\ \max(V_{pos.StrengthII}) \end{pmatrix} = \begin{pmatrix} 58 \\ 39 \end{pmatrix} \cdot kip \quad \begin{pmatrix} \text{"HL93"} \\ \text{"Permit Vehicle"} \end{pmatrix}$$



$\max(V_{u.Strength}) = 55.8 \cdot kip$ ControllingShearLimitState = "Strength I" $\max(Mshr_{u.Strength}) = 727 \cdot kip \cdot ft$

Shear Resistance

Stirrup Size and Spacings (values are for one half of the beam)



End Stirrup Space	2.5	in	*Distance from end of beam to the centerline of first stirrup or stirrup bundle.*
GFRP Stirrup Bar #	0	*used for GFRP Bars*	

Help on Stirrup Design | Help on Stirrup Design Iter. | Table of FDOT Standard Prestressed Beam End Reinforcement | Reset Load Data

Stirrup Design - see Stirrup Layout

Designation	A1	A2	A3	S1	S2	S3	S4
Spacing (in)	3	7	0	12	24	0	24
# of Spaces	3	1	0	4	10	0	Calculated
Area (in^2)	0.60	0.8	0.0	1.00	1.20	0.0	1.20
Interface Factor	0.33	1.0	0.0	0.80	0.67	0.0	0.67

Save and Initialize Data

Shear Demand/Capacity and Longitudinal Steel Demand/Capacity

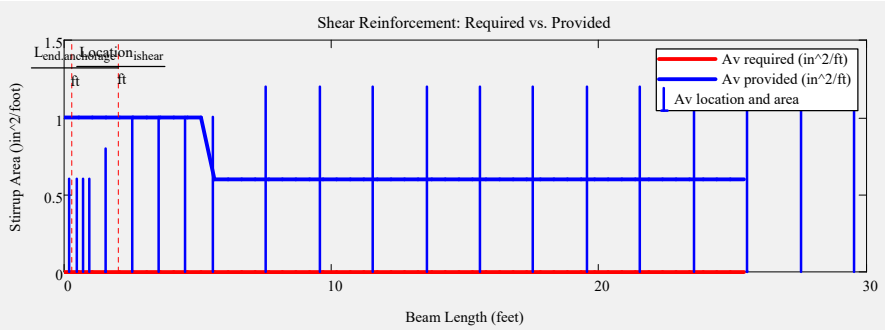
CheckShearCapacity = "OK"

CheckStirArea = "OK"

CheckMaxShearStress = "OK"

CheckMinStirArea = "OK"

CheckMaxStirSpacing = "OK"



$Location_{ishear} = 2.06\ ft$

Save Data File

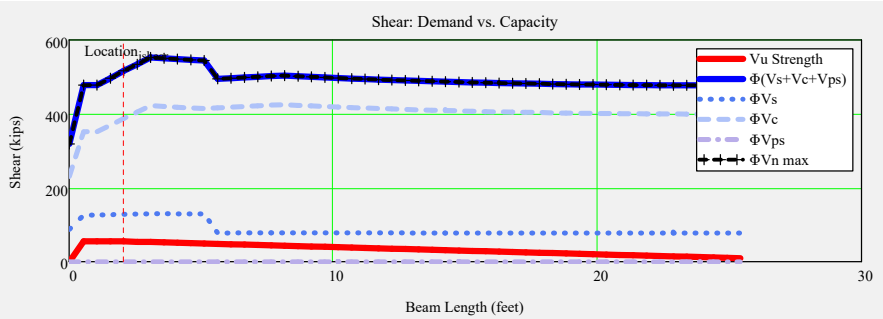
Use current input file

File Name Able Canal - Spans 2-4 - Exterior - H10 Load.dat

Save Data

DataFileFolder = "C:\Users\jjohnson2\Desktop\Able Canal\Superstructure\PrestressedBeamV6.1\DataFiles-FSB"

Note: Select an output folder by using the "Change Folder" option on page 1.



$fCheckD/CShear = \begin{pmatrix} \text{"CheckType"} & \text{"Location(ft)"} & \text{"Vu (kip)"} & \text{"Vr (kip)"} \\ \text{"Shear"} & 2.06 & 55.79 & 516.84 \end{pmatrix}$

$\max(D/C_{shear}) = 0.11$

Botom Longitudinal Reinf. (LRFD 5.7.3.5)

Longitutinal Reinforcement:

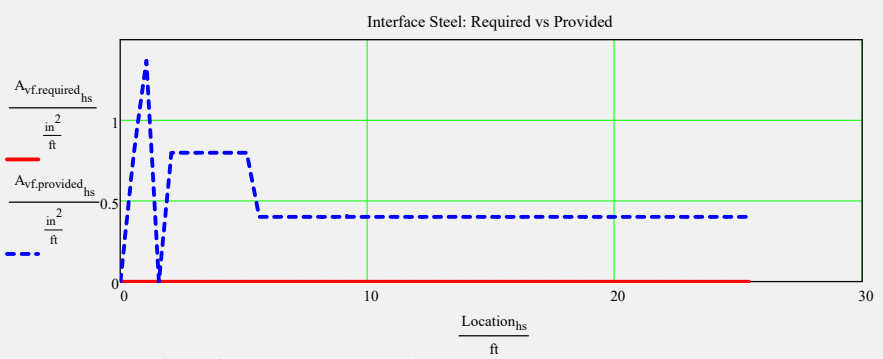
$CheckD/CLong = \begin{pmatrix} \text{"CheckType"} & \text{"Location(ft)"} & \text{"V long Req'd (kip)"} & \text{"V long Prov (kip)"} \\ \text{"Longitudinal Reinf."} & 23.4 & 682.1 & 1620.4 \end{pmatrix}$

$\max(D/C_{LongReinf}) = 0.42$ CheckLongReinf = "OK"

(LRFD 6.7.3.5)

Interface Shear Reinforcement

Interface Shear:



$TotalA_{vf.required} = 0 \cdot in^2$ $TotalA_{vf.provided} = 12.01 \cdot in^2$ $DC_{InterfaceReinf} = 0$ CheckInterfaceReinf = "OK"

Typically shear reinforcement is extended up into the deck slab.
These calculations are based on shear reinforcement functioning as interface reinforcing.
The interface_factor *can be used to adjust this assumption.*

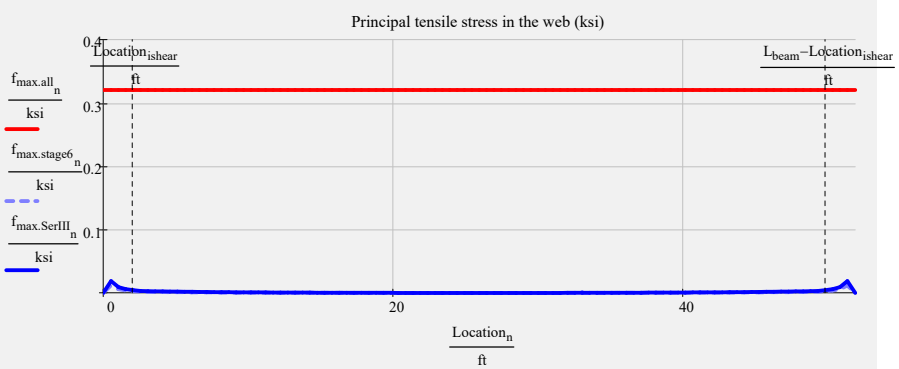
$\max(A_{vf.min.required}) = 0.000 \cdot \frac{in^2}{ft}$ $\max(A_{vf.des}) = 0.000 \cdot \frac{in^2}{ft}$

If max(Avf.min) or max(Avf.des) is greater than 0 in²/ft, interface reinforcement is required.

CheckInterfaceSpacing = "OK"

Pricipal Tensile Stresses in Webs - LRFD 5.9.2.3.3 and Splitting Resistance

Principal Stresses:



Crital Location for Principal Stress: $Location_{ishear} = 24.8 \cdot in$ $\max(DC_f_{max.web}) = 0.1$ $Check_f_{max.web} = "OK"$

Note: The check for pricipal tensile stresses in webs is only required for pretensioned girders with a compressive strength of concrete for use in design greater than 10 ksi per AASHTO BDS Article 5.9.2.3.3. The check is displayed for all concrete strengths for consideration when optimizing the strands design.

Splitting Reinforcement:

FDOT Splitting Rein Demand/Capacity Ratio and Check for FDOT Standard Beam End Reinforcement

$P_{r.standard} = 1460 \cdot kip$ $P_{prestress} = 1230 \cdot kip$ $D/C_{splitting.FDOT} = 0.84$

$CheckD/C_{splitting} = "OK"$

Confinement Bars:

$TotalNoConfineBars = 10$

Summary of Design Checks

$check_1 := "OK"$

$check_2 := CheckDeflectionOrSpanToDepth = "OK"$

$check_3 := CheckStrandFit = "OK"$

$check_4 := Check_f_{pt} = "OK"$

$check_5 := Check_f_{pe} = "OK"$

$check_6 := Check_f_{tension.rel} = "OK"$

$check_7 := Check_f_{comp.rel} = "OK"$

$check_8 := Check_f_{tension.ServiceIII} = "OK"$

$check_9 := Check_f_{comp.ServiceIa} = "OK"$

$check_{10} := Check_f_{comp.ServiceIb} = "OK"$

$check_{11} := Check_f_{comp.FatigueI} = "OK"$

$check_{12} := CheckMomentCapacity = "OK"$

$check_{13} := CheckMaxCapacity = "OK"$

$check_{14} := CheckStirArea = "OK"$

$check_{15} := CheckShearCapacity = "OK"$

$check_{16} := CheckMinStirArea = "OK"$

$check_{17} := CheckMaxStirSpacing = "OK"$

$check_{18} := CheckLongReinf = "OK"$

$check_{19} := CheckInterfaceSpacing = "OK"$

$check_{20} := CheckInterfaceReinf = "OK"$

$check_{21} := "OK"$

$check_{22} := CheckD/C_{splitting} = "OK"$

$check_{23} := CheckPattern_0 = "OK"$

$check_{24} := CheckPattern_1 = "OK"$

$check_{25} := CheckPattern_2 = "OK"$

$check_{26} := CheckPattern_3 = "OK"$

$check_{27} := CheckPattern_4 = "OK"$

$check_{28} := CheckWebPattern_0 = "OK"$

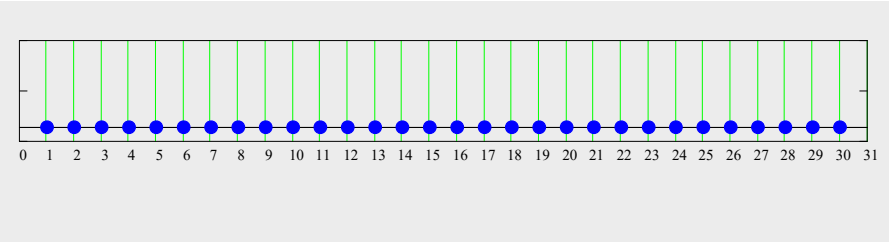
$check_{29} := CheckWebPattern_1 = "OK"$

$check_{30} := CheckBeamEndTopLongReinf = "OK"$

All Beam Checks

$check^T =$

	0	1	2	3	4	5	6	7	8	9	10
0	0	"OK"	"OK"	"OK"	"OK"	"OK"	"OK"	"OK"	"OK"	"OK"	...



$TotalCheck = "OK"$

Load Rating Analysis: Load and Resistance Factor Rating (LRFR)

ref.: Maintenance Office FDOT Bridge Load Rating Manual

Load Rating Computations

$$LRFR_{loadrating} = \begin{array}{c} \text{Moment (Strength) or Stress (Service)} \quad \text{Shear (Strength)} \\ \begin{pmatrix} 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 \\ 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 1.52 \\ 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 1.52 \\ 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 \\ 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 \\ 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 2.73 \times 10^{4} & 1.64 \times 10^{6} & 1.52 \end{pmatrix} \end{array} \quad \begin{pmatrix} \text{HL-93} \\ \text{HL-93} \\ \text{HL-93} \\ \text{HL-93*} \\ \text{Permit**} \end{pmatrix}$$

Notes:
** Not applicable for prestressed beams in good condition.*
*** FDOT Permit Rating truck is the FL120 .*
"Dim(ft)"*is measured from the centerline of bearings*

Longitudinal Reinforcement Check:

$DC_{LongReinf.HL93} = 0.42$ $DC_{LongReinf.Permit} = 0.28$ $CheckLongReinf_{loadrating} = \text{"OK"}$

QUICK CHECK: Factored Components of the Rating Factor Equation for the Design Operating Moment and Shear Ratings

$$HL93_{Operating.Moment} = \begin{pmatrix} \text{"}\varphi C\text{"} & 1471 & \text{"factored resistance (kip-ft)...."} \\ \text{"}\gamma DL\text{"} & 498 & \text{"factored dead load (kip-ft)...."} \\ \text{"}\gamma LL\text{"} & 177 & \text{"factored live load (kip-ft)....."} \\ \text{"RF"} & 5.49 & \text{"HL93 operating rating factor"} \end{pmatrix}$$

$$HL93_{Operating.Shear} = \begin{pmatrix} \text{"}\varphi C\text{"} & 516.8 & \text{"factored resistance (kip)....."} \\ \text{"}\gamma DL\text{"} & 37.3 & \text{"factored dead load (kip)....."} \\ \text{"}\gamma LL\text{"} & 14.3 & \text{"factored live load (kip)....."} \\ \text{"RF"} & 33.56 & \text{"HL93 operating rating factor"} \end{pmatrix}$$

FDOT LRFD Prestressed Beam Program v6.1

State Structures Design Office

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Select Beam Data Folder

- I-Beam
- FSB
- FUB
- FlatSlab
- InvertedT
- DoubleT

Category and Beams

I-Beams: *Florida I-Beams (FIBs) and formerly used Florida Bulb Tee and AASHTO Beams*
Slab Beams: *Florida Slab Beams (FSBs)*
U-Beams: *Florida U-Beams (FUBs)*
Flat Slab Previously used Florida Flat Slab Beams, superceded by FSBs
Inverted Tee: *Former FDOT Standard Inverted Tee Beams*
Double Tee: *Former FDOT Standard Double Tee Beams from the 1990's*

Data Files Folder

C:\Users\jjohnson2\Desktop\Able Canal\Superstructure\PrestressedBeamV6.1\DataFiles-FSB

Change Folder

Open Existing Data File (required). To save New Data Files, enter new data name at the end of Section IX.

- Able Canal - Span 1 _6 - Exterior.dat
- Able Canal - Spans 2 though 4 - Interior.dat
- Able Canal - Spans 2 through 4 - Exterior.dat
- Able Canal - Spans 2 through 4 - Interior.dat
- Able Canal - Spans 2-4 - Exterior - H10 Load.dat
- Able Canal - Spans 2-4 - Exterior - Ped. Load.dat
- Able Canal - Spans 2-4 - Interior - H10 Load.dat
- Able Canal - Spans 2-4 - Interior - Ped. Load.dat
- FSB-30ftSpan-12inx53inBeam.dat

Refresh List

Open File

Reset All Data

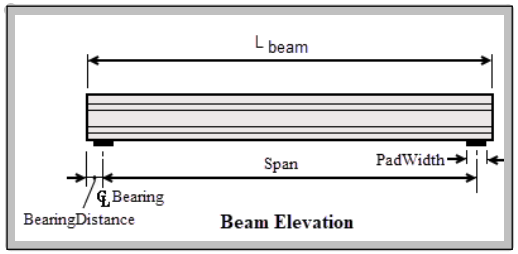
InputDataFile = "Able Canal - Spans 2-4 - Exterior - Ped. Load.dat"

Project Information

Project Information

Project Name	Able Canal Pathway - ~~Spans 2 through 4~~ - Exterior - Ped. Load		
Project No.	435351-2-38-01		
Designed by	JHJ	*Date*	02/2022
Checked by		*Date*	
Structure Name	53ft Span - 15"x54"		
Structure No.	XXXXXX		

Plan, Elevation, and Cross Section Data



Note: All dimensions shown in Beam Elevation measured along centerline of beam (requiring adjustment for a skew)

Note: The top of the precast beam is the location of the origin for the coordinate system.

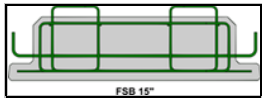
Superstructure Data

InputDataFile = "Able Canal - Spans 2-4 - Exterior - Ped. Load.dat"

Beam Type: FSB12, FSB15, FSB18 — *Beam Types are the designations found in FDOT standards. The user can also create a coordinate file for a custom shape. Top of the beam is at the y=0 ordinate.*

BMPfile := concat(substr(inpBeamType, 0, strlen(inpBeamType)), ".bmp")

BMPfile = "FSB15.bmp"



L_{beam}	51.833	ft	*See Beam Elevation above.*
Bearing Distance	6.5	in	*See Beam Elevation above.*
Pad Width	8	in	*Width of the bearing pad, see Beam Elevation above.*
Beam Spacing	4.583	ft	*Measured from beam centerline to centerline.*
Overhang	2.25	ft	*Measured from centerline of exterior beam.*
Deck Thickness	6	in	*Not including sacrificial thickness.*
$t_{sacrificial}$	0	in	*Sacrificial thickness cast with the deck (used for DL only, not section properties).* *(0" for* BridgeLengths ≤ 100 ft, *SDG 4.2.2)*
d_e	1.4167	ft	*Front face of barrier to centerline of exterior beam (3 ft max).* *(LRFD 4.6.2.2.1)*
Beam Position	Interior / Exterior		*Use either* "interior" *or* "exterior".
$Weight_{future.ws}$	0.00	kip/ft^2	*Future wearing surface.* (0.015ksf *for* BridgeLengths ≤ 100 ft, *SDG Table 2.1)*
$Weight_{barrier}$	0.080	kip/ft	*Weight of single barrier.* *(0.030 klf for Pedestrian Railing, SDG Table 2-2.1 & 0.050 klf for 8" x 6" Concrete Curb)*
Number of Barriers	2		*Number of barriers in x-section (multiplies single barrier weight).*
Number of Beams	3		*Number of beams in the span cross section.* *(LRFD 4.6.2.2.1)*
Skew	0	deg	*measured from the perpendicular to the longitudinal axis in bridge plan view.*

Dimensions Specific to Certain Beam Categories

I-Beams, U-Beams and Inverted-T Beams

$h_{buildup}$		in	*Buildup or haunch is the concrete between the bottom of deck and top of beam.*

Slab-Beams (Florida Slab Beams, Flat Slab Beams)

Slab Beam Width	54	in	*Width of the slab unit (not including Gap width)*
Slab Beam Thickness	15	in	*Thickness of the slab unit.*
$t_{deck.delta}$	1.0980	in	*maximum additional deck thickness over support to accommodate camber, used for additional DL only.*
Gap	1.016	in	*Gap distance between slab beams.(Beam Spacing - Slab Beam Width)*

Double Tee Beams

Width Double-T		ft	*Width of the Double-T unit.*
$Depth_{flange}$		in	*Depth of the flange of the Double-T unit, See Beam Cross Section above.*

Concrete Material Properties

FDOT Environmental Classification: Slightly / Moderately / Extremely

Environmental Classification determines the Allowable Tension Stress

f`c of deck or WS	4.5	ksi	*Strength of deck or wearing surface concrete.*
f`c of beam or slab	8.5	ksi	*28 day concrete strength of beam or slab.*
f`ci of beam or slab	6	ksi	*Release concrete strength of beam or slab.*

Help - Conc. Strength

Permit Truck Axle Loads and Spacings

Default value for the permit truck is the FL-120 truck

Reset Permit Truck Data

Number of Permit Axles 3 — *Number of wheel loads that comprise the permit truck*

Permit Uniform LL 0 kip/ft — *Permit Uniform Live Load coincident with truck load*

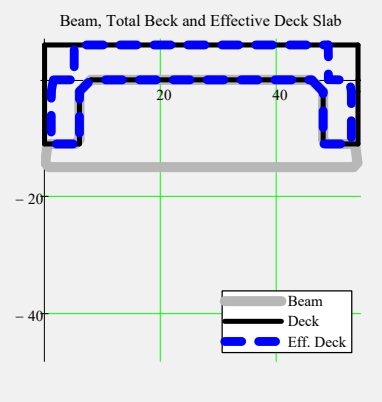
Help - Permit Truck

Axle Number	1	2	3	4	5	6	7	8	9	10	11
Load (kips)	0.01	0.01	0.01	0	0	0	0	0	0	0	0
Spacing (ft)	n/a	14	14	0	0	0	0	0	0	0	0

Custom Beam Coordinates, Initialize Data, Section Properties and Dead Loads

LRFD Live Load Distribution Factors, User Defined Live Load Distribution Factors and Section Properties

Section View: Beam, Deck and Effective Deck Slab



$g_{mom} = 1.00$

$g_{shear} = 1.00$

BeamType = "FSB15"

$t_{deck} = 6 \cdot in$

$h_{buildup} = 0 \cdot in$

$h_{beam} = 15 \cdot in$

$BeamSpacing = 54 \cdot in$

$b_e = 54.01 \cdot in$ *effective slab width*

Additional Dead Load Data:

Uniform Dead Loads

Additional Uniform Noncomposite DL	0.0	$\frac{kip}{ft}$
Additional Uniform Composite DL	0	$\frac{kip}{ft}$

$w_{deck} = 0.48 \cdot \frac{kip}{ft}$ $w_{beam} = 0.7 \cdot \frac{kip}{ft}$ $w_{forms} = 0.00 \cdot \frac{kip}{ft}$

$w_{future.ws} = 0 \cdot \frac{kip}{ft}$ $w_{barrier} = 0.05 \cdot \frac{kip}{ft}$

Diaphragms (Point) Dead Loads $L_{beam} = 51.83\ ft$

End Diaphragms or Point Loads over bearing included in bearing reaction calculation only.
Intermediate Diaphragms or Point Loads included in shear, moment, and bearing reaction calculations

Diaphragm (Point) Dead Loads and Location

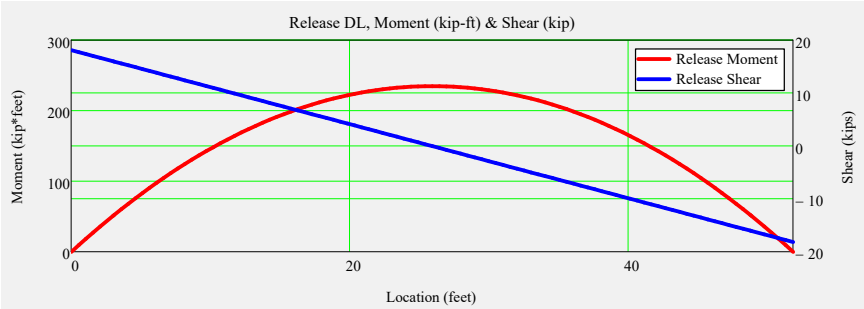
	Begin	Int. 1	Int. 2	Int. 3	End
Load (kips)	0	0	0	0	0
Distance (ft)	0	0	0	0	0

Distance measured from CL Bearing at begin bridge

Dead Loads, DL Moment, DL Shears

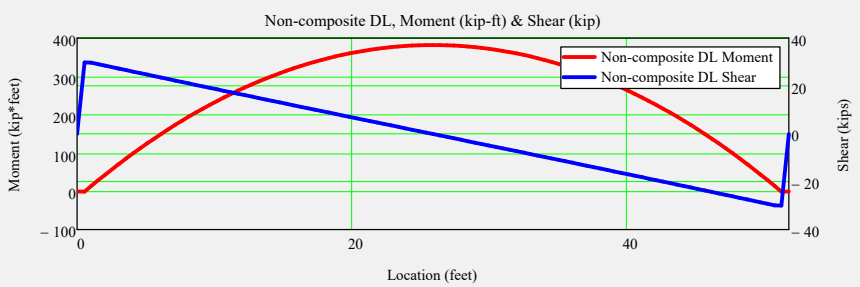
Release Dead Load Moments and Shears

$L_{span} = 50.75\ ft$



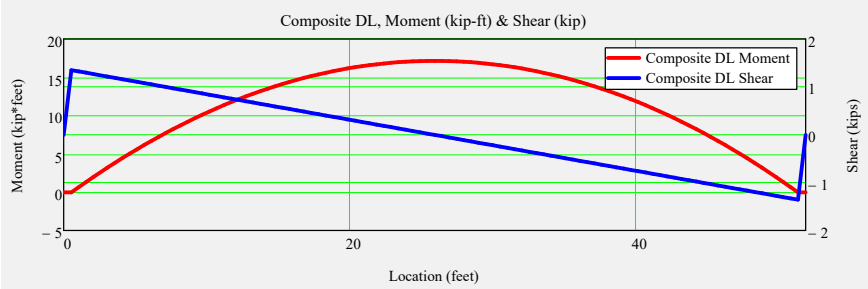
$\max(V_{release}) = 18.1 \cdot kip$ $\max(M_{release}) = 235.0 \cdot kip \cdot ft$ $\min(V_{release}) = -18.1 \cdot kip$

Noncomposite Dead Load Moments and Shears



$\max(V_{dl.non.comp}) = 29.9 \cdot kip$ $\max(M_{dl.non.comp}) = 382.3 \cdot kip \cdot ft$ $\min(V_{dl.non.comp}) = -29.9 \cdot kip$

Composite Dead Load Moments and Shear

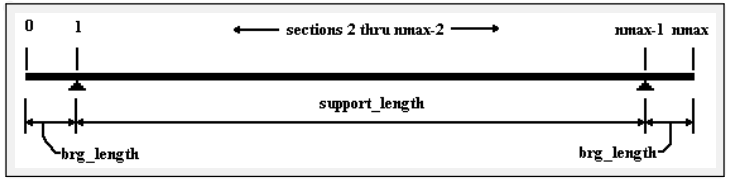


$\max(V_{dl.comp}) = 1.4 \cdot kip$ $\max(M_{dl.comp}) = 17.2 \cdot kip \cdot ft$ $\min(V_{dl.comp}) = -1.4 \cdot kip$

Live Load Moments and Shears

$WRITEPRN(\text{"coord.dat"}) := \frac{Coord_{beam}}{ft}$

Live Load Moments and Shears



$L_{c.c.bearings} := L_{beam} - 2 \cdot BearingDistance$ *Distance between the center line of bearings*

$Location_1 := BearingDistance$ $Location_0 := 0 \cdot ft$ *Location vector defines the x location of the sections investigated relative to the start of the beam*

$Location_{ns} := \frac{L_{c.c.bearings}}{nmax - 2} \cdot (ns - 1) + Location_1$ $Location_{nmax} := L_{beam}$ *The Location vector need not be in even increments*

$a_n := Location_n - BearingDistance$ *a vector gives the section x coordinate relative to the center line of the first bearing*

$b_n := L_{c.c.bearings} - a_n$ *b vector gives the distance from the section to the center line of the last bearing*

$P := \begin{pmatrix} 0 & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{pmatrix} \cdot kip$ *P vector set contains the magnitudes of the system of concentrated loads (HS20 is default)*

$d := \begin{pmatrix} 0 & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{pmatrix} \cdot ft$ *d vector set contains the spacings between the concentrated loads. The first and last values are place holders and should always be zero*

$PermitAxleLoad^T = (0.01 \quad 0.01 \quad 0.01) \cdot kip$

$PermitAxleSpacing^T = (0 \quad 14 \quad 14 \quad 0)\ ft$

$FatigueAxleLoad := \begin{pmatrix} 0 \\ 0 \\ 0 \end{pmatrix} \cdot kip$ $FatigueAxleSpacing := \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot ft$

$P^{\langle 3 \rangle} := \text{PermitAxleLoad}$ $\quad P^{\langle 4 \rangle} := \text{reverse(PermitAxleLoad)}$

$d^{\langle 3 \rangle} := \text{PermitAxleSpacing}$ $\quad d^{\langle 4 \rangle} := \text{reverse(PermitAxleSpacing)}$

$P^{\langle 5 \rangle} := \text{FatigueAxleLoad}$ $\quad P^{\langle 6 \rangle} := \text{reverse(FatigueAxleLoad)}$

$d^{\langle 5 \rangle} := \text{FatigueAxleSpacing}$ $\quad d^{\langle 6 \rangle} := \text{reverse(FatigueAxleSpacing)}$

$$P = \begin{pmatrix} 0 & 0 & 0 & 0.01 & 0.01 & 0 & 0 \\ 0 & 0 & 0 & 0.01 & 0.01 & 0 & 0 \\ 0 & 0 & 0 & 0.01 & 0.01 & 0 & 0 \end{pmatrix} \cdot kip \qquad d = \begin{pmatrix} 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 14 & 14 & 0 & 0 \\ 0 & 0 & 0 & 14 & 14 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 \end{pmatrix} ft$$

$\text{NumberOfTrucks} := \text{cols}(P) = 7$ *number of trucks*

$nt := 0 .. (\text{NumberOfTrucks} - 1)$ *nt vector of length equal to number of Trucks*

$\text{TruckLength}_{nt} := \sum_{qt} \left(d^{\langle nt \rangle}\right)_{qt}$ *total length of the concentrated load system*

$\text{inc}_{nt} := \text{floor}\left[\dfrac{300}{\left(\dfrac{\text{TruckLength}_{nt}}{L_{beam}}\right) + 1}\right]$ *number of even increments that the load system will use to advance across the span*

$\text{add_inc}_{nt} := \text{ceil}\left(\dfrac{\text{TruckLength}_{nt} \cdot \text{inc}_{nt}}{L_{beam}}\right)$ *additional increments required to get the load system off of the span*

$\text{lastp}_{nt} := 199$ *value of the last increment*

$\text{step}_{nt} := \dfrac{L_{beam} + \text{TruckLength}_{nt} - 0.0000001 \cdot ft}{\text{lastp}_{nt}}$ *distance between increments*

Remove units for calculations

$a := \dfrac{a}{ft} \qquad b := \dfrac{b}{ft} \qquad P := \dfrac{P}{kip} \qquad d := \dfrac{d}{ft}$

Functions used to determine moments and shears

$$fM_{int}(P, a, b, loc, brg, sl, M, n, z) := \begin{cases} Ans \leftarrow \dfrac{P_z \cdot a_n \cdot \left[sl - \left(loc_z - brg\right)\right]}{sl} + M_n & \text{if } a_n < \left(loc_z - brg\right) \\ Ans \leftarrow \dfrac{P_z \cdot b_n \cdot \left(loc_z - brg\right)}{sl} + M_n & \text{otherwise} \end{cases}$$

$$fV_{int}(P, loc, brg, sl, SZ, n, z) := \begin{cases} Ans \leftarrow \dfrac{P_z \cdot \left[sl - \left(loc_z - brg\right)\right]}{sl} + SZ_n & \text{if } a_n < \left(loc_z - brg\right) \\ Ans \leftarrow \dfrac{P_z \cdot \left(brg - loc_z\right)}{sl} + SZ_n & \text{otherwise} \end{cases}$$

$$fM_{start}(P, a, b, loc, brg, sl, M, n, z) := \begin{cases} Ans \leftarrow M_n + \left(loc_z - brg\right) \cdot P_z & \text{if } (n > 0) \cdot (n < 2) \\ Ans \leftarrow \left(loc_z - brg\right) \cdot P_z \cdot \dfrac{b_n}{sl} + M_n & \text{otherwise} \end{cases}$$

$$fV_{start}(P, loc, brg, sl, SZ, n, z) := Ans \leftarrow \left[P_z \cdot \dfrac{\left(brg - loc_z\right) + sl}{sl} - P_z\right] + SZ_n$$

$$fM_{end}(P, a, b, loc, brg, sl, M, n, z, k) := \begin{cases} Ans \leftarrow P_z \cdot \left[(sl + brg) - loc_z\right] + M_n & \text{if } [n > (k - 2)] \cdot (n < k) \\ Ans \leftarrow P_z \cdot \left[(sl + brg) - loc_z\right] \cdot \dfrac{sl - b_n}{sl} + M_n & \text{otherwise} \end{cases}$$

$$f_{end}(P, loc, brg, sl, SZ, n, z, k) := Ans \leftarrow P_z \cdot \left(1 - \dfrac{loc_z - brg}{sl}\right) + SZ_n$$

$$M_{ll}(a, b, L, k, brg, P, d, lp, nl, st) := \left| \begin{array}{l} sl \leftarrow L - 2 \cdot brg \\ \text{for } n \in 0 .. k \\ \quad \text{for } j \in 0 .. 5 \\ \quad\quad R_{n,j} \leftarrow 0 \\ \text{for } q \in 0 .. lp \\ \quad \left| \begin{array}{l} \text{for } n \in 0 .. k \\ \quad \left| \begin{array}{l} M_n \leftarrow 0 \\ SZ_n \leftarrow 0 \end{array} \right. \\ al \leftarrow q \cdot st \\ \text{for } z \in 0 .. (nl - 1) \\ \quad \left| \begin{array}{l} loc_z \leftarrow al - \sum_{t=0}^{z} d_t \\ \text{for } n \in 1 .. (k - 1) \qquad \text{if } \left(loc_z < brg\right) \cdot \left(loc_z \geq 0\right) \\ \quad \left| \begin{array}{l} M_n \leftarrow fM_{start}(P, a, b, loc, brg, sl, M, n, z) \\ SZ_n \leftarrow fV_{start}(P, loc, brg, sl, SZ, n, z) \end{array} \right. \\ \text{for } n \in 1 .. (k - 1) \qquad \text{if } \left(loc_z > brg\right) \cdot \left[loc_z \leq (sl + brg)\right] \\ \quad \left| \begin{array}{l} M_n \leftarrow fM_{int}(P, a, b, loc, brg, sl, M, n, z) \\ SZ_n \leftarrow fV_{int}(P, loc, brg, sl, SZ, n, z) \end{array} \right. \\ \text{for } n \in 2 .. (k - 1) \qquad \text{if } \left[loc_z > (sl + brg)\right] \cdot \left(loc_z \leq L\right) \end{array} \right. \end{array} \right.$$

```
M_n ← fM_end(P, a, b, loc, brg, sl, M, n, z, k)
SZ_n ← f_end(P, loc, brg, sl, SZ, n, z, k)
for n ∈ 1 .. k − 1
  R_n,0 ← M_n  if R_n,0 < M_n
  R_n,1 ← M_n  if R_n,1 > M_n
  if R_n,2 < SZ_n
    R_n,2 ← SZ_n
    R_n,3 ← M_n
  if R_n,4 > SZ_n
    R_n,4 ← SZ_n
    R_n,5 ← M_n
R
```

$$M_{truck_{nt}} := M_{ll}\left[a, b, \left(\frac{L_{beam}}{ft}\right), nmax, \left(\frac{BearingDistance}{ft}\right), P^{\langle nt \rangle}, d^{\langle nt \rangle}, lastp_{nt}, rows(P), \left(\frac{step_{nt}}{ft}\right)\right]$$

Graphs of the Lane, Truck, Permit Truck, Strength I, and Strength II Live Loads

The results are contained in a matrix with each row associated with the same numbered section and the columns containing the following data: *Column 0 --- the maximum positive moment*

Column 1 --- the maximum negative moment (if any depending on the Bearing Distance)
Column 2 --- maximum positive shear
Column 3 --- moment associated with the maximum positive shear
Column 4 --- maximum negative shear
Column 5 --- moment associated with the maximum negative shear

```
compl(A, ns, ne, col) :=
  res ← (A_ns)^⟨col⟩
  for j ∈ (ns + 1) .. ne
    res ← augment[res, (A_j)^⟨col⟩]
  res
```

```
sift(AA, ns, ne, nax) :=
  for ik ∈ 0 .. nax
    test ← 0
    res_ik ← 0
    for j ∈ ns .. ne
      if AA_ik,j > test
        test ← AA_ik,j
        res_ik ← j
  res
```

Design Truck Maximum Moments, Maximum Shears, and Moments associated with Maximum Shears

$pos_m_tot := compl(M_{truck}, 0, 2, 0)$

$neg_m_tot := compl(M_{truck}, 0, 2, 1)$

$M_{truck.pos_n} := max\left[\left(pos_m_tot^T\right)^{\langle n \rangle}\right] \cdot kip \cdot ft$

$M_{truck.neg_n} := min\left[\left(neg_m_tot^T\right)^{\langle n \rangle}\right] \cdot kip \cdot ft$

$pos_v_tot := compl(M_{truck}, 0, 2, 2)$

$pos_vm_tot := compl(M_{truck}, 0, 2, 3)$

$pos_v_index := sift[(pos_v_tot), 0, 2, nmax]$

$V_{truck.pos_n} := pos_v_tot_{n, pos_v_index_n} \cdot kip$

$Mshr_{truck.pos_n} := pos_vm_tot_{n, pos_v_index_n} \cdot kip \cdot ft$

$neg_v_tot := compl(M_{truck}, 0, 2, 4)$

$neg_vm_tot := compl(M_{truck}, 0, 2, 5)$

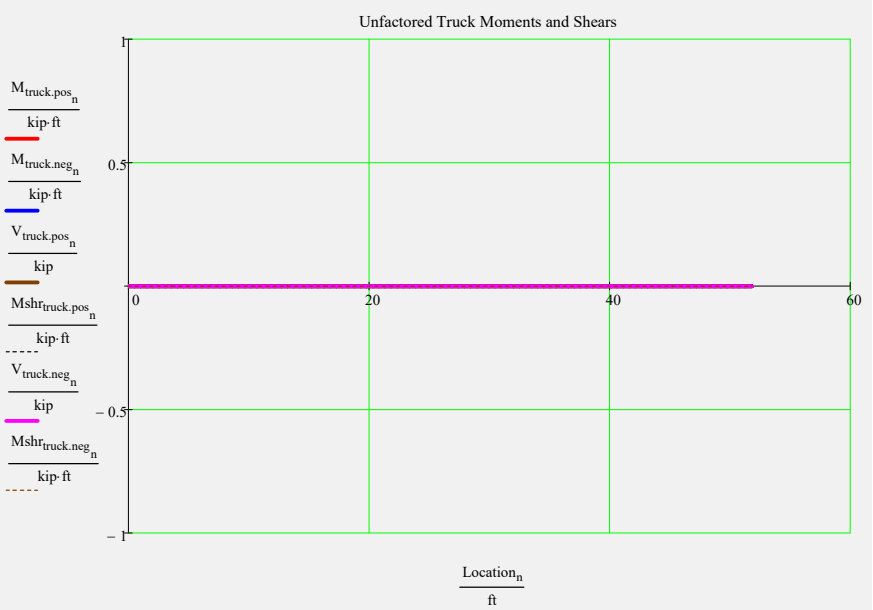
$neg_v_index := sift[neg_v_tot \cdot (-1), 0, 2, nmax]$

$V_{truck.neg_n} := neg_v_tot_{n, neg_v_index_n} \cdot kip$

$Mshr_{truck.neg_n} := neg_vm_tot_{n, neg_v_index_n} \cdot kip \cdot ft$

$pos_m_fatigue := compl(M_{truck}, 5, 6, 0)$

$M_{fatiguetruck.pos_n} := max\left[\left(pos_m_fatigue^T\right)^{\langle n \rangle}\right] \cdot kip \cdot ft$



$\max(M_{truck.pos}) = 0 \cdot kip \cdot ft$ $\quad$ $\min(M_{truck.neg}) = 0 \cdot kip \cdot ft$

$\max(V_{truck.pos}) = 0 \cdot kip$ $\quad$ $\max(Mshr_{truck.pos}) = 0 \cdot kip \cdot ft$

Lane Load Maximum Moments, Maximum Shears, and Moments associated with Maximum Shears

$$M_{lane.pos_n} := \frac{0.413 \cdot \frac{kip}{ft} \cdot a_n \cdot ft}{2} \cdot \left(L_{c.c.bearings} - a_n \cdot ft\right)$$

$M_{lane.pos_0} := 0 \cdot kip \cdot ft$ $\quad$ $M_{lane.pos_{nmax}} := 0 \cdot kip \cdot ft$

$$M_{lane.neg_n} := \frac{-\left(0.413 \cdot \frac{kip}{ft}\right) \cdot BearingDistance^2}{2}$$

$M_{lane.neg_0} := 0 \cdot kip \cdot ft$ $\quad$ $M_{lane.neg_{nmax}} := 0 \cdot kip \cdot ft$

$$V_{lane.neg_n} := \left[\frac{-0.413 \cdot \frac{kip}{ft} \cdot \left(a_n \cdot ft\right)^2}{2 \cdot L_{c.c.bearings}}\right]$$

$$V_{lane.pos_n} := \frac{0.413 \cdot \frac{kip}{ft} \cdot \left(b_n \cdot ft\right)^2}{2 \cdot L_{c.c.bearings}}$$

$V_{lane.neg_0} := 0 \cdot kip$ $\quad$ $V_{lane.neg_{nmax}} := 0 \cdot kip$ $\quad$ $V_{lane.pos_0} := 0 \cdot kip$ $\quad$ $V_{lane.pos_{nmax}} := 0 \cdot kip$

$Mshr_{lane.neg_n} := \left(-V_{lane.neg}\right)_n \cdot b_n \cdot ft$ $\quad$ $Mshr_{lane.pos_n} := V_{lane.pos_n} \cdot a_n \cdot ft$

$PermitUniformLoad := 0.0 \cdot klf$

$$M_{lane.permit.pos_n} := \frac{\left(PermitUniformLoad + 0.1 \cdot \frac{lbf}{ft}\right) \cdot a_n \cdot ft}{2} \cdot \left(L_{c.c.bearings} - a_n \cdot ft\right)$$

$M_{lane.permit.pos_0} := 0 \cdot kip \cdot ft$

$M_{lane.permit.pos_{nmax}} := 0 \cdot kip \cdot ft$

$$M_{lane.permit.neg_n} := \frac{-\left(PermitUniformLoad + 0.1 \cdot \frac{lbf}{ft}\right) \cdot BearingDistance^2}{2}$$

$M_{lane.permit.neg_0} := 0 \cdot kip \cdot ft$

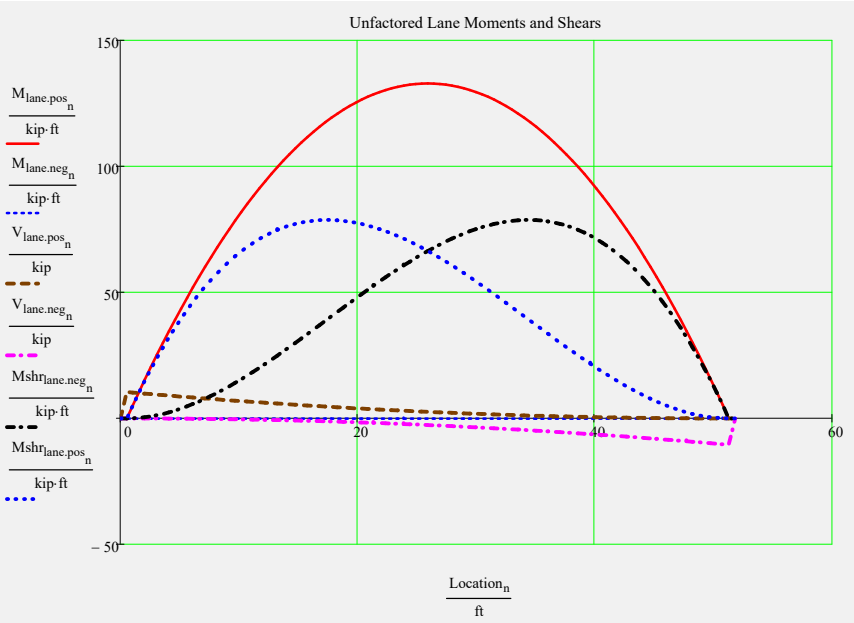
$M_{lane.permit.neg_{nmax}} := 0 \cdot kip \cdot ft$

$$V_{lane.permit.neg_n} := \left[\frac{-\left(PermitUniformLoad + 0.1 \cdot \frac{lbf}{ft}\right) \cdot \left(a_n \cdot ft\right)^2}{2 \cdot L_{c.c.bearings}}\right]$$

$$V_{lane.permit.pos_n} := \frac{\left(PermitUniformLoad + 0.1 \cdot \frac{lbf}{ft}\right) \cdot \left(b_n \cdot ft\right)^2}{2 \cdot L_{c.c.bearings}}$$

$V_{lane.permit.neg_0} := 0 \cdot kip$ $\quad$ $V_{lane.permit.neg_{nmax}} := 0 \cdot kip$ $\quad$ $V_{lane.permit.pos_0} := 0 \cdot kip$ $\quad$ $V_{lane.permit.pos_{nmax}} := 0 \cdot kip$

$Mshr_{lane.permit.neg_n} := \left(-V_{lane.permit.neg}\right)_n \cdot b_n \cdot ft$ $\quad$ $Mshr_{lane.permit.pos_n} := V_{lane.permit.pos_n} \cdot a_n \cdot ft$



$\max(M_{lane.pos}) = 133 \cdot kip \cdot ft$ $\qquad$ $\min(M_{lane.neg}) = -0.1 \cdot kip \cdot ft$

$\max(V_{lane.pos}) = 10.5 \cdot kip$ $\qquad$ $\max(Mshr_{lane.pos}) = 78.8 \cdot kip \cdot ft$

Strength I Live Load Maximum Moments, Maximum Shears, and Moments associated with Maximum Shears (Undistributed)

$M_{live.StrengthI.pos_n} := \left[M_{truck.pos_n} \cdot (1.33) + M_{lane.pos_n}\right] \cdot 1.75$ $\qquad$ $M_{live.StrengthI.neg_n} := \left[M_{truck.neg_n} \cdot (1.33) + M_{lane.neg_n}\right] \cdot 1.75$

$V_{live.StrengthIa_n} := \left[V_{truck.pos_n} \cdot (1.33) + V_{lane.pos_n}\right] \cdot 1.75$ $\qquad$ $V_{live.StrengthIb_n} := \left[V_{truck.neg_n} \cdot (1.33) + V_{lane.neg_n}\right] \cdot 1.75$

$V_{live.StrengthI_n} := \left|\text{if}\left(\left|V_{live.StrengthIa_n}\right| > \left|V_{live.StrengthIb_n}\right|, V_{live.StrengthIa_n}, V_{live.StrengthIb_n}\right)\right|$

$Mshr_{live.StrengthI_n} := \text{if}\left[\left|V_{live.StrengthIa_n}\right| > \left|V_{live.StrengthIb_n}\right|, \left(Mshr_{truck.pos_n} \cdot 1.33 + Mshr_{lane.pos_n}\right), \left(\left(Mshr_{truck.neg_n} \cdot 1.33 \ldots + Mshr_{lane.neg_n}\right)\right)\right] \cdot 1.75$

$\max(M_{live.StrengthI.pos}) = 233 \cdot kip \cdot ft$ $\qquad$ $\min(M_{live.StrengthI.neg}) = -0 \cdot kip \cdot ft$

$\max(V_{live.StrengthI}) = 18 \cdot kip$ $\qquad$ $\max(Mshr_{live.StrengthI}) = 138 \cdot kip \cdot ft$

Permit Truck Maximum Moments, Maximum Shears, and Moments associated with Maximum Shears

$ppos_m_tot := compl(M_{truck}, 3, 4, 0)$ $\qquad$ $M_{permit.pos_n} := \max\left[\left(ppos_m_tot^T\right)^{\langle n \rangle}\right] \cdot kip \cdot ft$

$pneg_m_tot := compl(M_{truck}, 3, 4, 1)$ $\qquad$ $M_{permit.neg_n} := \min\left[\left(pneg_m_tot^T\right)^{\langle n \rangle}\right] \cdot kip \cdot ft$

$ppos_v_tot := compl(M_{truck}, 3, 4, 2)$ $\qquad$ $ppos_vm_tot := compl(M_{truck}, 3, 4, 3)$

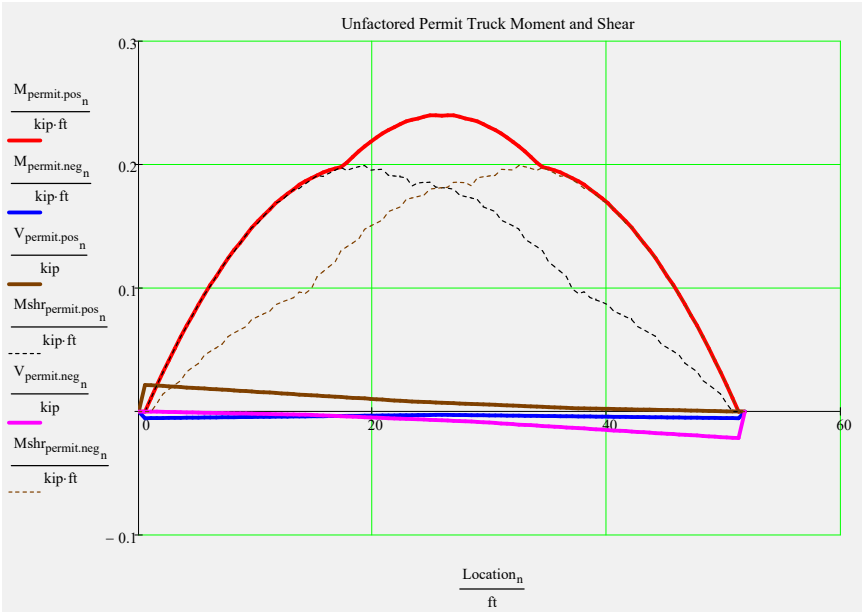
$ppos_v_index := sift[(ppos_v_tot), 0, 1, nmax]$ $\qquad$ $V_{permit.pos_n} := ppos_v_tot_{n, ppos_v_index_n} \cdot kip$

$Mshr_{permit.pos_n} := ppos_vm_tot_{n, ppos_v_index_n} \cdot kip \cdot ft$

$pneg_v_tot := compl(M_{truck}, 3, 4, 4)$ $\qquad$ $pneg_vm_tot := compl(M_{truck}, 3, 4, 5)$

$pneg_v_index := sift[pneg_v_tot \cdot (-1), 0, 1, nmax]$ $\qquad$ $V_{permit.neg_n} := pneg_v_tot_{n, pneg_v_index_n} \cdot kip$

$Mshr_{permit.neg_n} := pneg_vm_tot_{n, pneg_v_index_n} \cdot kip \cdot ft$



$\max(M_{permit.pos}) = 0 \cdot kip \cdot ft$ $\qquad$ $\min(M_{permit.neg}) = -0 \cdot kip \cdot ft$

$\max(V_{permit.pos}) = 0 \cdot kip$ $\qquad$ $\max(Mshr_{permit.pos}) = 0 \cdot kip \cdot ft$

Strength II Live Load Maximum Moments, Maximum Shears, and Moments associated with Maximum Shears (undistributed)

$M_{live.StrengthII.pos_n} := \left(1.33\, M_{permit.pos_n} + M_{lane.permit.pos_n}\right)\cdot 1.35$ *(LRFD C3.6.2.1)*

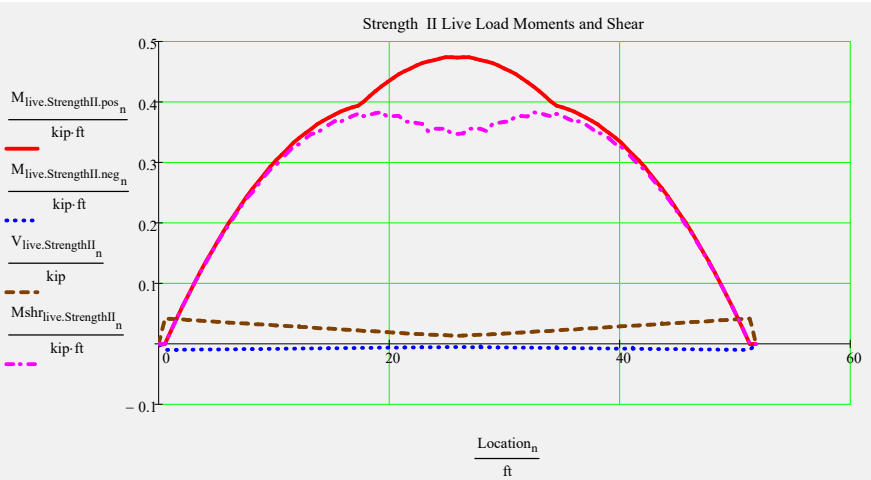
$M_{live.StrengthII.neg_n} := \left(1.33\, M_{permit.neg_n} + M_{lane.permit.neg_n}\right)\cdot 1.35$

$V_{live.StrengthIIa_n} := \left(1.33\cdot V_{permit.pos_n} + V_{lane.permit.pos_n}\right)\cdot 1.35$

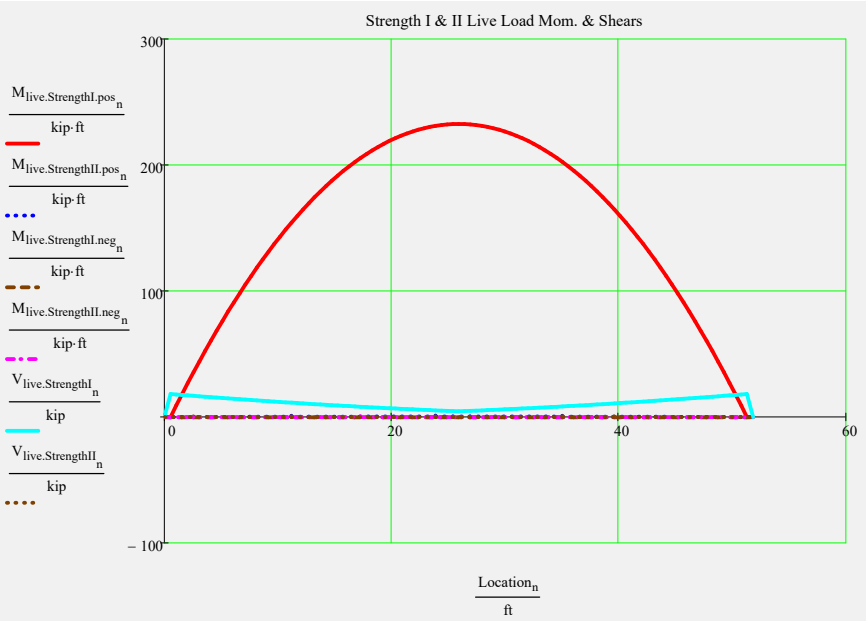
$V_{live.StrengthIIb_n} := \left(1.33\cdot V_{permit.neg_n} + V_{lane.permit.neg_n}\right)\cdot 1.35$

$V_{live.StrengthII_n} := \left|\text{if}\left(\left|V_{live.StrengthIIa_n}\right| > \left|V_{live.StrengthIIb_n}\right|, V_{live.StrengthIIa_n}, V_{live.StrengthIIb_n}\right)\right|$

$Mshr_{live.StrengthII_n} := \text{if}\left(\left|V_{live.StrengthIIa_n}\right| > \left|V_{live.StrengthIIb_n}\right|, Mshr_{permit.pos_n}\cdot 1.33 \ldots + Mshr_{lane.permit.pos_n}, Mshr_{permit.neg_n}\cdot 1.33 \ldots + Mshr_{lane.permit.neg_n}\right)\cdot 1.35$



$\max\left(M_{live.StrengthII.pos}\right) = 0\cdot kip\cdot ft$



$\max\left(M_{live.StrengthI.pos}\right) = 233\cdot kip\cdot ft$

Distributed Live Load Maximum Moments, Maximum Shears, and Moments associated with Maximum Shears

$M_{dist.live.pos_n} := \left(1.33\cdot M_{truck.pos_n} + M_{lane.pos_n}\right)\cdot g_{mom}$

$M_{dist.live.neg_n} := \left(1.33\cdot M_{truck.neg_n} + M_{lane.neg_n}\right)\cdot g_{mom}$

$V_{dist.live.pos_n} := \left(1.33\cdot V_{truck.pos_n} + V_{lane.pos_n}\right)\cdot g_{shear}$

$V_{dist.live.neg_n} := \left(1.33\cdot V_{truck.neg_n} + V_{lane.neg_n}\right)\cdot g_{shear}$

$Mshr_{dist.live.pos_n} := \left(1.33\cdot Mshr_{truck.pos_n} + Mshr_{lane.pos_n}\right)\cdot g_{mom}$

$Mshr_{dist.live.neg_n} := \left(1.33\cdot Mshr_{truck.neg_n} + Mshr_{lane.neg_n}\right)\cdot g_{mom}$

$\max\left(M_{dist.live.pos}\right) = 133\cdot kip\cdot ft$ $\max\left(V_{dist.live.pos}\right) = 10.5\cdot kip$ $\max\left(Mshr_{dist.live.pos}\right) = 78.8\cdot kip\cdot ft$

$\min(M_{dist.live.neg}) = -0.1 \cdot kip \cdot ft$ $\quad \min(V_{dist.live.neg}) = -10.5 \cdot kip$ $\quad \max(Mshr_{dist.live.neg}) = 78.8 \cdot kip \cdot ft$

Distributed Live Load - Permit Loading

$M_{dist.live.permit.pos_n} := \left(1.33 \cdot M_{permit.pos_n} + M_{lane.permit.pos_n}\right) \cdot g_{mom}$

$M_{dist.live.permit.neg_n} := \left(1.33 \cdot M_{permit.neg_n} + M_{lane.permit.neg_n}\right) \cdot g_{mom}$

$V_{dist.live.permit.pos_n} := \left(1.33 \cdot V_{permit.pos_n} + V_{lane.permit.pos_n}\right) \cdot g_{shear}$

$V_{dist.live.permit.neg_n} := \left(1.33 \cdot V_{permit.neg_n} + V_{lane.permit.neg_n}\right) \cdot g_{shear}$

$Mshr_{dist.live.permit.pos_n} := \left(1.33 \cdot Mshr_{permit.pos_n} + Mshr_{lane.permit.pos_n}\right) \cdot g_{mom}$

$Mshr_{dist.live.permit.neg_n} := \left(1.33 \cdot Mshr_{permit.neg_n} + Mshr_{lane.permit.neg_n}\right) \cdot g_{mom}$

$\max(M_{dist.live.permit.pos}) = 0.4 \cdot kip \cdot ft$ $\quad \max(V_{dist.live.permit.pos}) = 0 \cdot kip$

$\min(M_{dist.live.permit.neg}) = -0 \cdot kip \cdot ft$ $\quad \min(V_{dist.live.permit.neg}) = -0 \cdot kip$

$\max(Mshr_{dist.live.permit.pos}) = 0.3 \cdot kip \cdot ft$

$\max(Mshr_{dist.live.permit.neg}) = 0.3 \cdot kip \cdot ft$

Distributed Live Load - Fatigue Truck

$M_{dist.fatigue.pos_n} := \left(1.15 \cdot M_{fatiguetruck.pos_n}\right) \cdot g_{mom.fatigue}$

Bearing Reactions

$$f_{trk.reaction}(trk, asp, bd, lgth) := \left| \begin{array}{l} stop \leftarrow last(trk) \\ react \leftarrow 0 \\ \text{for } i \in 0..stop \\ \quad \left| \begin{array}{l} sum \leftarrow 0 \\ \text{for } j \in i..stop \\ \quad \left| \begin{array}{l} fact \leftarrow \dfrac{lgth - bd}{lgth - 2 \cdot bd} \ \text{ if } j = i \\ fact \leftarrow \dfrac{(lgth - 2 \cdot bd) - \left[\left(\sum_{n=i+1}^{j} asp_n\right) - bd\right]}{lgth - 2 \cdot bd} \ \text{ otherwise} \\ sum \leftarrow sum + fact \cdot trk_j \\ chk_{i,j} \leftarrow fact \end{array} \right. \\ react \leftarrow sum \ \text{ if } sum > react \end{array} \right. \\ react \end{array} \right.$$

$$Reaction_{truck} := \left| \begin{array}{l} ans \leftarrow 0 \\ \text{for } i \in 0..2 \\ \quad \left| \begin{array}{l} trkreact \leftarrow f_{trk.reaction}\left(P^{\langle i \rangle}, d^{\langle i \rangle}, \dfrac{BearingDistance}{ft}, \dfrac{L_{beam}}{ft}\right) \\ ans \leftarrow trkreact \ \text{ if } trkreact > ans \end{array} \right. \\ ans \end{array} \right. \cdot kip$$

$Reaction_{truck} = 0 \cdot kip$

$$Reaction_{permit.trk} := \left| \begin{array}{l} ans \leftarrow 0 \\ \text{for } i \in 3..4 \\ \quad \left| \begin{array}{l} trkreact \leftarrow f_{trk.reaction}\left(P^{\langle i \rangle}, d^{\langle i \rangle}, \dfrac{BearingDistance}{ft}, \dfrac{L_{beam}}{ft}\right) \\ ans \leftarrow trkreact \ \text{ if } trkreact > ans \end{array} \right. \\ ans \end{array} \right. \cdot kip$$

$Reaction_{permit.trk} = 0 \cdot kip$

$Reaction_{lane} := 0.413 \cdot \dfrac{kip}{ft} \cdot \dfrac{L_{beam}}{2} = 10.7 \cdot kip$

$Reaction_{lane.permit} := \left(PermitUniformLoad + 0.1 \dfrac{lbf}{ft}\right) \cdot \dfrac{L_{beam}}{2} = 0 \cdot kip$

$Reaction_{composite.dl} := w_{composite} \cdot \dfrac{L_{beam}}{2} = 1.4 \cdot kip$

$$Reaction_{noncomposite.dl} := w_{noncomposite} \cdot \frac{L_{beam}}{2} \ldots + \max\left[\left(\left|EndDiaphragmA\right| + \left|V_{point_1}\right|\right), \left(\left|EndDiaphragmE\right| + \left|V_{point_{nmax-1}}\right|\right)\right] = 30.6 \cdot kip$$

$$Reaction_{HL93} := \left(Reaction_{truck} \cdot 1.33 + Reaction_{lane}\right) \cdot g_{shear} = 10.7 \cdot kip$$

$$Reaction_{permit} := \left(Reaction_{permit.trk} \cdot 1.33 + Reaction_{lane.permit}\right) \cdot g_{shear} = 0 \cdot kip$$

$$Reaction_{LL} := Reaction_{HL93} = 10.7 \cdot kip$$

$$Reaction_{DL} := Reaction_{composite.dl} + Reaction_{noncomposite.dl} = 31.9 \cdot kip$$

```
WRITEPRN("beam.dat") := | r0 ← 0  if BeamType = "TypeII"
                        | r0 ← 1  if BeamType = "TypeIII"
                        | r0 ← 2  if BeamType = "TypeIV"
                        | r0 ← 3  if BeamType = "TypeV"
                        | r0 ← 4  if BeamType = "TypeVI"
                        | r0 ← 5  if BeamType = "FBT54"
                        | r0 ← 6  if BeamType = "FBT63"
                        | r0 ← 7  if BeamType = "FBT72"
                        | r0 ← 8  if BeamType = "FBT78"
                        | r0 ← 10  if BeamType = "FIB36"
                        | r0 ← 11  if BeamType = "FIB45"
                        | r0 ← 12  if BeamType = "FIB54"
                        | r0 ← 13  if BeamType = "FIB63"
                        | r0 ← 14  if BeamType = "FIB72"
                        | r0 ← 15  if BeamType = "FIB78"
                        | r0 ← 16  if BeamType = "FIB84"
                        | r0 ← 17  if BeamType = "FIB96"
                        | r0 ← 18  if BeamType = "FIB120"
                        | r0 ← 30  if substr(BeamType, 0, 3) = "FUB"
```



```
                        | r0 ← 50  if substr(BeamType, 0, 3) = "FUB"
                        | r0 ← 50  if BeamType = "FDT18"
                        | r0 ← 51  if BeamType = "FDT24"
                        | r0 ← 52  if BeamType = "FDT30"
                        | r0 ← 60  if (FSBBeamName = "FSB12x48")·(BeamType = "FSB12")
                        | r0 ← 61  if (FSBBeamName = "FSB15x48")·(BeamType = "FSB15")
                        | r0 ← 62  if (FSBBeamName = "FSB18x48")·(BeamType = "FSB18")
                        | r0 ← 63  if (FSBBeamName = "FSB12x49")·(BeamType = "FSB12")
                        | r0 ← 64  if (FSBBeamName = "FSB15x49")·(BeamType = "FSB15")
                        | r0 ← 65  if (FSBBeamName = "FSB18x49")·(BeamType = "FSB18")
                        | r0 ← 66  if (FSBBeamName = "FSB12x50")·(BeamType = "FSB12")
                        | r0 ← 67  if (FSBBeamName = "FSB15x50")·(BeamType = "FSB15")
                        | r0 ← 68  if (FSBBeamName = "FSB18x50")·(BeamType = "FSB18")
                        | r0 ← 69  if (FSBBeamName = "FSB12x51")·(BeamType = "FSB12")
                        | r0 ← 70  if (FSBBeamName = "FSB15x51")·(BeamType = "FSB15")
                        | r0 ← 71  if (FSBBeamName = "FSB18x51")·(BeamType = "FSB18")
                        | r0 ← 72  if (FSBBeamName = "FSB12x52")·(BeamType = "FSB12")
                        | r0 ← 73  if (FSBBeamName = "FSB15x52")·(BeamType = "FSB15")
                        | r0 ← 74  if (FSBBeamName = "FSB18x52")·(BeamType = "FSB18")
                        | r0 ← 75  if (FSBBeamName = "FSB12x53")·(BeamType = "FSB12")
                        | r0 ← 76  if (FSBBeamName = "FSB15x53")·(BeamType = "FSB15")
                        | r0 ← 77  if (FSBBeamName = "FSB18x53")·(BeamType = "FSB18")
                        | r0 ← 78  if (FSBBeamName = "FSB12x54")·(BeamType = "FSB12")
                        | r0 ← 79  if (FSBBeamName = "FSB15x54")·(BeamType = "FSB15")
                        | r0 ← 80  if (FSBBeamName = "FSB18x54")·(BeamType = "FSB18")
                        | r0 ← 81  if (FSBBeamName = "FSB12x55")·(BeamType = "FSB12")
                        | r0 ← 82  if (FSBBeamName = "FSB15x55")·(BeamType = "FSB15")
                        | r0 ← 83  if (FSBBeamName = "FSB18x55")·(BeamType = "FSB18")
                        | r0 ← 84  if (FSBBeamName = "FSB12x56")·(BeamType = "FSB12")
                        | r0 ← 85  if (FSBBeamName = "FSB15x56")·(BeamType = "FSB15")
                        | r0 ← 86  if (FSBBeamName = "FSB18x56")·(BeamType = "FSB18")
                        | r0 ← 87  if (FSBBeamName = "FSB12x57")·(BeamType = "FSB12")
                        | r0 ← 88  if (FSBBeamName = "FSB15x57")·(BeamType = "FSB15")
```

$r_0 \leftarrow 89$ if $(FSBBeamName = \text{"FSB18x57"}) \cdot (BeamType = \text{"FSB18"})$

$r_0 \leftarrow 90$ if $(FSBBeamName = \text{"FSB12x58"}) \cdot (BeamType = \text{"FSB12"})$

$r_0 \leftarrow 91$ if $(FSBBeamName = \text{"FSB15x58"}) \cdot (BeamType = \text{"FSB15"})$

$r_0 \leftarrow 92$ if $(FSBBeamName = \text{"FSB18x58"}) \cdot (BeamType = \text{"FSB18"})$

$r_0 \leftarrow 93$ if $(FSBBeamName = \text{"FSB12x59"}) \cdot (BeamType = \text{"FSB12"})$

$r_0 \leftarrow 94$ if $(FSBBeamName = \text{"FSB15x59"}) \cdot (BeamType = \text{"FSB15"})$

$r_0 \leftarrow 95$ if $(FSBBeamName = \text{"FSB18x59"}) \cdot (BeamType = \text{"FSB18"})$

$r_0 \leftarrow 96$ if $(FSBBeamName = \text{"FSB12x60"}) \cdot (BeamType = \text{"FSB12"})$

$r_0 \leftarrow 97$ if $(FSBBeamName = \text{"FSB15x60"}) \cdot (BeamType = \text{"FSB15"})$

$r_0 \leftarrow 98$ if $(FSBBeamName = \text{"FSB18x60"}) \cdot (BeamType = \text{"FSB18"})$

$r_0 \leftarrow 99$ if $substr(BeamType, 0, 3) = \text{"Inv"}$

$r_0 \leftarrow 99$ if $BeamType = \text{"Custom"}$

$r_0 \leftarrow 99$ otherwise

$r_1 \leftarrow \text{"e"}$

$r_2 \leftarrow \frac{h_{beam}}{ft}$

r

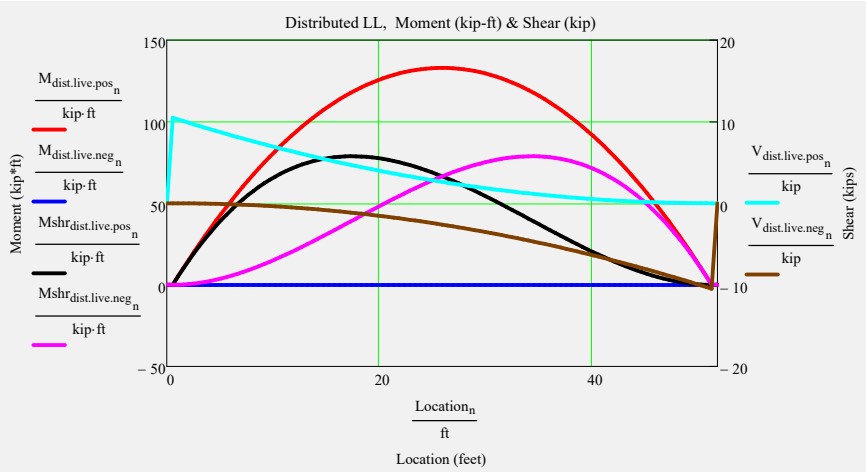
$BeamType = \text{"FSB15"}$ $FSBBeamName = \text{"FSB15x54"}$ $h_{beam} = 15 \cdot in$

Live Load Moments and Shears

Distributed Live Load Moments and Shears

$L_{span} = 50.75\,ft$

Live Load Distribution Factors $g_{mom} = 1$ $g_{mom.fatigue} = 1$ $g_{shear} = 1$



$\max(V_{dist.live.pos}) = 10.5 \cdot kip$ $\max(M_{dist.live.pos}) = 133.0 \cdot kip \cdot ft$ $\min(V_{dist.live.neg}) = -10.5 \cdot kip$

$Reaction_{LL} = 10.7 \cdot kip$ $\min(M_{dist.live.neg}) = -0.1 \cdot kip \cdot ft$

$Reaction_{DL} = 31.9 \cdot kip$ $\max(Mshr_{dist.live.pos}) = 79 \cdot kip \cdot ft$

Beam End Reactions...
with IM factor only

Reinforcement Properties

InputDataFile = "Able Canal - Spans 2-4 - Exterior - Ped. Load.dat"

Mild Reinforcement Type:

- Carbon Steel - Grade 60
- Stainless Steel - Grade 60
- Glass Fiber Reinforced Polymer
- Carbon Steel - Grade 80
- Stainless Steel - Grade 75

Help on Mild Reinforcement Types

Longitudinal Mild and Partial PS Reinforcement:

Excel Table of Standard FDOT Prestressed Beam Mild and Partial PS Longitudinal Reinforcement

Help on Longitudinal Mild and Partial PS Reinf

Mild Reinforcement

	Deck	Bottom of Beam	Beam End: Top	Beam End: Bottom
Location				
Area (in^2)	0.4108 *1	0	0	0
Distance (in)	2.3125 *2	0 *3		
Length (ft)			0	0
Bar Size		5 *4	0 *4	0 *4

Partial PS (Dormant) Strand

Location	Top of Beam
Diameter(in)	3/8 inch / 1/2 inch
Distance (in)	2
# Strands	4
Force per Strand (kip)	10

**1 - Area of longitudinal deck reinf. per unit width of deck, both layers combined. Typically one #5 Bar top & bottom for 8" decks.*
**2 - Distance measured from top of deck to centroid of deck longitudinal reinforcement, positive value.*
**3 - Distance measured from top of beam, positive value.*
**4 - Size of bars used to create* $A_{s.long}$ *needed to calculate development length.*

Prestressing Tendons:

Humidity	75 %	*% relative humidity (75% typical)*
Time: jacking to transfer	0.75 days	*Time in days between jacking and transfer.*

Prestress Strand Type

- Carbon Steel - Low Lax
- Carbon Steel - Stress Relieved
- Stainless Steel
- Carbon Fiber Reinforced Polymer

Help on Prestress Reinforcement Typ

Prestress Strand Size

- 0.6 in.
- 0.5 in.
- 0.5 in. Special
- 9/16 in.
- 0.7 in.

Help on Strand Generator | Help on Strand Debonding

Strand Pattern Generator for Entering Prestressing Strand Layout

Double click the icon to open the 'Strand Pattern Generator'. Specify the location and debonding of strands. When finished, press the 'Continue' button. Then press 'Read Strand Data' button below.

Press 'Read Strand Data' Button.to Read in Strand Generator Data

Read Strand Data

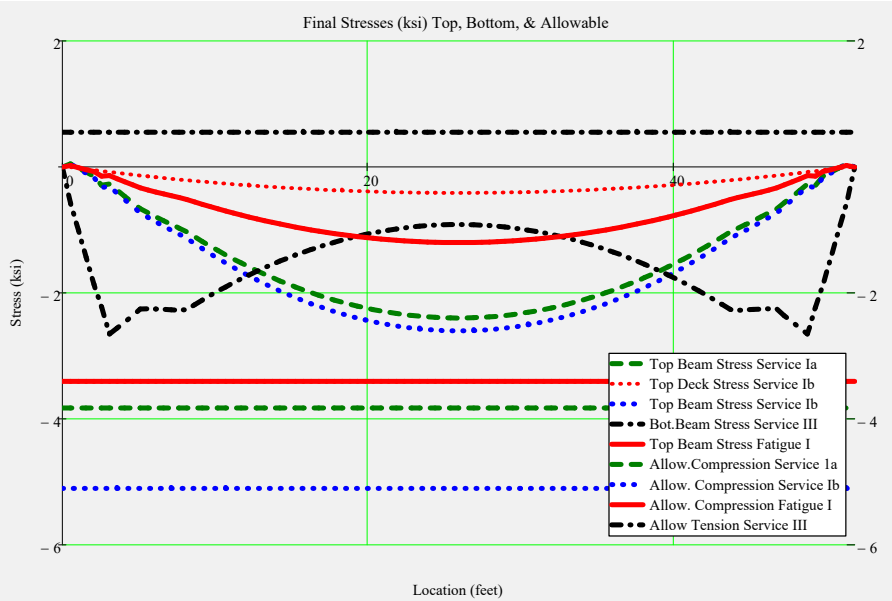
Press 'Read Strand Data' button to Read in Strand Generator Data.

Material Properties
Strand Data, Pattern, and Properties
Section Properties, Strand Profile Properties, and Service I and III Beam Stresses and Limits

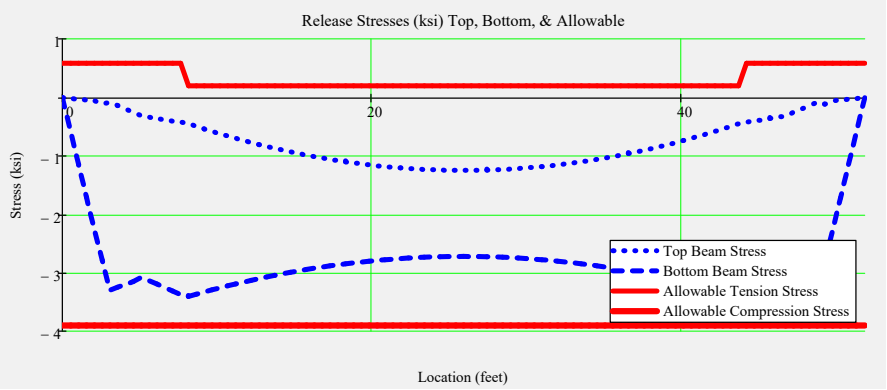
Service I and III Stresses and Limits and Tendon Layout Summary

tension (+), compression (--)

Final Stresses



Release Stresses



Stress Checks at Service Limit States

Release tension	$Check_f_{tension.rel}$ = "OK"	$\max(DC_f_{tension.rel}) = 0.68$
Release compression	$Check_f_{comp.rel}$ = "OK"	$\max(DC_f_{comp.rel}) = 0.87$
*Service III , PS+DL+0.8*LL*	$Check_f_{tension.ServiceIII}$ = "OK"	$\max(DC_f_{tension.ServiceIII}) = 0.83$
Service Ia , PS+DL	$Check_f_{comp.ServiceIa}$ = "OK"	$\max(DC_f_{comp.ServiceIa}) = 0.63$
Service Ib , PS+DL+LL	$Check_f_{comp.ServiceIb}$ = "OK"	$\max(DC_f_{comp.ServiceIb}) = 0.51$
*Fatigue I , (PS + DL)*0.5 +1.5 Fatigue Truck*	$Check_f_{comp.FatigueI}$ = "OK"	$\max(DC_f_{comp.FatigueI}) = 0.35$

StressChecks =

"StressCheck"	"Location(ft)"	"Stress(ksi)"	"AllowableStress(ksi)"
"Tension @ Release"	0.00	0.00	0.59
"Compression @ Release"	8.15	−3.40	−3.90
"Tension @ ServiceIII"	0.00	0.00	0.55
"CompressionIa @ ServiceI"	25.92	−2.40	−3.83
"CompressionIb @ ServiceI"	25.92	−2.60	−5.10
"Compression @ FatigueI"	25.92	−1.20	−3.40

CheckBeamEndTopLongReinf = "OK"

Strand Pattern Checks

1. *The number of strands debonded per row shall not exceed 45 percent of the strands provided in that row.*
2. *Debonding shall not be terminated for more than six strands in any given section. When a total of ten or fewer strands are debonded, debonding shall not be terminated for more than four strands in any given section*
3. *Longitudinal spacing of debonding termination locations shall be at least 60db apart.*
4. *Alternate bonded and debonded strand locations both horizontally and vertically.*
5. *For simple span precast, pretensioned girders, debonding length from the beam end should be limited to 20 percent of the span length or one half the span length minus the development length, whichever is less.*

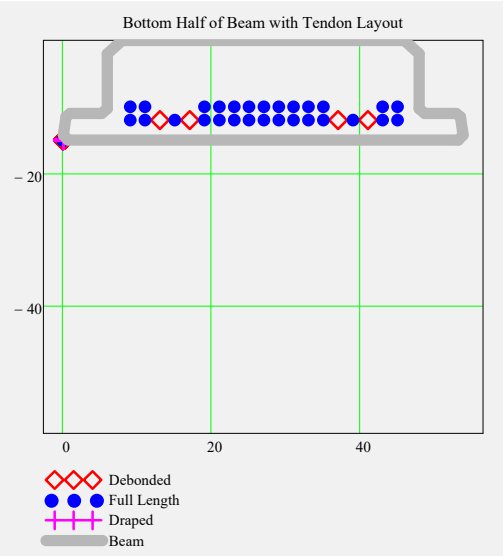
CheckPattern = ("OK", "OK", "OK", "OK", "OK")

For single-web flanged sections (I-beams, bulbtees, and inverted-tees):
1. *Bond all strands within the horizontal limits of the web*
2. *Bond the outer-most strands in all rows located within the full-width section of the flange.*
3. *Position debonded strands furthest from the vertical centerline.*

For multi-web sections having bottom flanges (voided slab, box beams and U-beams):
1. *Strands shall be bonded within 1.0 times the web width projection.*
2. *Bond the outer-most strands within the section.*
3. *Uniformly distribute debonded strands between webs.*

BeamType = "FSB15" CheckWebPattern = ("OK", "OK")

Tendon Layout



BeamType = "FSB15"

$h_{beam} = 15 \cdot in$

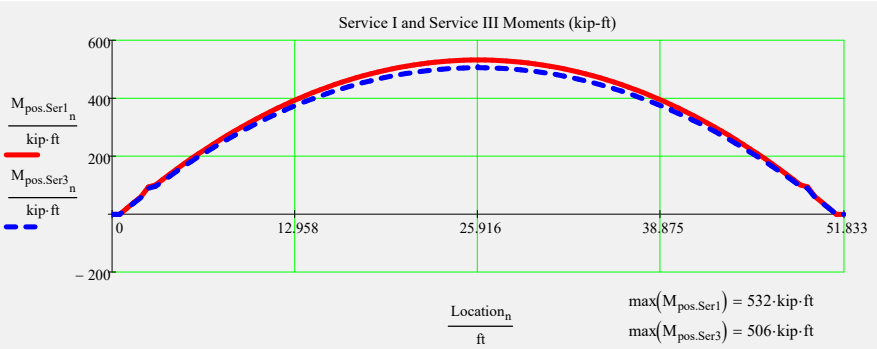
$A_{strand} = 0.217 \cdot in^2$

#Strands = 32

Prestress Losses Summary

$f_{pj} = 202.5 \cdot ksi$	$Check_f_{pt}$ = "OK"
$\Delta f_{pES} = 0 \cdot ksi$	*Note: Elastic shortening losses are zero in concrete stress calculations when using transformed section properties per* *LRFD 5.9.3.2.3*
$\Delta f_{pT} = -25 \cdot ksi$	$\frac{\Delta f_{pT}}{f_{pj}} = -12.22 \cdot \%$
$f_{pe} = 178 \cdot ksi$	$\frac{f_{pe}}{f_{pj}} = 87.78 \cdot \%$
$0.8 \cdot f_{py} = 194 \cdot ksi$	$Check_f_{pe}$ = "OK"

Moments at Service Limit States



Summary of Values at Midspan

Stresses =

"Stage"	"Top of Beam (ksi)"	"Bottom of Beam (ksi)"
"1"	–1.25	–2.72
"2"	–1.30	–2.21
"4"	–1.23	–2.27
"6"	–2.37	–1.29
"8"	–2.60	–0.91

PrestressForce =

"Time"	"Axial (kip)"	"Moment (kip*ft)"
"@Release"	–1446	–368
"@Final (about composite centroid)"	–1269	–305

Properties =

"Section"	"Area (in^2) "	"Inertia (in^4) "	"Top of Beam to Centroid (in)"
"Net Beam"	664.6	12861	–7.9
"Transformed Beam (initial)"	709.3	13394	–8.1
"Transformed Beam (final)"	704.4	13339	–8.1
"Composite Section"	1086.1	39332	–5

ServiceMoments =

"Load Type "	"Moment (kip*ft)"
"Release Dead Load"	235
"Non-Composite DL (incl. beam wt.)"	382
"Composite Dead Load"	17
"Distributed Live Load"	133

Stage 1 ---> At release with span length equal to length of the beam. Prestress losses are elastic shortening and overnight relax

Stage 2 ---> At final, starting with Stage 1 and adding the remaining prestress losses applied to the transformed beam

Stage 4 ---> Starting with Stage 2 and changing the support locations from the end of the beam to the bearing locations

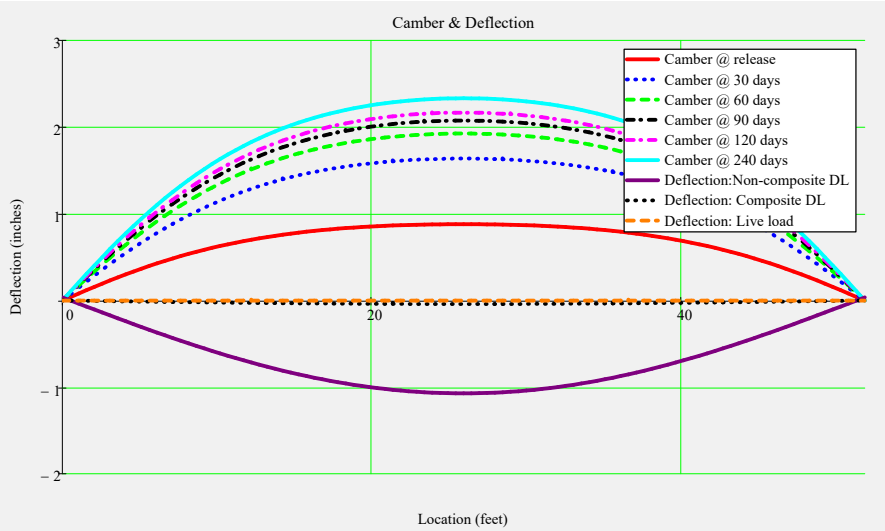
Stage 6 ---> Starting with Stage 4 and adding non-composite dead load (excluding beam wt included in Stage 1)

Stage 8 ---> Starting with Stage 6 and adding composite dead load and distributed live loads applied to the composite section

Camber at Release, Short Term Camber, & Summary of Slope Data

Camber at Different Stages and Deflection due to Dead Load and Live Load

$L_{span} = 50.75\,ft$



Slope&Defl =

"Stage"	"ΔL @ Beam Top (in)"	"ΔL @ Beam Bot. (in)"	"Slope at End (deg)"	"Midspan Defl (in)"
"Release"	–0.1000	–0.3936	0.3771	0.8815
"30 Days"	–0.2547	–0.7282	0.6822	1.6389
"60 Days"	–0.3126	–0.8547	0.7986	1.9277
"90 Days"	–0.3426	–0.9202	0.8588	2.0772
"120 Days"	–0.3610	–0.9603	0.8956	2.1685
"240 Days"	–0.3944	–1.0331	0.9624	2.3341
"Non-comp DL"	–0.0908	0.0783	–0.3230	–1.0705
"Comp DL"	–0.0021	0.0042	–0.0119	–0.0396
"LL"	0.0000	0.0000	0.0000	0.0000

Check net camber at end of construction (120 day) *SDG 4.3.2.D & 4.4.3.A*

$NetCamber := \max(Defl_{120day}) + \min(Defl_{dl.nc}) + \min(Defl_{dl.c}) = 1.06 \cdot in$

$CheckNetCamber := if(NetCamber > 0, "OK", "No Good")$ CheckNetCamber = "OK"

Moment Resistance

Moments and Moment Resistance at StrengthI and StrengthII Limit State

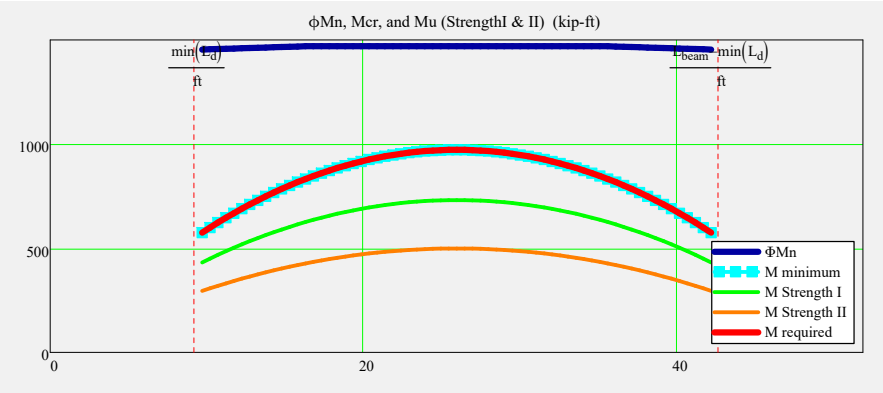
Values at midspan

$LimitStateForMoment_{51}$ = "Strength I" $\max(M_{Strength}) = 732 \cdot kip \cdot ft$

$IsSectionNonCompControlled?_{51}$ = "Yes" $M_{minimum_{51}} = 974 \cdot kip \cdot ft$ $M_{reqd_{51}} = 974 \cdot kip \cdot ft$

StrandType = "Carbon Steel - Low Lax" $\phi_{mom_{51}} = 0.84$ $\phi_{mom_{51}} \cdot (M_{n_{51}}) = 1471 \cdot kip \cdot ft$

$FailureMode_{51}$ = "N.A." CheckMinSSLimit = "N.A."



$\min(L_d) = 9.2\,ft$ $L_{beam} - \min(L_d) = 42.6\,ft$

$DC_{moment_{mn}} := \dfrac{M_{reqd_{mn}}}{\phi_{mom_{mn}} \cdot (M_{n_{mn}})}$ *Demand/Capacity Ratio at locations where all strands are fully developed* $\max(DC_{moment}) = 0.66$

CheckMomentCapacity := if(max(DC_moment) < 1.0, "OK", "No Good!") = "OK"

FSB only - Design Check of Transverse Reinforcing Bars E

FSB only - Design Check of Bottom Transverse Reinforcing Bars E

Spacing of Bars E is checked per Control of Cracking by Distribution of Reinforcement [LRFD 5.6.7 or AASHTO GFRP 2.6.7].
The Service I moment in transverse direction is calculated based on the r ratio to the longitudinal moment per LRFD 9.7.3.2.

$s \le \dfrac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$ *The maximum spacing of the mild steel reinforcement for control of cracking at the service limit state shall satisfy* *[LRFD 5.6.7]*

$\gamma_e := 1.00$

$s \le \min\left(1.15 \cdot \dfrac{C_b \cdot E_f \cdot w}{f_{fs}} - 2.5c_c, 0.92 \cdot \dfrac{C_b \cdot E_f \cdot w}{f_{fs}}\right)$ *[AASHTO GFRP 2.6.7-1]*

$d_c \le \dfrac{C_b E_f w}{2 f_{fs} \xi}$ (2.6.7-2) *[AASHTO GFRP Eq. 2.6.7-2]*

$C_b := 0.83$ *GFRP bars bond coefficient*

$w_{max} := 0.028in$ *limiting crack width*

$A_{BarsE} := 0.31in^2$ *Area of Bars E*

$s_{BarsE} := 6in$ *Typical Spacing of Bars E*

$s_{BarsE.NearEnds} := 12in$ *Spacing of Bars E near ends of span*

$n_{BarsE.NearEnds} := 6$ *Number of spaces of Bars E near ends of span*

BarsESize := 5 *Size of Bars E*

$c_c := 1.5in$ *clear cover*

Area and spacing of Bars E per location (up to half span)

$hs := 0 .. \dfrac{last(Location)}{2} - 1$

$A_{BarsE.prov_{hs}} :=$ if substr(BeamType, 0, 3) = "FSB"
  $\dfrac{A_{BarsE}}{s_{BarsE.NearEnds}}$ if $Location_{hs} \le n_{BarsE.NearEnds} \cdot s_{BarsE.NearEnds}$
  $\dfrac{A_{BarsE}}{s_{BarsE}}$ otherwise
  0 otherwise

$s_{BarsE.prov_{hs}} :=$ if substr(BeamType, 0, 3) = "FSB"
  $s_{BarsE.NearEnds}$ if $Location_{hs} \le n_{BarsE.NearEnds} \cdot s_{BarsE.NearEnds}$
  s_{BarsE} otherwise
  0 otherwise

$ratio := \dfrac{100}{\sqrt{\dfrac{L_{span}}{ft}}} \cdot \dfrac{1}{100}$ *LRFD 9.7.3.2*

ratio := if(ratio ≤ 0.5, ratio, 0.5) ratio = 0.14

$M_{transverse.pos.Ser1} := ratio \cdot \frac{M_{dist.live.pos} + M_{dl.comp}}{b_e} \cdot 1ft$ $\quad \max(M_{transverse.pos.Ser1}) = 4.68 \cdot kip \cdot ft$ *per foot width*

Note: there is no stress induced in Bars E from the self-weight of the FSB's or the CIP deck overlay.

$d_c := c_c + 0.5 \cdot \frac{BarsESize}{8} \cdot in = 1.81 \cdot in$ *Distance from extreme tension fiber to center of closest bar*

$h := t_{deck} + Height_{pocket} = 17 \cdot in$ *Overall thickness or depth of the component*

$d_s := h - d_c = 15.19 \cdot in$

$\beta_s := 1 + \frac{d_c}{0.7(h - d_c)} = 1.17$

$E_{c.deck} := 120000 \cdot K_1 \cdot \left(w_{c.deck} \cdot \frac{ft^3}{kip}\right)^2 \cdot \left(\frac{f_{c.deck}}{ksi}\right)^{0.33} \cdot ksi = 4145 \cdot ksi$

$n_s := \frac{E_s}{E_{c.deck}} = 7$ *Modular ratio*

$\rho_{hs} := \frac{A_{BarsE.prov_{hs}}}{d_s}$ *Reinforcement ratio*

$kk_{hs} := \sqrt{2 \cdot \rho_{hs} \cdot n_s + (\rho_{hs} \cdot n_s)^2} - \rho_{hs} \cdot n_s$

$j_{hs} := 1 - \frac{kk_{hs}}{3}$

$f_{r.pocket} := 0.24 \cdot \sqrt{f_{c.deck} \cdot ksi}$ $\quad f_{r.pocket} = 0.51 \cdot ksi$

$S_{pocket} := \frac{1}{6} \cdot 12 \cdot in \cdot h^2$ $\quad S_{pocket} = 578 \cdot in^3$

$\gamma_1 := 1.6$ *Flexural cracking variability factor*

$\gamma_3 := 0.67$ *Ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement (0.67 for ASTM A615, Grade 60 reinforcing steel per SDG 1.4.1)*

$M_{cr.pocket} := \gamma_1 \cdot \gamma_3 \cdot f_{r.pocket} \cdot S_{pocket}$ $\quad M_{cr.pocket} = 26.29 \cdot kip \cdot ft$ *per feet width*

$$f_{ss.Ser1_{hs}} := \begin{cases} \text{if substr(BeamType, 0, 3) = "FSB"} \\ \quad \begin{cases} 0 & \text{if } M_{transverse.pos.Ser1_{hs}} \le 0 \\ \frac{M_{transverse.pos.Ser1_{hs}}}{A_{BarsE.prov_{hs}} \cdot 1ft \cdot j_{hs} \cdot d_s} & \text{otherwise} \end{cases} \\ 0 \quad \text{otherwise} \end{cases}$$

Stress in Bars E (if moment is negative, stress in bottom steel is set to 0)

Required reinforcement spacing

$$Spa_{BarsE.reqd_{hs}} := \begin{cases} \text{if substr(BeamType, 0, 3) = "FSB"} \\ \quad \begin{cases} 10000in & \text{if } f_{ss.Ser1_{hs}} \le 0 \\ \text{otherwise} \\ \quad \begin{cases} \left(\frac{700 \cdot \gamma_e \cdot \frac{kip}{in}}{\beta_s \cdot f_{ss.Ser1_{hs}}} - 2 \cdot d_c\right) & \text{if (MildReinfType = "Carbon Steel") + (MildReinfType = "Stainless Steel")} \\ \min\left(1.15 \cdot \frac{C_b \cdot E_f \cdot w_{max}}{f_{ss.Ser1_{hs}}} - 2.5 \cdot c_c, 0.92 \cdot \frac{C_b \cdot E_f \cdot w_{max}}{f_{ss.Ser1_{hs}}}\right) & \text{if MildReinfType = "Glass Fiber Reinforced Polymer"} \end{cases} \end{cases} \\ 0 \quad \text{otherwise} \end{cases}$$

$CriticalSpa_{BarsE.reqd} := \min(Spa_{BarsE.reqd}) = 90.1 \cdot in$ *The most critical spacing along cap sections*

$DC_{BarsE_{hs}} := \frac{s_{BarsE.prov_{hs}}}{Spa_{BarsE.reqd_{hs}}}$ $\quad \max(DC_{BarsE}) = 0.09$

$$d_{c.max_{hs}} := \begin{cases} \text{if substr(BeamType, 0, 3) = "FSB"} \\ \quad \begin{cases} dcmax \leftarrow 10000in & \text{if } f_{ss.Ser1_{hs}} \le 0 \\ \text{otherwise} \\ \quad \begin{cases} \zeta_{hs} \leftarrow \frac{h - kk_{hs} \cdot d_s}{d_s - kk_{hs} \cdot d_s} \\ dcmax \leftarrow \frac{C_b \cdot E_f \cdot w_{max}}{2 \cdot f_{ss.Ser1_{hs}} \cdot \zeta_{hs}} \end{cases} \end{cases} \\ dcmax \leftarrow 0 \quad \text{otherwise} \\ dcmax \end{cases}$$

limiting d.c value

[AASHTO GFRP Eq. 2.6.7-2]

$\min(d_{c.max}) = 10.31 \cdot in$

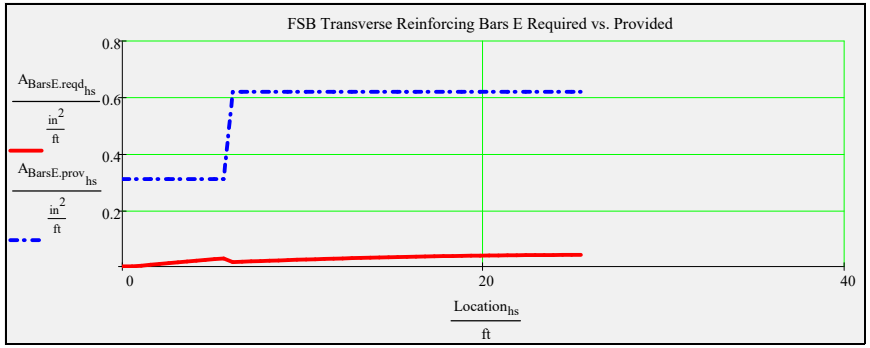
$d_c = 1.81 \cdot in$

$f_{s.max} := \text{if}(f_y < 75ksi, 0.8 \cdot f_y, 0.6 \cdot f_y) = 48 \cdot ksi$ *[SDG 4.1.8]*

$$Check_{FSB.BarsE} := \begin{cases} \text{if substr(BeamType, 0, 3) = "FSB"} \\ \quad \begin{cases} \text{"OK"} & \text{if } \max(DC_{BarsE}) \le 1 \wedge \max(f_{ss.Ser1}) \le f_{s.max} \wedge \text{(MildReinfType = "Carbon Steel") + (MildReinfType} \\ \text{"OK"} & \text{if } \max(DC_{BarsE}) \le 1 \wedge d_c \le \min(d_{c.max}) \wedge \text{MildReinfType = "Glass Fiber Reinforced Polymer"} \\ \text{"No Good"} & \text{otherwise} \end{cases} \\ \text{"N/A"} \quad \text{otherwise} \end{cases}$$

$Check_{FSB.BarsE} = \text{"OK"}$

$$A_{BarsE.reqd_{hs}} := \text{if}\left(\text{substr}(BeamType, 0, 3) = \text{"FSB"}, \frac{A_{BarsE}}{Spa_{BarsE.reqd_{hs}}}, 0\right)$$



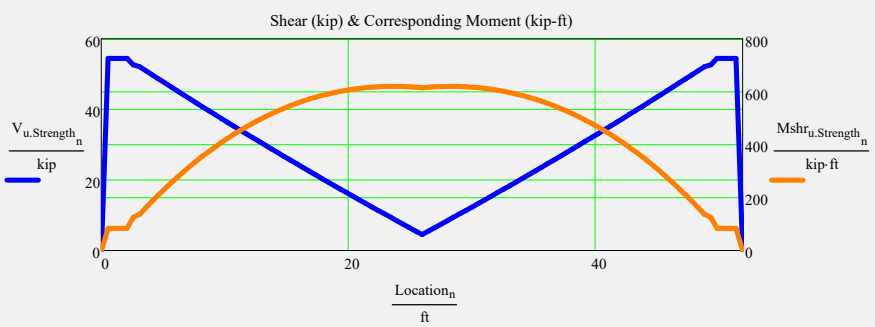
FSB only - Design Check of Transverse Reinforcing Bars E

Shear Loads

Shear and Corresponding Moments:

Calculations use the greater of Strength I or Strength II Limit State

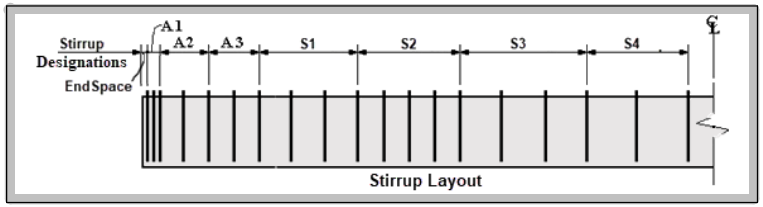
$$\begin{pmatrix} \max(V_{pos.StrengthI}) \\ \max(V_{pos.StrengthII}) \end{pmatrix} = \begin{pmatrix} 57 \\ 39 \end{pmatrix} \cdot kip \quad \begin{pmatrix} \text{"HL93"} \\ \text{"Permit Vehicle"} \end{pmatrix}$$



$\max(V_{u.Strength}) = 54.5 \cdot kip$ ControllingShearLimitState = "Strength I" $\max(Mshr_{u.Strength}) = 621 \cdot kip \cdot ft$

Shear Resistance

Stirrup Size and Spacings (values are for one half of the beam)



End Stirrup Space	2.5	in	*Distance from end of beam to the centerline of first stirrup or stirrup bundle.*
GFRP Stirrup Bar #	0	*used for GFRP Bars*	

Help on Stirrup Design | Help on Stirrup Design Iter. | Table of FDOT Standard Prestressed Beam End Reinforcement | Reset Load Data

Stirrup Design - see Stirrup Layout

Designation	A1	A2	A3	S1	S2	S3	S4
Spacing (in)	3	7	0	12	24	0	24
# of Spaces	3	1	0	4	10	0	Calculated
Area (in^2)	0.60	0.80	0.0	1.00	1.20	0.0	1.20
Interface Factor	0.33	1.0	0.0	0.80	0.67	0.0	0.67

Save and Initialize Data

Shear Demand/Capacity and Longitudinal Steel Demand/Capacity

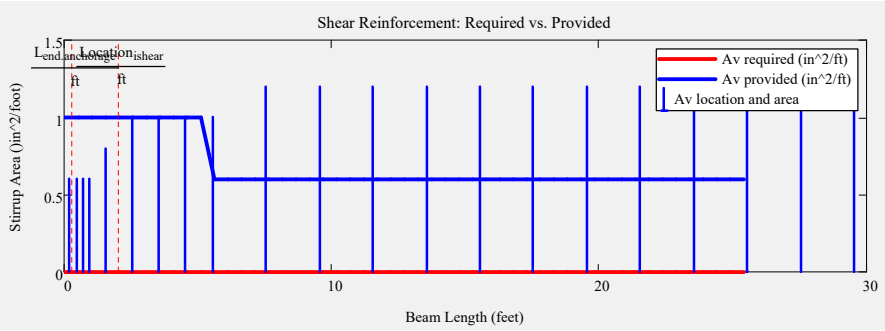
CheckShearCapacity = "OK"

CheckStirArea = "OK"

CheckMaxShearStress = "OK"

CheckMinStirArea = "OK"

CheckMaxStirSpacing = "OK"



$Location_{ishear} = 2.06\ ft$

Save Data File

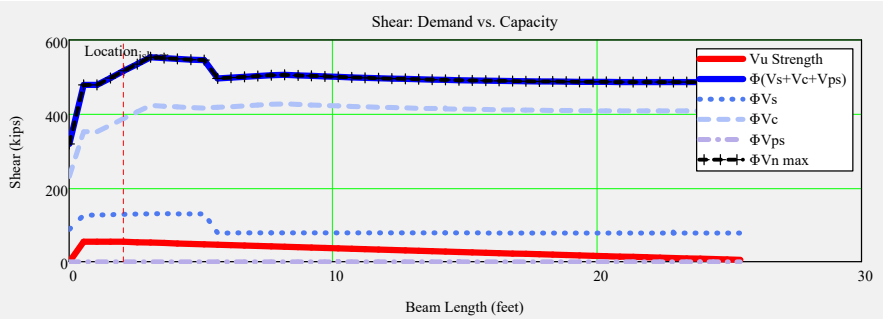
Use current input file

File Name Able Canal - Spans 2-4 - Exterior - Ped. Load.dat

Save Data

DataFileFolder = "C:\Users\jjohnson2\Desktop\Able Canal\Superstructure\PrestressedBeamV6.1\DataFiles-FSB"

Note: Select an output folder by using the "Change Folder" option on page 1.



$fCheckD/CShear = \begin{pmatrix} \text{"CheckType"} & \text{"Location(ft)"} & \text{"Vu (kip)"} & \text{"Vr (kip)"} \\ \text{"Shear"} & 2.06 & 54.52 & 517.12 \end{pmatrix}$

$\max(D/C_{shear}) = 0.11$

Botom Longitudinal Reinf. (LRFD 5.7.3.5)

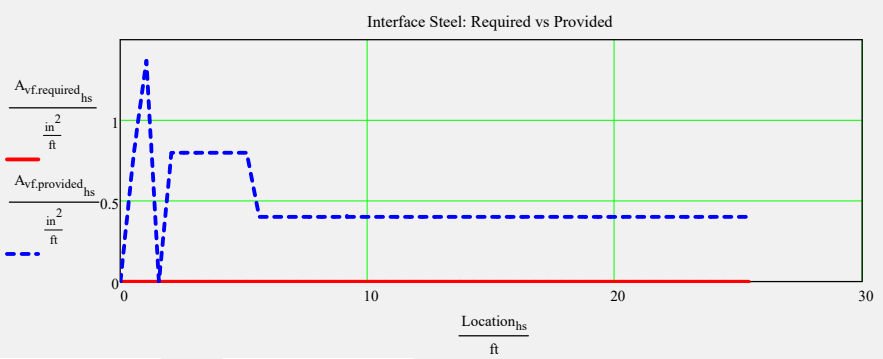
Longitutinal Reinforcement:

$CheckD/CLong = \begin{pmatrix} \text{"CheckType"} & \text{"Location(ft)"} & \text{"V long Req'd (kip)"} & \text{"V long Prov (kip)"} \\ \text{"Longitudinal Reinf."} & 22.4 & 583.4 & 1620.4 \end{pmatrix}$

$\max(D/C_{LongReinf}) = 0.36$ CheckLongReinf = "OK" *(LRFD 6.7.3.5)*

Interface Shear Reinforcement

Interface Shear:



$TotalA_{vf.required} = 0 \cdot in^2$ $TotalA_{vf.provided} = 12.01 \cdot in^2$ $DC_{InterfaceReinf} = 0$ CheckInterfaceReinf = "OK"

Typically shear reinforcement is extended up into the deck slab.
These calculations are based on shear reinforcement functioning as interface reinforcing.
The interface_factor *can be used to adjust this assumption.*

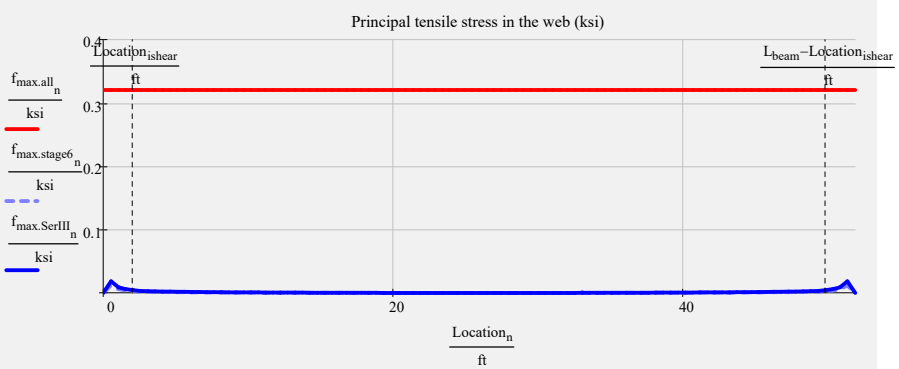
$\max(A_{vf.min.required}) = 0.000 \cdot \frac{in^2}{ft}$ $\max(A_{vf.des}) = 0.000 \cdot \frac{in^2}{ft}$

If max(Avf.min) or max(Avf.des) is greater than 0 in²/ft, interface reinforcement is required.

CheckInterfaceSpacing = "OK"

Pricipal Tensile Stresses in Webs - LRFD 5.9.2.3.3 and Splitting Resistance

Principal Stresses:



Crital Location for Principal Stress: $Location_{ishear} = 24.8 \cdot in$ $\max(DC_f_{max.web}) = 0.1$ $Check_f_{max.web} = "OK"$

Note: The check for pricipal tensile stresses in webs is only required for pretensioned girders with a compressive strength of concrete for use in design greater than 10 ksi per AASHTO BDS Article 5.9.2.3.3. The check is displayed for all concrete strengths for consideration when optimizing the strands design.

Splitting Reinforcement:

FDOT Splitting Rein Demand/Capacity Ratio and Check for FDOT Standard Beam End Reinforcement

$P_{r.standard} = 1460 \cdot kip$ $P_{prestress} = 1230 \cdot kip$ $D/C_{splitting.FDOT} = 0.84$

$CheckD/C_{splitting} = "OK"$

Confinement Bars:

$TotalNoConfineBars = 10$

Summary of Design Checks

$check_1 := "OK"$

$check_2 := CheckDeflectionOrSpanToDepth = "OK"$

$check_3 := CheckStrandFit = "OK"$

$check_4 := Check_f_{pt} = "OK"$

$check_5 := Check_f_{pe} = "OK"$

$check_6 := Check_f_{tension.rel} = "OK"$

$check_7 := Check_f_{comp.rel} = "OK"$

$check_8 := Check_f_{tension.ServiceIII} = "OK"$

$check_9 := Check_f_{comp.ServiceIa} = "OK"$

$check_{10} := Check_f_{comp.ServiceIb} = "OK"$

$check_{11} := Check_f_{comp.FatigueI} = "OK"$

$check_{12} := CheckMomentCapacity = "OK"$

$check_{13} := CheckMaxCapacity = "OK"$

$check_{14} := CheckStirArea = "OK"$

$check_{15} := CheckShearCapacity = "OK"$

$check_{16} := CheckMinStirArea = "OK"$

$check_{17} := CheckMaxStirSpacing = "OK"$

$check_{18} := CheckLongReinf = "OK"$

$check_{19} := CheckInterfaceSpacing = "OK"$

$check_{20} := CheckInterfaceReinf = "OK"$

$check_{21} := "OK"$

$check_{22} := CheckD/C_{splitting} = "OK"$

$check_{23} := CheckPattern_0 = "OK"$

$check_{24} := CheckPattern_1 = "OK"$

$check_{25} := CheckPattern_2 = "OK"$

$check_{26} := CheckPattern_3 = "OK"$

$check_{27} := CheckPattern_4 = "OK"$

$check_{28} := CheckWebPattern_0 = "OK"$

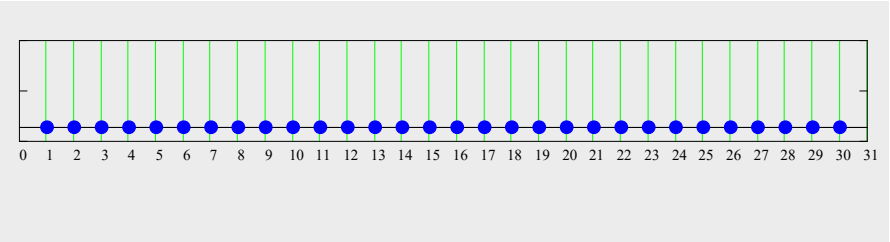
$check_{29} := CheckWebPattern_1 = "OK"$

$check_{30} := CheckBeamEndTopLongReinf = "OK"$

All Beam Checks

$check^T =$

	0	1	2	3	4	5	6	7	8	9	10
0	0	"OK"	"OK"	"OK"	"OK"	"OK"	"OK"	"OK"	"OK"	"OK"	...



$TotalCheck = "OK"$

Load Rating Analysis: Load and Resistance Factor Rating (LRFR)

ref.: Maintenance Office FDOT Bridge Load Rating Manual

Load Rating Computations

$$LRFR_{loadrating} = \begin{array}{c} \text{Moment (Strength) or Stress (Service)} \quad \text{Shear (Strength)} \\ \begin{pmatrix} 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 \\ 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 1.52 \\ 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 1.52 \\ 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 \\ 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 \\ 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 1.19 \times 10^{4} & 7.15 \times 10^{5} & 1.52 \end{pmatrix} \end{array} \quad \begin{pmatrix} \text{HL-93} \\ \text{HL-93} \\ \text{HL-93} \\ \text{HL-93*} \\ \text{Permit**} \end{pmatrix}$$

Notes:
** Not applicable for prestressed beams in good condition.*
*** FDOT Permit Rating truck is the FL120 .*
"Dim(ft)"*is measured from the centerline of bearings*

Longitudinal Reinforcement Check:

$DC_{LongReinf.HL93} = 0.36$ $DC_{LongReinf.Permit} = 0.28$ $CheckLongReinf_{loadrating} = \text{"OK"}$

QUICK CHECK: Factored Components of the Rating Factor Equation for the Design Operating Moment and Shear Ratings

$$HL93_{Operating.Moment} = \begin{pmatrix} \text{"}\varphi C\text{"} & 1471 & \text{"factored resistance (kip-ft)...."} \\ \text{"}\gamma DL\text{"} & 499 & \text{"factored dead load (kip-ft)...."} \\ \text{"}\gamma LL\text{"} & 179 & \text{"factored live load (kip-ft)....."} \\ \text{"RF"} & 5.41 & \text{"HL93 operating rating factor"} \end{pmatrix}$$

$$HL93_{Operating.Shear} = \begin{pmatrix} \text{"}\varphi C\text{"} & 517.1 & \text{"factored resistance (kip)....."} \\ \text{"}\gamma DL\text{"} & 37.3 & \text{"factored dead load (kip)....."} \\ \text{"}\gamma LL\text{"} & 13.3 & \text{"factored live load (kip)....."} \\ \text{"RF"} & 36.05 & \text{"HL93 operating rating factor"} \end{pmatrix}$$

FDOT LRFD Prestressed Beam Program v6.1

State Structures Design Office

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Select Beam Data Folder

- I-Beam
- FSB
- FUB
- FlatSlab
- InvertedT
- DoubleT

Category and Beams

I-Beams: *Florida I-Beams (FIBs) and formerly used Florida Bulb Tee and AASHTO Beams*
Slab Beams: *Florida Slab Beams (FSBs)*
U-Beams: *Florida U-Beams (FUBs)*
Flat Slab Previously used Florida Flat Slab Beams, superceded by FSBs
Inverted Tee: *Former FDOT Standard Inverted Tee Beams*
Double Tee: *Former FDOT Standard Double Tee Beams from the 1990's*

Data Files Folder

C:\Users\jjohnson2\Desktop\Able Canal\Superstructure\PrestressedBeamV6.1\DataFiles-FSB

Change Folder

Open Existing Data File (required). To save New Data Files, enter new data name at the end of Section IX.

- Able Canal - Span 1 _6 - Exterior.dat
- Able Canal - Spans 2 though 4 - Interior.dat
- Able Canal - Spans 2 through 4 - Exterior.dat
- Able Canal - Spans 2 through 4 - Interior.dat
- Able Canal - Spans 2-4 - Exterior - H10 Load.dat
- Able Canal - Spans 2-4 - Exterior - Ped. Load.dat
- Able Canal - Spans 2-4 - Interior - H10 Load.dat
- Able Canal - Spans 2-4 - Interior - Ped. Load.dat
- FSB-30ftSpan-12inx53inBeam.dat

Refresh List

Open File

Reset All Data

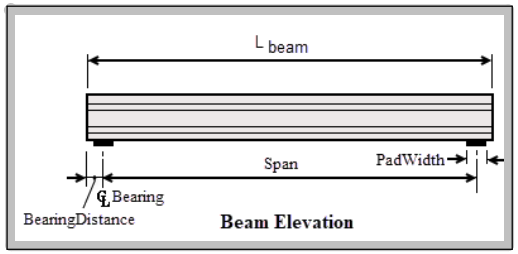
InputDataFile = "Able Canal - Spans 2-4 - Interior - H10 Load.dat"

Project Information

Project Information

Project Name	Able Canal Pathway - Spans 2 through 4 - Interior - H10 Load		
Project No.	435351-2-38-01		
Designed by	JHJ	*Date*	02/2022
Checked by		*Date*	
Structure Name	53ft Span - 15"x54"		
Structure No.	XXXXXX		

Plan, Elevation, and Cross Section Data



Note: All dimensions shown in Beam Elevation measured along centerline of beam (requiring adjustment for a skew)

Note: The top of the precast beam is the location of the origin for the coordinate system.

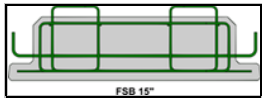
Superstructure Data

InputDataFile = "Able Canal - Spans 2-4 - Interior - H10 Load.dat"

Beam Type: FSB12, FSB15, FSB18 — *Beam Types are the designations found in FDOT standards. The user can also create a coordinate file for a custom shape. Top of the beam is at the y=0 ordinate.*

BMPfile := concat(substr(inpBeamType, 0, strlen(inpBeamType)), ".bmp")

BMPfile = "FSB15.bmp"



L_{beam}	51.833	ft	*See Beam Elevation above.*
Bearing Distance	6.5	in	*See Beam Elevation above.*
Pad Width	8	in	*Width of the bearing pad, see Beam Elevation above.*
Beam Spacing	4.583	ft	*Measured from beam centerline to centerline.*
Overhang	0	ft	*Measured from centerline of exterior beam.*
Deck Thickness	6	in	*Not including sacrificial thickness.*
$t_{sacrificial}$	0	in	*Sacrificial thickness cast with the deck (used for DL only, not section properties).* *(0" for* BridgeLengths ≤ 100·ft, *SDG 4.2.2)*
d_e	1.4167	ft	*Front face of barrier to centerline of exterior beam (3 ft max).* *(LRFD 4.6.2.2.1)*
Beam Position	Interior / Exterior		*Use either* "interior" *or* "exterior".
$Weight_{future.ws}$	0.00	kip/ft^2	*Future wearing surface.* (0.015ksf *for* BridgeLengths ≤ 100·ft, *SDG Table 2.1)*
$Weight_{barrier}$	0.080	kip/ft	*Weight of single barrier.* *(0.030 klf for Pedestrian Railing, SDG Table 2-2.1 & 0.050 klf for 8" x 6" Concrete Curb)*
Number of Barriers	2		*Number of barriers in x-section (multiplies single barrier weight).*
Number of Beams	3		*Number of beams in the span cross section.* *(LRFD 4.6.2.2.1)*
Skew	0	deg	*measured from the perpendicular to the longitudinal axis in bridge plan view.*

Dimensions Specific to Certain Beam Categories

I-Beams, U-Beams and Inverted-T Beams

$h_{buildup}$		in	*Buildup or haunch is the concrete between the bottom of deck and top of beam.*

Slab-Beams (Florida Slab Beams, Flat Slab Beams)

Slab Beam Width	54	in	*Width of the slab unit (not including Gap width)*
Slab Beam Thickness	15	in	*Thickness of the slab unit.*
$t_{deck.delta}$	1.0443	in	*maximum additional deck thickness over support to accommodate camber, used for additional DL only.*
Gap	1.016	in	*Gap distance between slab beams.(Beam Spacing - Slab Beam Width)*

Double Tee Beams

Width Double-T		ft	*Width of the Double-T unit.*
Depth flange		in	*Depth of the flange of the Double-T unit, See Beam Cross Section above.*

Concrete Material Properties

FDOT Environmental Classification: Slightly / Moderately / Extremely

Environmental Classification determines the Allowable Tension Stress

f`c of deck or WS	4.5	ksi	*Strength of deck or wearing surface concrete.*
f`c of beam or slab	8.5	ksi	*28 day concrete strength of beam or slab.*
f`ci of beam or slab	6	ksi	*Release concrete strength of beam or slab.*

Help - Conc. Strength

Permit Truck Axle Loads and Spacings

Default value for the permit truck is the FL-120 truck

Reset Permit Truck Data

Number of Permit Axles 3 — *Number of wheel loads that comprise the permit truck*

Permit Uniform LL 0 kip/ft — *Permit Uniform Live Load coincident with truck load*

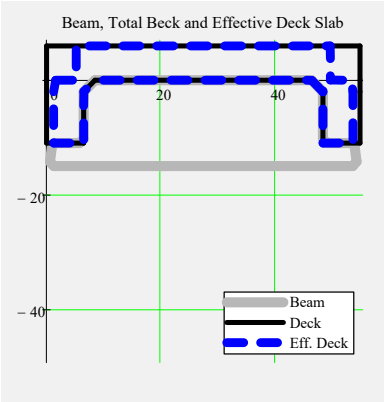
Help - Permit Truck

	Axle Number										
	1	2	3	4	5	6	7	8	9	10	11
Load (kips)	0.01	0.01	0.01	0	0	0	0	0	0	0	0
Spacing (ft)	n/a	14	14	0	0	0	0	0	0	0	0

Custom Beam Coordinates, Initialize Data, Section Properties and Dead Loads

LRFD Live Load Distribution Factors, User Defined Live Load Distribution Factors and Section Properties

Section View: Beam, Deck and Effective Deck Slab



$g_{mom} = 0.69$

$g_{shear} = 0.69$

BeamType = "FSB15"

$t_{deck} = 6 \cdot in$

$h_{buildup} = 0 \cdot in$

$h_{beam} = 15 \cdot in$

BeamSpacing = $54 \cdot in$

$b_e = 55.02 \cdot in$ *effective slab width*

Additional Dead Load Data:

Uniform Dead Loads

Additional Uniform Noncomposite DL	0.007	$\frac{kip}{ft}$
Additional Uniform Composite DL	0	$\frac{kip}{ft}$

$w_{deck} = 0.50 \cdot \frac{kip}{ft}$ $w_{beam} = 0.7 \cdot \frac{kip}{ft}$ $w_{forms} = 0.00 \cdot \frac{kip}{ft}$

$w_{future.ws} = 0 \cdot \frac{kip}{ft}$ $w_{barrier} = 0.05 \cdot \frac{kip}{ft}$

Diaphragms (Point) Dead Loads $L_{beam} = 51.83$ ft

End Diaphragms or Point Loads over bearing included in bearing reaction calculation only.
Intermediate Diaphragms or Point Loads included in shear, moment, and bearing reaction calculations

Diaphragm (Point) Dead Loads and Location

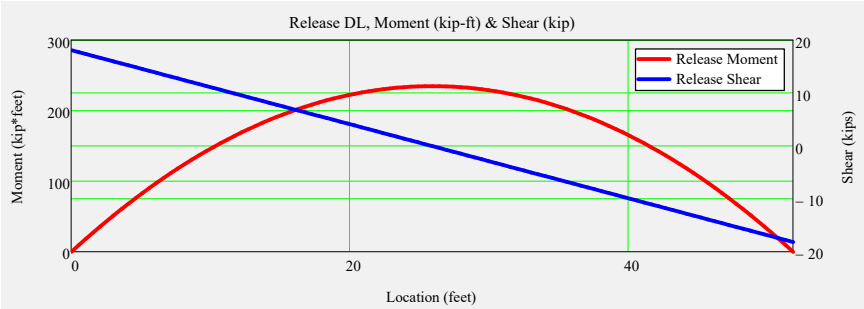
	Begin	Int. 1	Int. 2	Int. 3	End
Load (kips)	0	0	0	0	0
Distance (ft)	0	0	0	0	0

Distance measured from CL Bearing at begin bridge

Dead Loads, DL Moment, DL Shears

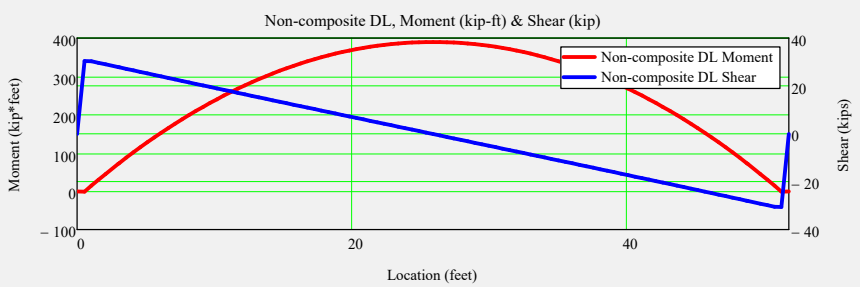
Release Dead Load Moments and Shears

$L_{span} = 50.75\ ft$



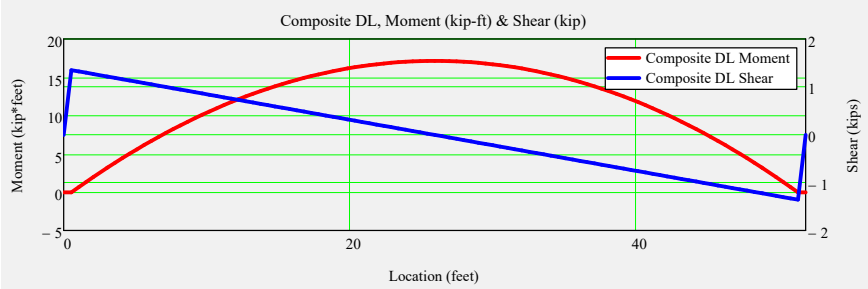
$\max(V_{release}) = 18.1 \cdot kip$ $\max(M_{release}) = 235.0 \cdot kip \cdot ft$ $\min(V_{release}) = -18.1 \cdot kip$

Noncomposite Dead Load Moments and Shears



$\max(V_{dl.non.comp}) = 30.5 \cdot kip$ $\max(M_{dl.non.comp}) = 390.2 \cdot kip \cdot ft$ $\min(V_{dl.non.comp}) = -30.5 \cdot kip$

Composite Dead Load Moments and Shear

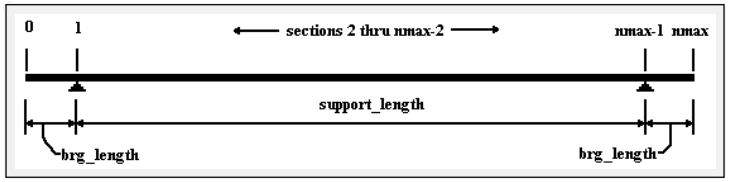


$\max(V_{dl.comp}) = 1.4 \cdot kip$ $\max(M_{dl.comp}) = 17.2 \cdot kip \cdot ft$ $\min(V_{dl.comp}) = -1.4 \cdot kip$

Live Load Moments and Shears

$$\text{WRITEPRN("coord.dat")} := \frac{Coord_{beam}}{ft}$$

Live Load Moments and Shears



$L_{c.c.bearings} := L_{beam} - 2 \cdot BearingDistance$ *Distance between the center line of bearings*

$Location_1 := BearingDistance$

$Location_0 := 0 \cdot ft$ *Location vector defines the x location of the sections investigated relative to the start of the beam*

$Location_{ns} := \frac{L_{c.c.bearings}}{nmax - 2} \cdot (ns - 1) + Location_1$

$Location_{nmax} := L_{beam}$ *The Location vector need not be in even increments*

$a_n := Location_n - BearingDistance$ *a vector gives the section x coordinate relative to the center line of the first bearing*

$b_n := L_{c.c.bearings} - a_n$ *b vector gives the distance from the section to the center line of the last bearing*

$$P := \begin{pmatrix} 4 & 16 & 0 \\ 16 & 4 & 0 \\ 0 & 0 & 0 \end{pmatrix} \cdot kip$$

P vector set contains the magnitudes of the system of concentrated loads (HS20 is default)

$$d := \begin{pmatrix} 0 & 0 & 0 \\ 14 & 14 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 0 \end{pmatrix} \cdot ft$$

d vector set contains the spacings between the concentrated loads. The first and last values are place holders and should always be zero

$PermitAxleLoad^T = (0.01 \quad 0.01 \quad 0.01) \cdot kip$

$PermitAxleSpacing^T = (0 \quad 14 \quad 14 \quad 0)\ ft$

$$FatigueAxleLoad := \begin{pmatrix} 0 \\ 0 \\ 0 \end{pmatrix} \cdot kip$$

$$FatigueAxleSpacing := \begin{pmatrix} 0 \\ 0 \\ 0 \\ 0 \end{pmatrix} \cdot ft$$

$P^{\langle 3 \rangle} := \text{PermitAxleLoad}$ $\quad P^{\langle 4 \rangle} := \text{reverse(PermitAxleLoad)}$

$d^{\langle 3 \rangle} := \text{PermitAxleSpacing}$ $\quad d^{\langle 4 \rangle} := \text{reverse(PermitAxleSpacing)}$

$P^{\langle 5 \rangle} := \text{FatigueAxleLoad}$ $\quad P^{\langle 6 \rangle} := \text{reverse(FatigueAxleLoad)}$

$d^{\langle 5 \rangle} := \text{FatigueAxleSpacing}$ $\quad d^{\langle 6 \rangle} := \text{reverse(FatigueAxleSpacing)}$

$$P = \begin{pmatrix} 4 & 16 & 0 & 0.01 & 0.01 & 0 & 0 \\ 16 & 4 & 0 & 0.01 & 0.01 & 0 & 0 \\ 0 & 0 & 0 & 0.01 & 0.01 & 0 & 0 \end{pmatrix} \cdot kip \qquad d = \begin{pmatrix} 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 14 & 14 & 0 & 14 & 14 & 0 & 0 \\ 0 & 0 & 0 & 14 & 14 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 \end{pmatrix} ft$$

$NumberOfTrucks := cols(P) = 7$ *number of trucks*

$nt := 0 .. (NumberOfTrucks - 1)$ *nt vector of length equal to number of Trucks*

$$TruckLength_{nt} := \sum_{qt} \left(d^{\langle nt \rangle}\right)_{qt}$$ *total length of the concentrated load system*

$$inc_{nt} := floor\left[\frac{300}{\left(\frac{TruckLength_{nt}}{L_{beam}}\right) + 1}\right]$$ *number of even increments that the load system will use to advance across the span*

$$add_inc_{nt} := ceil\left(\frac{TruckLength_{nt} \cdot inc_{nt}}{L_{beam}}\right)$$ *additional increments required to get the load system off of the span*

$lastp_{nt} := 199$ *value of the last increment*

$$step_{nt} := \frac{L_{beam} + TruckLength_{nt} - 0.0000001 \cdot ft}{lastp_{nt}}$$ *distance between increments*

Remove units for calculations

$a := \frac{a}{ft} \qquad b := \frac{b}{ft} \qquad P := \frac{P}{kip} \qquad d := \frac{d}{ft}$

Functions used to determine moments and shears

$$fM_{int}(P, a, b, loc, brg, sl, M, n, z) := \begin{cases} Ans \leftarrow \frac{P_z \cdot a_n \cdot \left[sl - \left(loc_z - brg\right)\right]}{sl} + M_n & \text{if } a_n < \left(loc_z - brg\right) \\ Ans \leftarrow \frac{P_z \cdot b_n \cdot \left(loc_z - brg\right)}{sl} + M_n & \text{otherwise} \end{cases}$$

$$fV_{int}(P, loc, brg, sl, SZ, n, z) := \begin{cases} Ans \leftarrow \frac{P_z \cdot \left[sl - \left(loc_z - brg\right)\right]}{sl} + SZ_n & \text{if } a_n < \left(loc_z - brg\right) \\ Ans \leftarrow \frac{P_z \cdot \left(brg - loc_z\right)}{sl} + SZ_n & \text{otherwise} \end{cases}$$

$$fM_{start}(P, a, b, loc, brg, sl, M, n, z) := \begin{cases} Ans \leftarrow M_n + \left(loc_z - brg\right) \cdot P_z & \text{if } (n > 0) \cdot (n < 2) \\ Ans \leftarrow \left(loc_z - brg\right) \cdot P_z \cdot \frac{b_n}{sl} + M_n & \text{otherwise} \end{cases}$$

$$fV_{start}(P, loc, brg, sl, SZ, n, z) := Ans \leftarrow \left[P_z \cdot \frac{\left(brg - loc_z\right) + sl}{sl} - P_z\right] + SZ_n$$

$$fM_{end}(P, a, b, loc, brg, sl, M, n, z, k) := \begin{cases} Ans \leftarrow P_z \cdot \left[(sl + brg) - loc_z\right] + M_n & \text{if } [n > (k - 2)] \cdot (n < k) \\ Ans \leftarrow P_z \cdot \left[(sl + brg) - loc_z\right] \cdot \frac{sl - b_n}{sl} + M_n & \text{otherwise} \end{cases}$$

$$f_{end}(P, loc, brg, sl, SZ, n, z, k) := Ans \leftarrow P_z \cdot \left(1 - \frac{loc_z - brg}{sl}\right) + SZ_n$$

$$M_{ll}(a, b, L, k, brg, P, d, lp, nl, st) := \begin{array}{l} sl \leftarrow L - 2 \cdot brg \\ \text{for } n \in 0 .. k \\ \quad \text{for } j \in 0 .. 5 \\ \quad\quad R_{n,j} \leftarrow 0 \\ \text{for } q \in 0 .. lp \\ \quad \text{for } n \in 0 .. k \\ \quad\quad M_n \leftarrow 0 \\ \quad\quad SZ_n \leftarrow 0 \\ \quad al \leftarrow q \cdot st \\ \quad \text{for } z \in 0 .. (nl - 1) \\ \quad\quad loc_z \leftarrow al - \sum_{t=0}^{z} d_t \\ \quad\quad \text{for } n \in 1 .. (k - 1) \qquad \text{if } \left(loc_z < brg\right) \cdot \left(loc_z \geq 0\right) \\ \quad\quad\quad M_n \leftarrow fM_{start}(P, a, b, loc, brg, sl, M, n, z) \\ \quad\quad\quad SZ_n \leftarrow fV_{start}(P, loc, brg, sl, SZ, n, z) \\ \quad\quad \text{for } n \in 1 .. (k - 1) \qquad \text{if } \left(loc_z > brg\right) \cdot \left[loc_z \leq (sl + brg)\right] \\ \quad\quad\quad M_n \leftarrow fM_{int}(P, a, b, loc, brg, sl, M, n, z) \\ \quad\quad\quad SZ_n \leftarrow fV_{int}(P, loc, brg, sl, SZ, n, z) \\ \quad\quad \text{for } n \in 2 .. (k - 1) \qquad \text{if } \left[loc_z > (sl + brg)\right] \cdot \left(loc_z \leq L\right) \end{array}$$

```
    M_n ← fM_end(P, a, b, loc, brg, sl, M, n, z, k)
    SZ_n ← f_end(P, loc, brg, sl, SZ, n, z, k)
  for n ∈ 1 .. k − 1
    R_n,0 ← M_n  if R_n,0 < M_n
    R_n,1 ← M_n  if R_n,1 > M_n
    if R_n,2 < SZ_n
      R_n,2 ← SZ_n
      R_n,3 ← M_n
    if R_n,4 > SZ_n
      R_n,4 ← SZ_n
      R_n,5 ← M_n
R
```

$$M_{truck_{nt}} := M_{ll}\left[a, b, \left(\frac{L_{beam}}{ft}\right), nmax, \left(\frac{BearingDistance}{ft}\right), P^{\langle nt \rangle}, d^{\langle nt \rangle}, lastp_{nt}, rows(P), \left(\frac{step_{nt}}{ft}\right)\right]$$

Graphs of the Lane, Truck, Permit Truck, Strength I, and Strength II Live Loads

The results are contained in a matrix with each row associated with the same numbered section and the columns containing the following data: *Column 0 --- the maximum positive moment*
Column 1 --- the maximum negative moment (if any depending on the Bearing Distance)
Column 2 --- maximum positive shear
Column 3 --- moment associated with the maximum positive shear
Column 4 --- maximum negative shear
Column 5 --- moment associated with the maximum negative shear

```
compl(A, ns, ne, col) :=
  res ← (A_ns)^⟨col⟩
  for j ∈ (ns + 1) .. ne
    res ← augment[res, (A_j)^⟨col⟩]
  res
```

```
sift(AA, ns, ne, nax) :=
  for ik ∈ 0 .. nax
    test ← 0
    res_ik ← 0
    for j ∈ ns .. ne
      if AA_ik,j > test
        test ← AA_ik,j
        res_ik ← j
  res
```

Design Truck Maximum Moments, Maximum Shears, and Moments associated with Maximum Shears

$pos_m_tot := compl(M_{truck}, 0, 2, 0)$

$M_{truck.pos_n} := max\left[\left(pos_m_tot^T\right)^{\langle n \rangle}\right] \cdot kip \cdot ft$

$neg_m_tot := compl(M_{truck}, 0, 2, 1)$

$M_{truck.neg_n} := min\left[\left(neg_m_tot^T\right)^{\langle n \rangle}\right] \cdot kip \cdot ft$

$pos_v_tot := compl(M_{truck}, 0, 2, 2)$

$pos_vm_tot := compl(M_{truck}, 0, 2, 3)$

$pos_v_index := sift[(pos_v_tot), 0, 2, nmax]$

$V_{truck.pos_n} := pos_v_tot_{n, pos_v_index_n} \cdot kip$

$Mshr_{truck.pos_n} := pos_vm_tot_{n, pos_v_index_n} \cdot kip \cdot ft$

$neg_v_tot := compl(M_{truck}, 0, 2, 4)$

$neg_vm_tot := compl(M_{truck}, 0, 2, 5)$

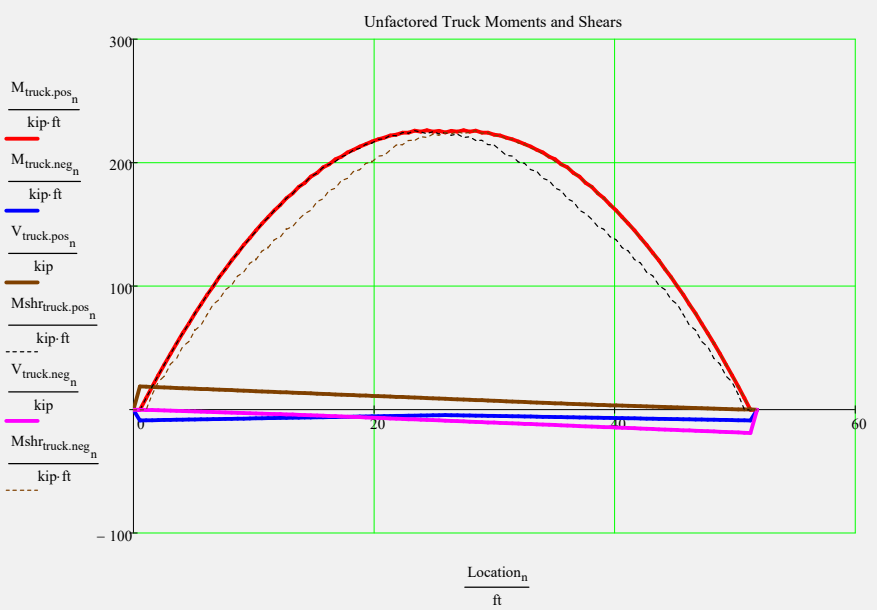
$neg_v_index := sift[neg_v_tot \cdot (-1), 0, 2, nmax]$

$V_{truck.neg_n} := neg_v_tot_{n, neg_v_index_n} \cdot kip$

$Mshr_{truck.neg_n} := neg_vm_tot_{n, neg_v_index_n} \cdot kip \cdot ft$

$pos_m_fatigue := compl(M_{truck}, 5, 6, 0)$

$M_{fatiguetruck.pos_n} := max\left[\left(pos_m_fatigue^T\right)^{\langle n \rangle}\right] \cdot kip \cdot ft$



$\max(M_{truck.pos}) = 226.4 \cdot kip \cdot ft$ $\quad$ $\min(M_{truck.neg}) = -8.7 \cdot kip \cdot ft$

$\max(V_{truck.pos}) = 18.9 \cdot kip$ $\quad$ $\max(Mshr_{truck.pos}) = 226 \cdot kip \cdot ft$

Lane Load Maximum Moments, Maximum Shears, and Moments associated with Maximum Shears

$$M_{lane.pos_n} := \frac{0 \cdot \frac{kip}{ft} \cdot a_n \cdot ft}{2} \cdot \left(L_{c.c.bearings} - a_n \cdot ft\right)$$

$M_{lane.pos_0} := 0 \cdot kip \cdot ft$ $\quad$ $M_{lane.pos_{nmax}} := 0 \cdot kip \cdot ft$

$$M_{lane.neg_n} := \frac{-\left(0 \cdot \frac{kip}{ft}\right) \cdot BearingDistance^2}{2}$$

$M_{lane.neg_0} := 0 \cdot kip \cdot ft$ $\quad$ $M_{lane.neg_{nmax}} := 0 \cdot kip \cdot ft$

$$V_{lane.neg_n} := \left[\frac{-0 \cdot \frac{kip}{ft} \cdot \left(a_n \cdot ft\right)^2}{2 \cdot L_{c.c.bearings}}\right]$$

$$V_{lane.pos_n} := \frac{0 \cdot \frac{kip}{ft} \cdot \left(b_n \cdot ft\right)^2}{2 \cdot L_{c.c.bearings}}$$

$V_{lane.neg_0} := 0 \cdot kip$ $\quad$ $V_{lane.neg_{nmax}} := 0 \cdot kip$ $\quad$ $V_{lane.pos_0} := 0 \cdot kip$ $\quad$ $V_{lane.pos_{nmax}} := 0 \cdot kip$

$Mshr_{lane.neg_n} := \left(-V_{lane.neg}\right)_n \cdot b_n \cdot ft$ $\quad$ $Mshr_{lane.pos_n} := V_{lane.pos_n} \cdot a_n \cdot ft$

$PermitUniformLoad := 0.0 \cdot klf$

$$M_{lane.permit.pos_n} := \frac{\left(PermitUniformLoad + 0.1 \cdot \frac{lbf}{ft}\right) \cdot a_n \cdot ft}{2} \cdot \left(L_{c.c.bearings} - a_n \cdot ft\right)$$

$M_{lane.permit.pos_0} := 0 \cdot kip \cdot ft$

$M_{lane.permit.pos_{nmax}} := 0 \cdot kip \cdot ft$

$$M_{lane.permit.neg_n} := \frac{-\left(PermitUniformLoad + 0.1 \cdot \frac{lbf}{ft}\right) \cdot BearingDistance^2}{2}$$

$M_{lane.permit.neg_0} := 0 \cdot kip \cdot ft$

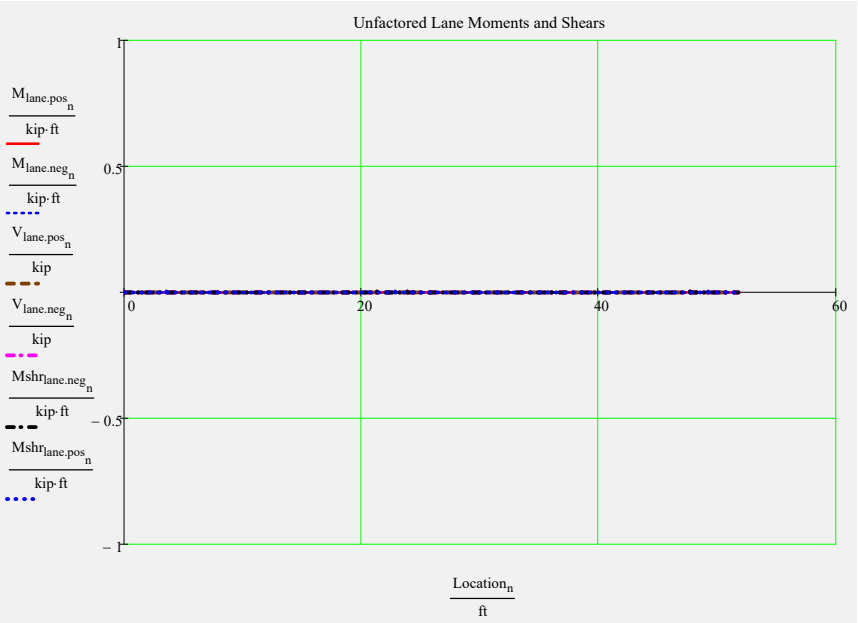
$M_{lane.permit.neg_{nmax}} := 0 \cdot kip \cdot ft$

$$V_{lane.permit.neg_n} := \left[\frac{-\left(PermitUniformLoad + 0.1 \cdot \frac{lbf}{ft}\right) \cdot \left(a_n \cdot ft\right)^2}{2 \cdot L_{c.c.bearings}}\right]$$

$$V_{lane.permit.pos_n} := \frac{\left(PermitUniformLoad + 0.1 \cdot \frac{lbf}{ft}\right) \cdot \left(b_n \cdot ft\right)^2}{2 \cdot L_{c.c.bearings}}$$

$V_{lane.permit.neg_0} := 0 \cdot kip$ $\quad$ $V_{lane.permit.neg_{nmax}} := 0 \cdot kip$ $\quad$ $V_{lane.permit.pos_0} := 0 \cdot kip$ $\quad$ $V_{lane.permit.pos_{nmax}} := 0 \cdot kip$

$Mshr_{lane.permit.neg_n} := \left(-V_{lane.permit.neg}\right)_n \cdot b_n \cdot ft$ $\quad$ $Mshr_{lane.permit.pos_n} := V_{lane.permit.pos_n} \cdot a_n \cdot ft$



$\max(M_{lane.pos}) = 0 \cdot kip \cdot ft$ $\qquad$ $\min(M_{lane.neg}) = 0 \cdot kip \cdot ft$

$\max(V_{lane.pos}) = 0 \cdot kip$ $\qquad$ $\max(Mshr_{lane.pos}) = 0 \cdot kip \cdot ft$

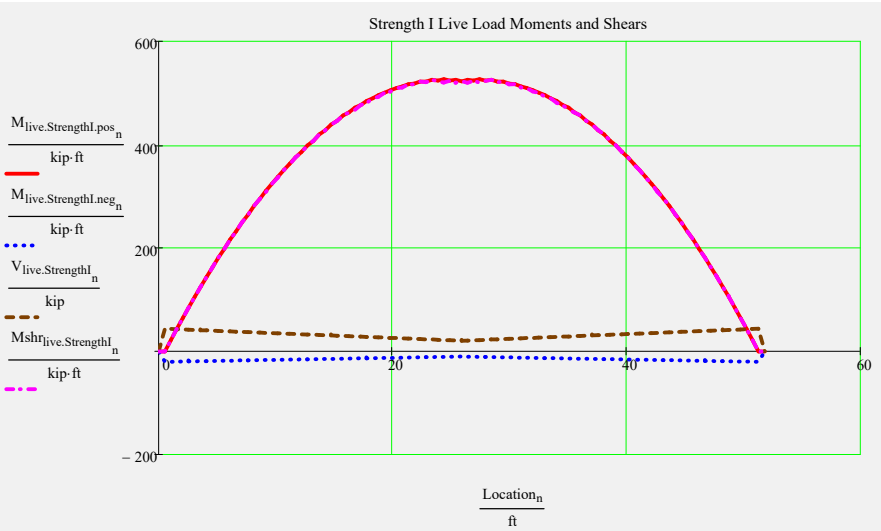
Strength I Live Load Maximum Moments, Maximum Shears, and Moments associated with Maximum Shears (Undistributed)

$M_{live.StrengthI.pos_n} := \left[M_{truck.pos_n} \cdot (1.33) + M_{lane.pos_n}\right] \cdot 1.75$ $\qquad$ $M_{live.StrengthI.neg_n} := \left[M_{truck.neg_n} \cdot (1.33) + M_{lane.neg_n}\right] \cdot 1.75$

$V_{live.StrengthIa_n} := \left[V_{truck.pos_n} \cdot (1.33) + V_{lane.pos_n}\right] \cdot 1.75$ $\qquad$ $V_{live.StrengthIb_n} := \left[V_{truck.neg_n} \cdot (1.33) + V_{lane.neg_n}\right] \cdot 1.75$

$V_{live.StrengthI_n} := \left|\text{if}\left(\left|V_{live.StrengthIa_n}\right| > \left|V_{live.StrengthIb_n}\right|, V_{live.StrengthIa_n}, V_{live.StrengthIb_n}\right)\right|$

$Mshr_{live.StrengthI_n} := \text{if}\left[\left|V_{live.StrengthIa_n}\right| > \left|V_{live.StrengthIb_n}\right|, \left(Mshr_{truck.pos_n} \cdot 1.33 + Mshr_{lane.pos_n}\right), \left(\left(Mshr_{truck.neg_n} \cdot 1.33 \ldots + Mshr_{lane.neg_n}\right)\right)\right] \cdot 1.75$



$\max(M_{live.StrengthI.pos}) = 527 \cdot kip \cdot ft$ $\qquad$ $\min(M_{live.StrengthI.neg}) = -20 \cdot kip \cdot ft$

$\max(V_{live.StrengthI}) = 44 \cdot kip$ $\qquad$ $\max(Mshr_{live.StrengthI}) = 526 \cdot kip \cdot ft$

Permit Truck Maximum Moments, Maximum Shears, and Moments associated with Maximum Shears

$ppos_m_tot := compl(M_{truck}, 3, 4, 0)$ $\qquad$ $M_{permit.pos_n} := \max\left[\left(ppos_m_tot^T\right)^{\langle n \rangle}\right] \cdot kip \cdot ft$

$pneg_m_tot := compl(M_{truck}, 3, 4, 1)$ $\qquad$ $M_{permit.neg_n} := \min\left[\left(pneg_m_tot^T\right)^{\langle n \rangle}\right] \cdot kip \cdot ft$

$ppos_v_tot := compl(M_{truck}, 3, 4, 2)$ $\qquad$ $ppos_vm_tot := compl(M_{truck}, 3, 4, 3)$

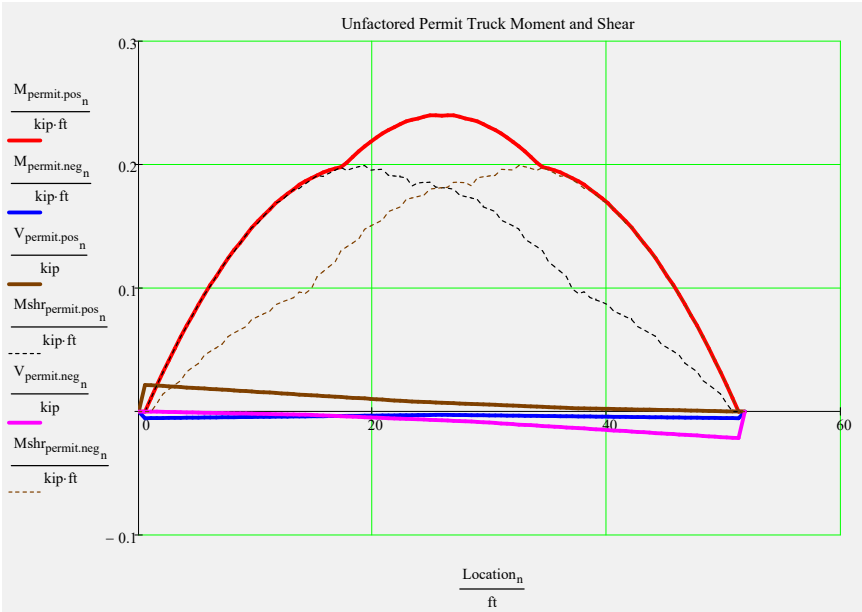
$ppos_v_index := sift[(ppos_v_tot), 0, 1, nmax]$ $\qquad$ $V_{permit.pos_n} := ppos_v_tot_{n, ppos_v_index_n} \cdot kip$

$Mshr_{permit.pos_n} := ppos_vm_tot_{n, ppos_v_index_n} \cdot kip \cdot ft$

$pneg_v_tot := compl(M_{truck}, 3, 4, 4)$ $\qquad$ $pneg_vm_tot := compl(M_{truck}, 3, 4, 5)$

$pneg_v_index := sift[pneg_v_tot \cdot (-1), 0, 1, nmax]$ $\qquad$ $V_{permit.neg_n} := pneg_v_tot_{n, pneg_v_index_n} \cdot kip$

$Mshr_{permit.neg_n} := pneg_vm_tot_{n, pneg_v_index_n} \cdot kip \cdot ft$



$\max(M_{permit.pos}) = 0 \cdot kip \cdot ft$ $\qquad$ $\min(M_{permit.neg}) = -0 \cdot kip \cdot ft$

$\max(V_{permit.pos}) = 0 \cdot kip$ $\qquad$ $\max(Mshr_{permit.pos}) = 0 \cdot kip \cdot ft$

Strength II Live Load Maximum Moments, Maximum Shears, and Moments associated with Maximum Shears (undistributed)

$M_{live.StrengthII.pos_n} := \left(1.33\, M_{permit.pos_n} + M_{lane.permit.pos_n}\right)\cdot 1.35$ *(LRFD C3.6.2.1)*

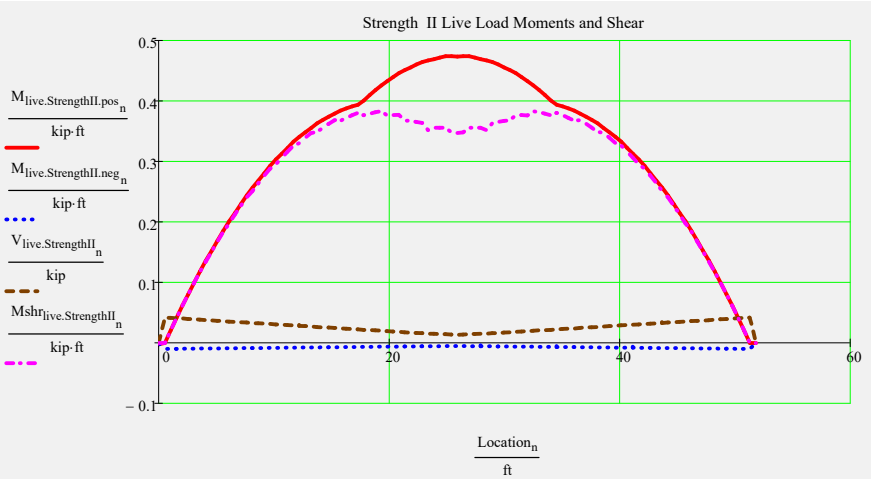
$M_{live.StrengthII.neg_n} := \left(1.33\, M_{permit.neg_n} + M_{lane.permit.neg_n}\right)\cdot 1.35$

$V_{live.StrengthIIa_n} := \left(1.33\cdot V_{permit.pos_n} + V_{lane.permit.pos_n}\right)\cdot 1.35$

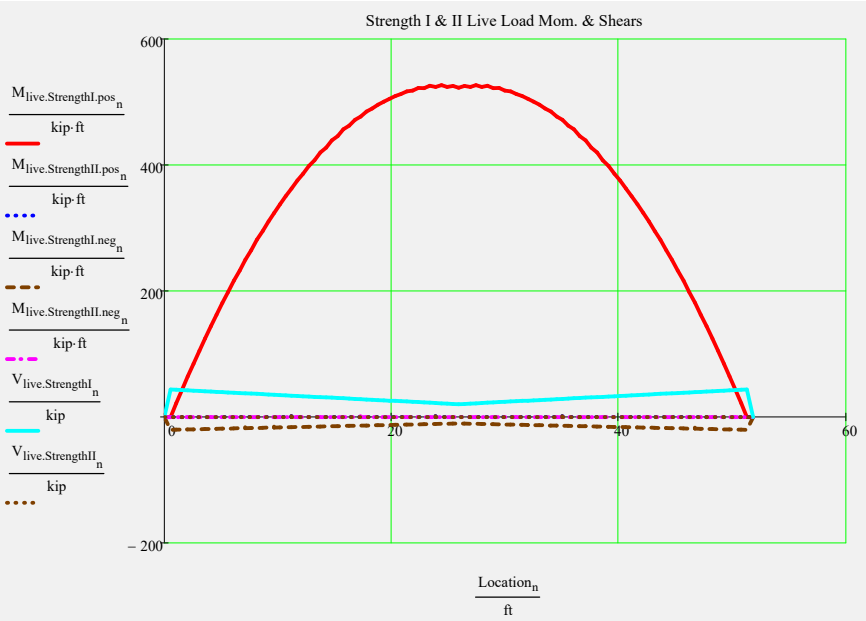
$V_{live.StrengthIIb_n} := \left(1.33\cdot V_{permit.neg_n} + V_{lane.permit.neg_n}\right)\cdot 1.35$

$V_{live.StrengthII_n} := \left|\text{if}\left(\left|V_{live.StrengthIIa_n}\right| > \left|V_{live.StrengthIIb_n}\right|, V_{live.StrengthIIa_n}, V_{live.StrengthIIb_n}\right)\right|$

$Mshr_{live.StrengthII_n} := \text{if}\left(\left|V_{live.StrengthIIa_n}\right| > \left|V_{live.StrengthIIb_n}\right|, Mshr_{permit.pos_n}\cdot 1.33 \ldots + Mshr_{lane.permit.pos_n}, Mshr_{permit.neg_n}\cdot 1.33 \ldots + Mshr_{lane.permit.neg_n}\right)\cdot 1.35$



$\max\left(M_{live.StrengthII.pos}\right) = 0\cdot kip\cdot ft$



$\max\left(M_{live.StrengthI.pos}\right) = 527\cdot kip\cdot ft$

Distributed Live Load Maximum Moments, Maximum Shears, and Moments associated with Maximum Shears

$M_{dist.live.pos_n} := \left(1.33\cdot M_{truck.pos_n} + M_{lane.pos_n}\right)\cdot g_{mom}$

$M_{dist.live.neg_n} := \left(1.33\cdot M_{truck.neg_n} + M_{lane.neg_n}\right)\cdot g_{mom}$

$V_{dist.live.pos_n} := \left(1.33\cdot V_{truck.pos_n} + V_{lane.pos_n}\right)\cdot g_{shear}$

$V_{dist.live.neg_n} := \left(1.33\cdot V_{truck.neg_n} + V_{lane.neg_n}\right)\cdot g_{shear}$

$Mshr_{dist.live.pos_n} := \left(1.33\cdot Mshr_{truck.pos_n} + Mshr_{lane.pos_n}\right)\cdot g_{mom}$

$Mshr_{dist.live.neg_n} := \left(1.33\cdot Mshr_{truck.neg_n} + Mshr_{lane.neg_n}\right)\cdot g_{mom}$

$\max\left(M_{dist.live.pos}\right) = 208\cdot kip\cdot ft$ $\max\left(V_{dist.live.pos}\right) = 17.4\cdot kip$ $\max\left(Mshr_{dist.live.pos}\right) = 207.7\cdot kip\cdot ft$

$\min(M_{dist.live.neg}) = -8 \cdot kip \cdot ft$ $\min(V_{dist.live.neg}) = -17.4 \cdot kip$ $\max(Mshr_{dist.live.neg}) = 207.7 \cdot kip \cdot ft$

Distributed Live Load - Permit Loading

$M_{dist.live.permit.pos_n} := \left(1.33 \cdot M_{permit.pos_n} + M_{lane.permit.pos_n}\right) \cdot g_{mom}$

$M_{dist.live.permit.neg_n} := \left(1.33 \cdot M_{permit.neg_n} + M_{lane.permit.neg_n}\right) \cdot g_{mom}$

$V_{dist.live.permit.pos_n} := \left(1.33 \cdot V_{permit.pos_n} + V_{lane.permit.pos_n}\right) \cdot g_{shear}$

$V_{dist.live.permit.neg_n} := \left(1.33 \cdot V_{permit.neg_n} + V_{lane.permit.neg_n}\right) \cdot g_{shear}$

$Mshr_{dist.live.permit.pos_n} := \left(1.33 \cdot Mshr_{permit.pos_n} + Mshr_{lane.permit.pos_n}\right) \cdot g_{mom}$

$Mshr_{dist.live.permit.neg_n} := \left(1.33 \cdot Mshr_{permit.neg_n} + Mshr_{lane.permit.neg_n}\right) \cdot g_{mom}$

$\max(M_{dist.live.permit.pos}) = 0.2 \cdot kip \cdot ft$ $\max(V_{dist.live.permit.pos}) = 0 \cdot kip$

$\min(M_{dist.live.permit.neg}) = -0 \cdot kip \cdot ft$ $\min(V_{dist.live.permit.neg}) = -0 \cdot kip$

$\max(Mshr_{dist.live.permit.pos}) = 0.2 \cdot kip \cdot ft$

$\max(Mshr_{dist.live.permit.neg}) = 0.2 \cdot kip \cdot ft$

Distributed Live Load - Fatigue Truck

$M_{dist.fatigue.pos_n} := \left(1.15 \cdot M_{fatiguetruck.pos_n}\right) \cdot g_{mom.fatigue}$

Bearing Reactions

```
f_trk.reaction(trk, asp, bd, lgth) := | stop ← last(trk)
                                      | react ← 0
                                      | for i ∈ 0 .. stop
                                      |   | sum ← 0
                                      |   | for j ∈ i .. stop
                                      |   |   | fact ← (lgth − bd)/(lgth − 2·bd)   if j = i
                                      |   |   | fact ← ((lgth − 2·bd) − [(Σ_{n=i+1}^{j} asp_n) − bd])/(lgth − 2·bd)   otherwise
                                      |   |   | sum ← sum + fact·trk_j
                                      |   |   | chk_{i,j} ← fact
                                      |   | react ← sum   if sum > react
                                      | react
```

```
Reaction_truck := | ans ← 0
                  | for i ∈ 0 .. 2
                  |   | trkreact ← f_trk.reaction(P^⟨i⟩, d^⟨i⟩, BearingDistance/ft, L_beam/ft)
                  |   | ans ← trkreact   if trkreact > ans
                  | ans
```

$\cdot kip$ $Reaction_{truck} = 19.1 \cdot kip$

```
Reaction_permit.trk := | ans ← 0
                       | for i ∈ 3 .. 4
                       |   | trkreact ← f_trk.reaction(P^⟨i⟩, d^⟨i⟩, BearingDistance/ft, L_beam/ft)
                       |   | ans ← trkreact   if trkreact > ans
                       | ans
```

$\cdot kip$ $Reaction_{permit.trk} = 0 \cdot kip$

$Reaction_{lane} := 0 \cdot \frac{kip}{ft} \cdot \frac{L_{beam}}{2} = 0 \cdot kip$

$Reaction_{lane.permit} := \left(PermitUniformLoad + 0.1 \frac{lbf}{ft}\right) \cdot \frac{L_{beam}}{2} = 0 \cdot kip$

$Reaction_{composite.dl} := w_{composite} \cdot \frac{L_{beam}}{2} = 1.4 \cdot kip$

$$Reaction_{noncomposite.dl} := w_{noncomposite} \cdot \frac{L_{beam}}{2} \ldots + \max\left[\left(\left|EndDiaphragmA\right| + \left|V_{point_1}\right|\right), \left(\left|EndDiaphragmE\right| + \left|V_{point_{nmax-1}}\right|\right)\right] = 31.2 \cdot kip$$

$$Reaction_{HL93} := \left(Reaction_{truck} \cdot 1.33 + Reaction_{lane}\right) \cdot g_{shear} = 17.6 \cdot kip$$

$$Reaction_{permit} := \left(Reaction_{permit.trk} \cdot 1.33 + Reaction_{lane.permit}\right) \cdot g_{shear} = 0 \cdot kip$$

$$Reaction_{LL} := Reaction_{HL93} = 17.6 \cdot kip$$

$$Reaction_{DL} := Reaction_{composite.dl} + Reaction_{noncomposite.dl} = 32.6 \cdot kip$$

```
WRITEPRN("beam.dat") := | r0 ← 0  if BeamType = "TypeII"
                        | r0 ← 1  if BeamType = "TypeIII"
                        | r0 ← 2  if BeamType = "TypeIV"
                        | r0 ← 3  if BeamType = "TypeV"
                        | r0 ← 4  if BeamType = "TypeVI"
                        | r0 ← 5  if BeamType = "FBT54"
                        | r0 ← 6  if BeamType = "FBT63"
                        | r0 ← 7  if BeamType = "FBT72"
                        | r0 ← 8  if BeamType = "FBT78"
                        | r0 ← 10  if BeamType = "FIB36"
                        | r0 ← 11  if BeamType = "FIB45"
                        | r0 ← 12  if BeamType = "FIB54"
                        | r0 ← 13  if BeamType = "FIB63"
                        | r0 ← 14  if BeamType = "FIB72"
                        | r0 ← 15  if BeamType = "FIB78"
                        | r0 ← 16  if BeamType = "FIB84"
                        | r0 ← 17  if BeamType = "FIB96"
                        | r0 ← 18  if BeamType = "FIB120"
                        | r0 ← 30  if substr(BeamType, 0, 3) = "FUB"
```



```
                        | r0 ← 50  if substr(BeamType, 0, 3) = "FUB"
                        | r0 ← 50  if BeamType = "FDT18"
                        | r0 ← 51  if BeamType = "FDT24"
                        | r0 ← 52  if BeamType = "FDT30"
                        | r0 ← 60  if (FSBBeamName = "FSB12x48")·(BeamType = "FSB12")
                        | r0 ← 61  if (FSBBeamName = "FSB15x48")·(BeamType = "FSB15")
                        | r0 ← 62  if (FSBBeamName = "FSB18x48")·(BeamType = "FSB18")
                        | r0 ← 63  if (FSBBeamName = "FSB12x49")·(BeamType = "FSB12")
                        | r0 ← 64  if (FSBBeamName = "FSB15x49")·(BeamType = "FSB15")
                        | r0 ← 65  if (FSBBeamName = "FSB18x49")·(BeamType = "FSB18")
                        | r0 ← 66  if (FSBBeamName = "FSB12x50")·(BeamType = "FSB12")
                        | r0 ← 67  if (FSBBeamName = "FSB15x50")·(BeamType = "FSB15")
                        | r0 ← 68  if (FSBBeamName = "FSB18x50")·(BeamType = "FSB18")
                        | r0 ← 69  if (FSBBeamName = "FSB12x51")·(BeamType = "FSB12")
                        | r0 ← 70  if (FSBBeamName = "FSB15x51")·(BeamType = "FSB15")
                        | r0 ← 71  if (FSBBeamName = "FSB18x51")·(BeamType = "FSB18")
                        | r0 ← 72  if (FSBBeamName = "FSB12x52")·(BeamType = "FSB12")
                        | r0 ← 73  if (FSBBeamName = "FSB15x52")·(BeamType = "FSB15")
                        | r0 ← 74  if (FSBBeamName = "FSB18x52")·(BeamType = "FSB18")
                        | r0 ← 75  if (FSBBeamName = "FSB12x53")·(BeamType = "FSB12")
                        | r0 ← 76  if (FSBBeamName = "FSB15x53")·(BeamType = "FSB15")
                        | r0 ← 77  if (FSBBeamName = "FSB18x53")·(BeamType = "FSB18")
                        | r0 ← 78  if (FSBBeamName = "FSB12x54")·(BeamType = "FSB12")
                        | r0 ← 79  if (FSBBeamName = "FSB15x54")·(BeamType = "FSB15")
                        | r0 ← 80  if (FSBBeamName = "FSB18x54")·(BeamType = "FSB18")
                        | r0 ← 81  if (FSBBeamName = "FSB12x55")·(BeamType = "FSB12")
                        | r0 ← 82  if (FSBBeamName = "FSB15x55")·(BeamType = "FSB15")
                        | r0 ← 83  if (FSBBeamName = "FSB18x55")·(BeamType = "FSB18")
                        | r0 ← 84  if (FSBBeamName = "FSB12x56")·(BeamType = "FSB12")
                        | r0 ← 85  if (FSBBeamName = "FSB15x56")·(BeamType = "FSB15")
                        | r0 ← 86  if (FSBBeamName = "FSB18x56")·(BeamType = "FSB18")
                        | r0 ← 87  if (FSBBeamName = "FSB12x57")·(BeamType = "FSB12")
                        | r0 ← 88  if (FSBBeamName = "FSB15x57")·(BeamType = "FSB15")
```

$r_0 \leftarrow 89$ if $(FSBBeamName = "FSB18x57") \cdot (BeamType = "FSB18")$

$r_0 \leftarrow 90$ if $(FSBBeamName = "FSB12x58") \cdot (BeamType = "FSB12")$

$r_0 \leftarrow 91$ if $(FSBBeamName = "FSB15x58") \cdot (BeamType = "FSB15")$

$r_0 \leftarrow 92$ if $(FSBBeamName = "FSB18x58") \cdot (BeamType = "FSB18")$

$r_0 \leftarrow 93$ if $(FSBBeamName = "FSB12x59") \cdot (BeamType = "FSB12")$

$r_0 \leftarrow 94$ if $(FSBBeamName = "FSB15x59") \cdot (BeamType = "FSB15")$

$r_0 \leftarrow 95$ if $(FSBBeamName = "FSB18x59") \cdot (BeamType = "FSB18")$

$r_0 \leftarrow 96$ if $(FSBBeamName = "FSB12x60") \cdot (BeamType = "FSB12")$

$r_0 \leftarrow 97$ if $(FSBBeamName = "FSB15x60") \cdot (BeamType = "FSB15")$

$r_0 \leftarrow 98$ if $(FSBBeamName = "FSB18x60") \cdot (BeamType = "FSB18")$

$r_0 \leftarrow 99$ if $substr(BeamType, 0, 3) = "Inv"$

$r_0 \leftarrow 99$ if $BeamType = "Custom"$

$r_0 \leftarrow 99$ otherwise

$r_1 \leftarrow "e"$

$r_2 \leftarrow \frac{h_{beam}}{ft}$

r

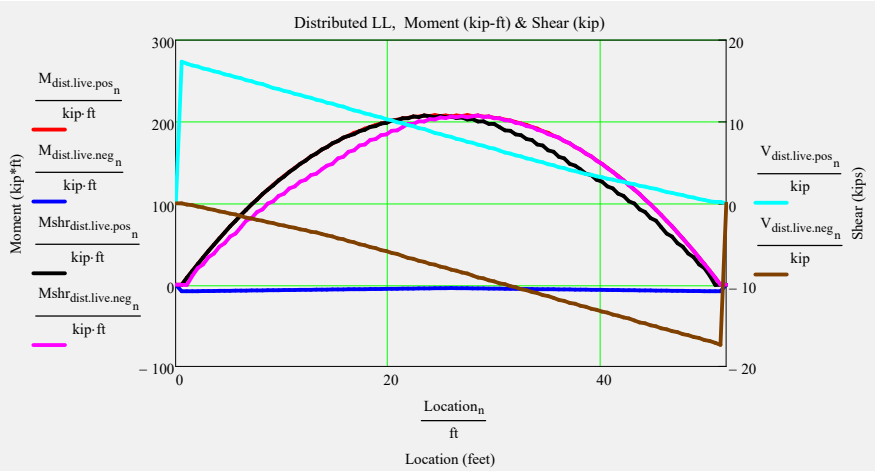
$BeamType = "FSB15"$ $FSBBeamName = "FSB15x54"$ $h_{beam} = 15 \cdot in$

Live Load Moments and Shears

Distributed Live Load Moments and Shears

$L_{span} = 50.75\ ft$

Live Load Distribution Factors $g_{mom} = 0.6909$ $g_{mom.fatigue} = 0.6909$ $g_{shear} = 0.6909$



$\max(V_{dist.live.pos}) = 17.4 \cdot kip$ $\max(M_{dist.live.pos}) = 208.0 \cdot kip \cdot ft$ $\min(V_{dist.live.neg}) = -17.4 \cdot kip$

$Reaction_{LL} = 17.6 \cdot kip$ $\min(M_{dist.live.neg}) = -8.0 \cdot kip \cdot ft$

$Reaction_{DL} = 32.6 \cdot kip$ $\max(Mshr_{dist.live.pos}) = 208 \cdot kip \cdot ft$

Beam End Reactions...
with IM factor only

Reinforcement Properties

InputDataFile = "Able Canal - Spans 2-4 - Interior - H10 Load.dat"

Mild Reinforcement Type:

- Carbon Steel - Grade 60
- Stainless Steel - Grade 60
- Glass Fiber Reinforced Polymer
- Carbon Steel - Grade 80
- Stainless Steel - Grade 75

Help on Mild Reinforcement Types

Longitudinal Mild and Partial PS Reinforcement:

Excel Table of Standard FDOT Prestressed Beam Mild and Partial PS Longitudinal Reinforcement

Help on Longitudinal Mild and Partial PS Reinf

Mild Reinforcement

	Deck	Bottom of Beam	Beam End: Top	Beam End: Bottom
Location				
Area (in^2)	0.4108 *1	0	0	0
Distance (in)	2.3125 *2	0 *3		
Length (ft)			0	0
Bar Size		5 *4	0 *4	0 *4

Partial PS (Dormant) Strand

Location	Top of Beam
Diameter(in)	3/8 inch / 1/2 inch
Distance (in)	2
# Strands	4
Force per Strand (kip)	10

**1 - Area of longitudinal deck reinf. per unit width of deck, both layers combined. Typically one #5 Bar top & bottom for 8" decks.*
**2 - Distance measured from top of deck to centroid of deck longitudinal reinforcement, positive value.*
**3 - Distance measured from top of beam, positive value.*
**4 - Size of bars used to create* $A_{s.long}$ *needed to calculate development length.*

Prestressing Tendons:

Humidity 75 % *% relative humidity (75% typical)*

Time: jacking to transfer 0.75 days *Time in days between jacking and transfer.*

Prestress Strand Type

- Carbon Steel - Low Lax
- Carbon Steel - Stress Relieved
- Stainless Steel
- Carbon Fiber Reinforced Polymer

Help on Prestress Reinforcement Typ

Prestress Strand Size

- 0.6 in.
- 0.5 in.
- 0.5 in. Special
- 9/16 in.
- 0.7 in.

Help on Strand Generator | Help on Strand Debonding

Strand Pattern Generator for Entering Prestressing Strand Layout

Double click the icon to open the 'Strand Pattern Generator'. Specify the location and debonding of strands. When finished, press the 'Continue' button. Then press 'Read Strand Data' button below.

Press 'Read Strand Data' Button.to Read in Strand Generator Data

Read Strand Data

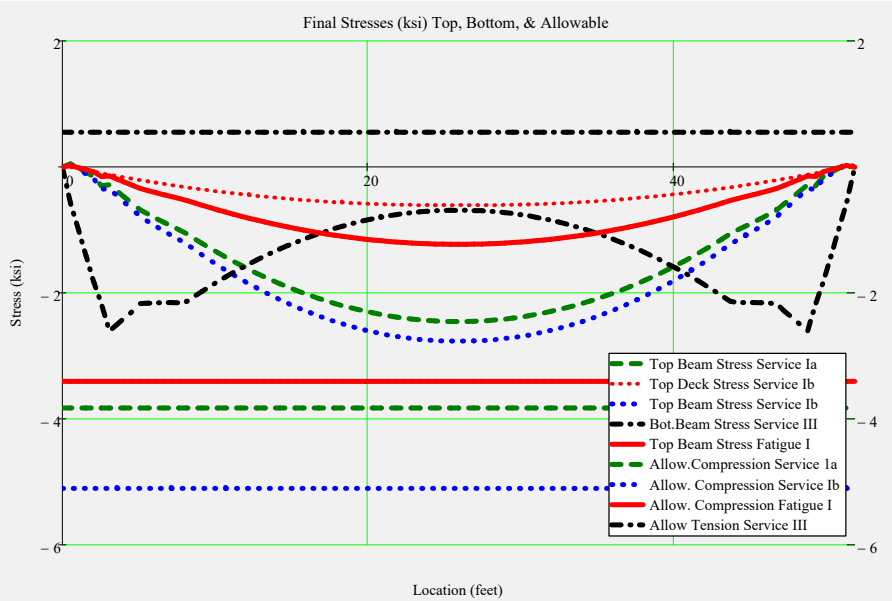
Press 'Read Strand Data' button to Read in Strand Generator Data.

Material Properties
Strand Data, Pattern, and Properties
Section Properties, Strand Profile Properties, and Service I and III Beam Stresses and Limits

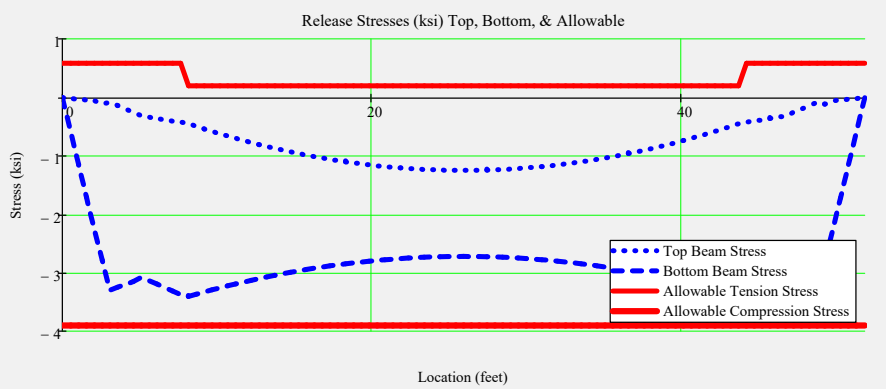
Service I and III Stresses and Limits and Tendon Layout Summary

tension (+), compression (--)

Final Stresses



Release Stresses



Stress Checks at Service Limit States

Release tension	$Check_f_{tension.rel}$ = "OK"	$\max(DC_f_{tension.rel}) = 0.68$
Release compression	$Check_f_{comp.rel}$ = "OK"	$\max(DC_f_{comp.rel}) = 0.87$
*Service III , PS+DL+0.8*LL*	$Check_f_{tension.ServiceIII}$ = "OK"	$\max(DC_f_{tension.ServiceIII}) = 0.82$
Service Ia , PS+DL	$Check_f_{comp.ServiceIa}$ = "OK"	$\max(DC_f_{comp.ServiceIa}) = 0.64$
Service Ib , PS+DL+LL	$Check_f_{comp.ServiceIb}$ = "OK"	$\max(DC_f_{comp.ServiceIb}) = 0.54$
*Fatigue I , (PS + DL)*0.5 +1.5 Fatigue Truck*	$Check_f_{comp.FatigueI}$ = "OK"	$\max(DC_f_{comp.FatigueI}) = 0.36$

StressChecks =

"StressCheck"	"Location(ft)"	"Stress(ksi)"	"AllowableStress(ksi)"
"Tension @ Release"	0.00	0.00	0.59
"Compression @ Release"	8.15	−3.40	−3.90
"Tension @ ServiceIII"	0.00	0.00	0.55
"CompressionIa @ ServiceI"	25.92	−2.45	−3.83
"CompressionIb @ ServiceI"	26.42	−2.76	−5.10
"Compression @ FatigueI"	25.92	−1.23	−3.40

CheckBeamEndTopLongReinf = "OK"

Strand Pattern Checks

1. The number of strands debonded per row shall not exceed 45 percent of the strands provided in that row.
2. Debonding shall not be terminated for more than six strands in any given section. When a total of ten or fewer strands are debonded, debonding shall not be terminated for more than four strands in any given section
3. Longitudinal spacing of debonding termination locations shall be at least 60db apart.
4. Alternate bonded and debonded strand locations both horizontally and vertically.
5. For simple span precast, pretensioned girders, debonding length from the beam end should be limited to 20 percent of the span length or one half the span length minus the development length, whichever is less.

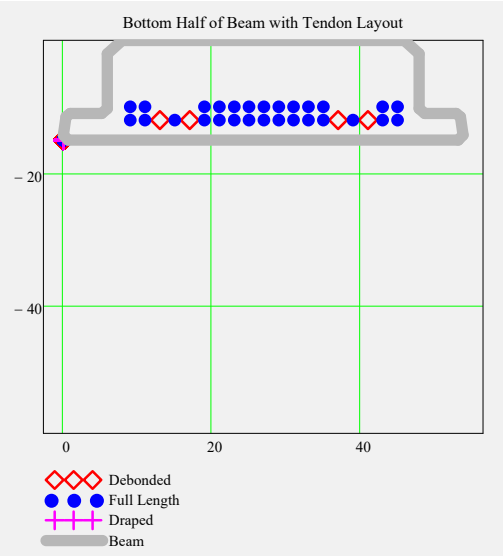
$$CheckPattern = \begin{pmatrix} \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \\ \text{"OK"} \end{pmatrix}$$

For single-web flanged sections (I-beams, bulbtees, and inverted-tees):
1. Bond all strands within the horizontal limits of the web
2. Bond the outer-most strands in all rows located within the full-width section of the flange.
3. Position debonded strands furthest from the vertical centerline.
For multi-web sections having bottom flanges (voided slab, box beams and U-beams):
1. Strands shall be bonded within 1.0 times the web width projection.
2. Bond the outer-most strands within the section.
3. Uniformly distribute debonded strands between webs.

BeamType = "FSB15"

$$CheckWebPattern = \begin{pmatrix} \text{"OK"} \\ \text{"OK"} \end{pmatrix}$$

Tendon Layout



BeamType = "FSB15"

$h_{beam} = 15 \cdot in$

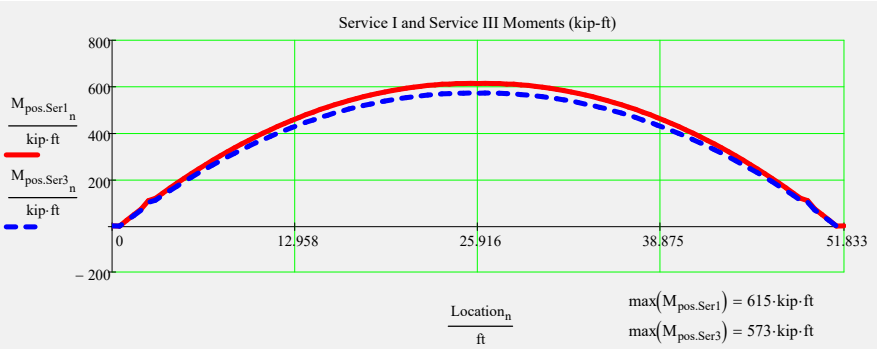
$A_{strand} = 0.217 \cdot in^2$

#Strands = 32

Prestress Losses Summary

$f_{pj} = 202.5 \cdot ksi$	$Check_f_{pt}$ = "OK"
$\Delta f_{pES} = 0 \cdot ksi$	*Note: Elastic shortening losses are zero in concrete stress calculations when using transformed section properties per* *LRFD 5.9.3.2.3*
$\Delta f_{pT} = -25 \cdot ksi$	$\frac{\Delta f_{pT}}{f_{pj}} = -12.22 \cdot \%$
$f_{pe} = 178 \cdot ksi$	$\frac{f_{pe}}{f_{pj}} = 87.78 \cdot \%$
$0.8 \cdot f_{py} = 194 \cdot ksi$	$Check_f_{pe}$ = "OK"

Moments at Service Limit States



Summary of Values at Midspan

Stresses =

"Stage"	"Top of Beam (ksi)"	"Bottom of Beam (ksi)"
"1"	–1.25	–2.72
"2"	–1.30	–2.21
"4"	–1.23	–2.27
"6"	–2.43	–1.24
"8"	–2.76	–0.69

PrestressForce =

"Time"	"Axial (kip)"	"Moment (kip*ft)"
"@Release"	–1446	–368
"@Final (about composite centroid)"	–1269	–305

Properties =

"Section"	"Area (in^2) "	"Inertia (in^4) "	"Top of Beam to Centroid (in)"
"Net Beam"	664.6	12861	–7.9
"Transformed Beam (initial)"	709.3	13394	–8.1
"Transformed Beam (final)"	704.4	13339	–8.1
"Composite Section"	1100.1	39765	–5

ServiceMoments =

"Load Type "	"Moment (kip*ft)"
"Release Dead Load"	235
"Non-Composite DL (incl. beam wt.)"	390
"Composite Dead Load"	17
"Distributed Live Load"	206

Stage 1 ---> At release with span length equal to length of the beam. Prestress losses are elastic shortening and overnight relax

Stage 2 ---> At final, starting with Stage 1 and adding the remaining prestress losses applied to the transformed beam

Stage 4 ---> Starting with Stage 2 and changing the support locations from the end of the beam to the bearing locations

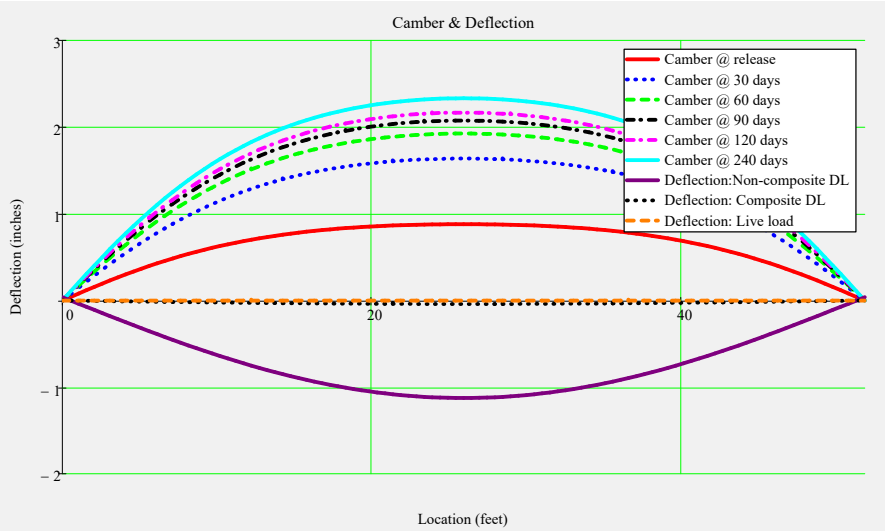
Stage 6 ---> Starting with Stage 4 and adding non-composite dead load (excluding beam wt included in Stage 1)

Stage 8 ---> Starting with Stage 6 and adding composite dead load and distributed live loads applied to the composite section

Camber at Release, Short Term Camber, & Summary of Slope Data

Camber at Different Stages and Deflection due to Dead Load and Live Load

$L_{span} = 50.75\,ft$



Slope&Defl =

"Stage"	"ΔL @ Beam Top (in)"	"ΔL @ Beam Bot. (in)"	"Slope at End (deg)"	"Midspan Defl (in)"
"Release"	–0.1000	–0.3936	0.3771	0.8815
"30 Days"	–0.2547	–0.7282	0.6822	1.6389
"60 Days"	–0.3126	–0.8547	0.7986	1.9277
"90 Days"	–0.3426	–0.9202	0.8588	2.0772
"120 Days"	–0.3610	–0.9603	0.8956	2.1685
"240 Days"	–0.3944	–1.0331	0.9624	2.3341
"Non-comp DL"	–0.0953	0.0822	–0.3392	–1.1242
"Comp DL"	–0.0020	0.0041	–0.0118	–0.0391
"LL"	0.0000	0.0000	0.0000	0.0000

Check net camber at end of construction (120 day) *SDG 4.3.2.D & 4.4.3.A*

$NetCamber := \max(Defl_{120day}) + \min(Defl_{dl.nc}) + \min(Defl_{dl.c}) = 1.01 \cdot in$

$CheckNetCamber := if(NetCamber > 0, "OK", "No Good")$ CheckNetCamber = "OK"

Moment Resistance

Moments and Moment Resistance at StrengthI and StrengthII Limit State

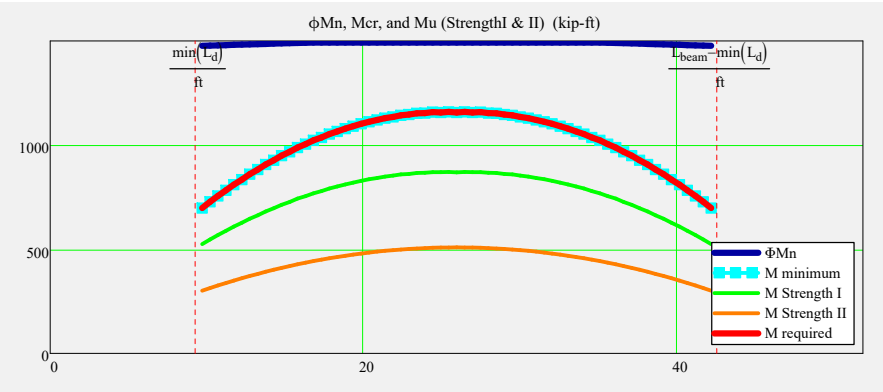
Values at midspan

$LimitStateForMoment_{51}$ = "Strength I" $\max(M_{Strength}) = 872 \cdot kip \cdot ft$

$IsSectionNonCompControlled?_{51}$ = "Yes" $M_{minimum_{51}} = 1158 \cdot kip \cdot ft$ $M_{reqd_{51}} = 1158 \cdot kip \cdot ft$

StrandType = "Carbon Steel - Low Lax" $\phi_{mom_{51}} = 0.85$ $\phi_{mom_{51}} \cdot (M_{n_{51}}) = 1493 \cdot kip \cdot ft$

$FailureMode_{51}$ = "N.A." CheckMinSSLimit = "N.A."



$\min(L_d) = 9.2\,ft$ $L_{beam} - \min(L_d) = 42.6\,ft$

$DC_{moment_{mn}} := \dfrac{M_{reqd_{mn}}}{\phi_{mom_{mn}} \cdot (M_{n_{mn}})}$ *Demand/Capacity Ratio at locations where all strands are fully developed* $\max(DC_{moment}) = 0.78$

CheckMomentCapacity := $\text{if}(\max(DC_{moment}) < 1.0, \text{"OK"}, \text{"No Good!"}) = \text{"OK"}$

FSB only - Design Check of Transverse Reinforcing Bars E

FSB only - Design Check of Bottom Transverse Reinforcing Bars E

Spacing of Bars E is checked per Control of Cracking by Distribution of Reinforcement [LRFD 5.6.7 or AASHTO GFRP 2.6.7].
The Service I moment in transverse direction is calculated based on the r ratio to the longitudinal moment per LRFD 9.7.3.2.

$s \le \dfrac{700 \cdot \gamma_e}{\beta_s \cdot f_{ss}} - 2 \cdot d_c$ *The maximum spacing of the mild steel reinforcement for control of cracking at the service limit state shall satisfy* *[LRFD 5.6.7]*

$\gamma_e := 1.00$

$s \le \min\left(1.15 \cdot \dfrac{C_b \cdot E_f \cdot w}{f_{fs}} - 2.5c_c, 0.92 \cdot \dfrac{C_b \cdot E_f \cdot w}{f_{fs}}\right)$ *[AASHTO GFRP 2.6.7-1]*

$d_c \le \dfrac{C_b E_f w}{2 f_{fs} \xi}$ (2.6.7-2) *[AASHTO GFRP Eq. 2.6.7-2]*

$C_b := 0.83$ *GFRP bars bond coefficient*

$w_{max} := 0.028in$ *limiting crack width*

$A_{BarsE} := 0.31in^2$ *Area of Bars E*

$s_{BarsE} := 6in$ *Typical Spacing of Bars E*

$s_{BarsE.NearEnds} := 12in$ *Spacing of Bars E near ends of span*

$n_{BarsE.NearEnds} := 6$ *Number of spaces of Bars E near ends of span*

BarsESize := 5 *Size of Bars E*

$c_c := 1.5in$ *clear cover*

Area and spacing of Bars E per location (up to half span)

$hs := 0 .. \dfrac{last(Location)}{2} - 1$

$A_{BarsE.prov_{hs}} := \begin{cases} \begin{cases} \dfrac{A_{BarsE}}{s_{BarsE.NearEnds}} & \text{if } Location_{hs} \le n_{BarsE.NearEnds} \cdot s_{BarsE.NearEnds} \\ \dfrac{A_{BarsE}}{s_{BarsE}} & \text{otherwise} \end{cases} & \text{if substr(BeamType, 0, 3) = "FSB"} \\ 0 & \text{otherwise} \end{cases}$

$s_{BarsE.prov_{hs}} := \begin{cases} \begin{cases} s_{BarsE.NearEnds} & \text{if } Location_{hs} \le n_{BarsE.NearEnds} \cdot s_{BarsE.NearEnds} \\ s_{BarsE} & \text{otherwise} \end{cases} & \text{if substr(BeamType, 0, 3) = "FSB"} \\ 0 & \text{otherwise} \end{cases}$

$ratio := \dfrac{100}{\sqrt{\dfrac{L_{span}}{ft}}} \cdot \dfrac{1}{100}$ *LRFD 9.7.3.2*

$ratio := \text{if}(ratio \le 0.5, ratio, 0.5)$ $ratio = 0.14$

$M_{transverse.pos.Ser1} := ratio \cdot \frac{M_{dist.live.pos} + M_{dl.comp}}{b_e} \cdot 1ft$ $\quad \max(M_{transverse.pos.Ser1}) = 6.89 \cdot kip \cdot ft$ *per foot width*

Note: there is no stress induced in Bars E from the self-weight of the FSB's or the CIP deck overlay.

$d_c := c_c + 0.5 \cdot \frac{BarsESize}{8} \cdot in = 1.81 \cdot in$ *Distance from extreme tension fiber to center of closest bar*

$h := t_{deck} + Height_{pocket} = 17 \cdot in$ *Overall thickness or depth of the component*

$d_s := h - d_c = 15.19 \cdot in$

$\beta_s := 1 + \frac{d_c}{0.7(h - d_c)} = 1.17$

$E_{c.deck} := 120000 \cdot K_1 \cdot \left(w_{c.deck} \cdot \frac{ft^3}{kip}\right)^2 \cdot \left(\frac{f_{c.deck}}{ksi}\right)^{0.33} \cdot ksi = 4145 \cdot ksi$

$n_s := \frac{E_s}{E_{c.deck}} = 7$ *Modular ratio*

$\rho_{hs} := \frac{A_{BarsE.prov_{hs}}}{d_s}$ *Reinforcement ratio*

$kk_{hs} := \sqrt{2 \cdot \rho_{hs} \cdot n_s + (\rho_{hs} \cdot n_s)^2} - \rho_{hs} \cdot n_s$

$j_{hs} := 1 - \frac{kk_{hs}}{3}$

$f_{r.pocket} := 0.24 \cdot \sqrt{f_{c.deck} \cdot ksi}$ $\quad f_{r.pocket} = 0.51 \cdot ksi$

$S_{pocket} := \frac{1}{6} \cdot 12 \cdot in \cdot h^2$ $\quad S_{pocket} = 578 \cdot in^3$

$\gamma_1 := 1.6$ *Flexural cracking variability factor*

$\gamma_3 := 0.67$ *Ratio of specified minimum yield strength to ultimate tensile strength of the reinforcement (0.67 for ASTM A615, Grade 60 reinforcing steel per SDG 1.4.1)*

$M_{cr.pocket} := \gamma_1 \cdot \gamma_3 \cdot f_{r.pocket} \cdot S_{pocket}$ $\quad M_{cr.pocket} = 26.29 \cdot kip \cdot ft$ *per feet width*

$$f_{ss.Ser1_{hs}} := \begin{cases} \text{if substr(BeamType, 0, 3) = "FSB"} \\ \quad \begin{cases} 0 & \text{if } M_{transverse.pos.Ser1_{hs}} \le 0 \\ \frac{M_{transverse.pos.Ser1_{hs}}}{A_{BarsE.prov_{hs}} \cdot 1ft \cdot j_{hs} \cdot d_s} & \text{otherwise} \end{cases} \\ 0 \quad \text{otherwise} \end{cases}$$

Stress in Bars E (if moment is negative, stress in bottom steel is set to 0)

Required reinforcement spacing

$$Spa_{BarsE.reqd_{hs}} := \begin{cases} \text{if substr(BeamType, 0, 3) = "FSB"} \\ \quad \begin{cases} 10000in & \text{if } f_{ss.Ser1_{hs}} \le 0 \\ \text{otherwise} \\ \quad \begin{cases} \left(\frac{700 \cdot \gamma_e \cdot \frac{kip}{in}}{\beta_s \cdot f_{ss.Ser1_{hs}}} - 2 \cdot d_c\right) & \text{if (MildReinfType = "Carbon Steel") + (MildReinfType = "Stainless Steel")} \\ \min\left(1.15 \cdot \frac{C_b \cdot E_f \cdot w_{max}}{f_{ss.Ser1_{hs}}} - 2.5 \cdot c_c, 0.92 \cdot \frac{C_b \cdot E_f \cdot w_{max}}{f_{ss.Ser1_{hs}}}\right) & \text{if MildReinfType = "Glass Fiber Reinforced Polymer"} \end{cases} \end{cases} \\ 0 \quad \text{otherwise} \end{cases}$$

$CriticalSpa_{BarsE.reqd} := \min(Spa_{BarsE.reqd}) = 60.01 \cdot in$ *The most critical spacing along cap sections*

$DC_{BarsE_{hs}} := \frac{s_{BarsE.prov_{hs}}}{Spa_{BarsE.reqd_{hs}}}$ $\quad \max(DC_{BarsE}) = 0.15$

$$d_{c.max_{hs}} := \begin{cases} \text{if substr(BeamType, 0, 3) = "FSB"} \\ \quad \begin{cases} dcmax \leftarrow 10000in & \text{if } f_{ss.Ser1_{hs}} \le 0 \\ \text{otherwise} \\ \quad \begin{cases} \zeta_{hs} \leftarrow \frac{h - kk_{hs} \cdot d_s}{d_s - kk_{hs} \cdot d_s} \\ dcmax \leftarrow \frac{C_b \cdot E_f \cdot w_{max}}{2 \cdot f_{ss.Ser1_{hs}} \cdot \zeta_{hs}} \end{cases} \end{cases} \\ dcmax \leftarrow 0 \quad \text{otherwise} \\ dcmax \end{cases}$$

limiting d.c value

[AASHTO GFRP Eq. 2.6.7-2]

$\min(d_{c.max}) = 7 \cdot in$

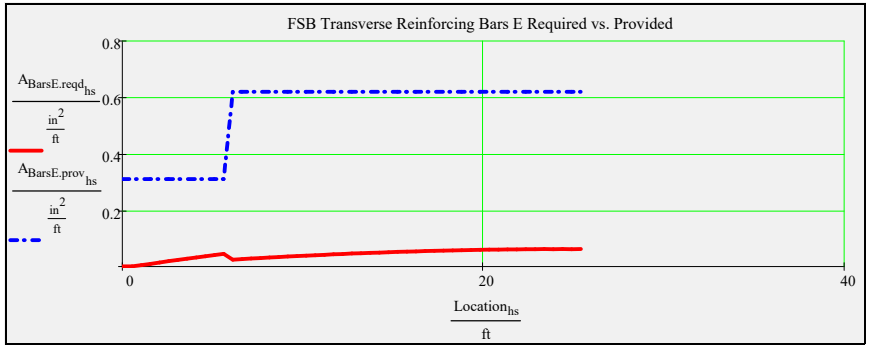
$d_c = 1.81 \cdot in$

$f_{s.max} := \text{if}(f_y < 75ksi, 0.8 \cdot f_y, 0.6 \cdot f_y) = 48 \cdot ksi$ *[SDG 4.1.8]*

$$Check_{FSB.BarsE} := \begin{cases} \text{if substr(BeamType, 0, 3) = "FSB"} \\ \quad \begin{cases} \text{"OK"} & \text{if } \max(DC_{BarsE}) \le 1 \wedge \max(f_{ss.Ser1}) \le f_{s.max} \wedge \text{(MildReinfType = "Carbon Steel") + (MildReinfType} \\ \text{"OK"} & \text{if } \max(DC_{BarsE}) \le 1 \wedge d_c \le \min(d_{c.max}) \wedge \text{MildReinfType = "Glass Fiber Reinforced Polymer"} \\ \text{"No Good"} & \text{otherwise} \end{cases} \\ \text{"N/A"} \quad \text{otherwise} \end{cases}$$

$Check_{FSB.BarsE} = \text{"OK"}$

$$A_{BarsE.reqd_{hs}} := \text{if}\left(\text{substr}(BeamType,0,3) = \text{"FSB"}, \frac{A_{BarsE}}{Spa_{BarsE.reqd_{hs}}}, 0\right)$$



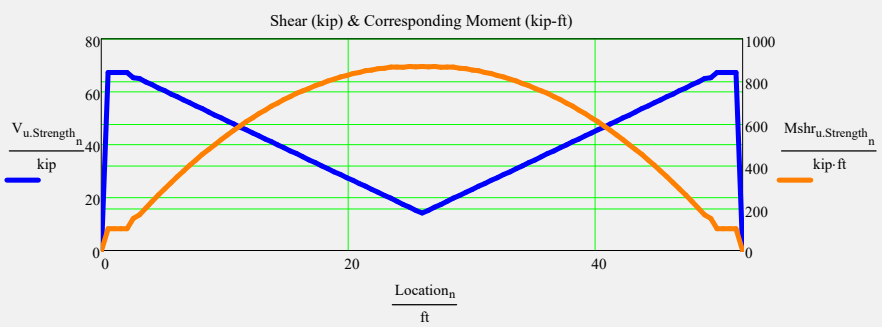
FSB only - Design Check of Transverse Reinforcing Bars E

Shear Loads

Shear and Corresponding Moments:

Calculations use the greater of Strength I or Strength II Limit State

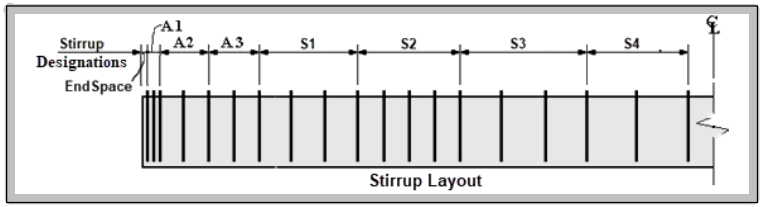
$$\begin{pmatrix} \max(V_{pos.StrengthI}) \\ \max(V_{pos.StrengthII}) \end{pmatrix} = \begin{pmatrix} 70 \\ 40 \end{pmatrix} \cdot kip \quad \begin{pmatrix} \text{"HL93"} \\ \text{"Permit Vehicle"} \end{pmatrix}$$



$\max(V_{u.Strength}) = 67.3 \cdot kip$ ControllingShearLimitState = "Strength I" $\max(Mshr_{u.Strength}) = 870 \cdot kip \cdot ft$

Shear Resistance

Stirrup Size and Spacings (values are for one half of the beam)



End Stirrup Space	2.5	in	*Distance from end of beam to the centerline of first stirrup or stirrup bundle.*
GFRP Stirrup Bar #	0	*used for GFRP Bars*	

Help on Stirrup Design | Help on Stirrup Design Iter. | Table of FDOT Standard Prestressed Beam End Reinforcement | Reset Load Data

Stirrup Design - see Stirrup Layout

Designation	A1	A2	A3	S1	S2	S3	S4
Spacing (in)	3	7	0	12	24	0	24
# of Spaces	3	1	0	4	10	0	Calculated
Area (in^2)	0.60	0.80	0.0	1.00	1.20	0.0	1.20
Interface Factor	0.33	1.0	0.0	0.80	0.67	0.0	0.67

Save and Initialize Data

Shear Demand/Capacity and Longitudinal Steel Demand/Capacity

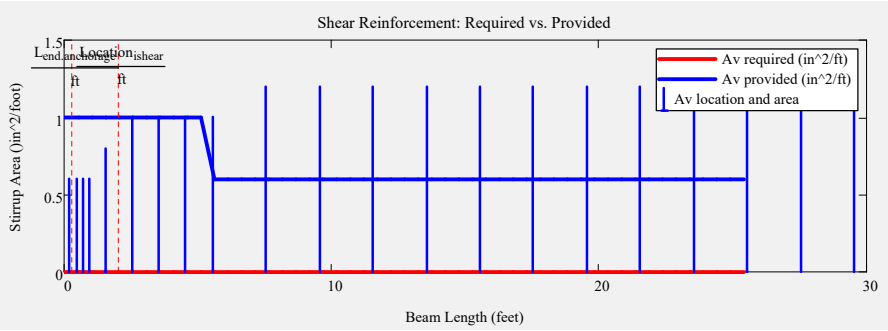
CheckShearCapacity = "OK"

CheckStirArea = "OK"

CheckMaxShearStress = "OK"

CheckMinStirArea = "OK"

CheckMaxStirSpacing = "OK"



$Location_{ishear} = 2.06\ ft$

Save Data File

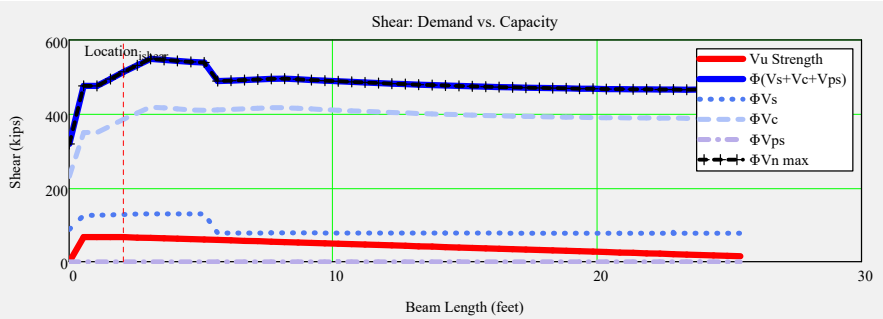
Use current input file

File Name Able Canal - Spans 2-4 - Interior - H10 Load.dat

Save Data

DataFileFolder = "C:\Users\jjohnson2\Desktop\Able Canal\Superstructure\PrestressedBeamV6.1\DataFiles-FSB"

Note: Select an output folder by using the "Change Folder" option on page 1.

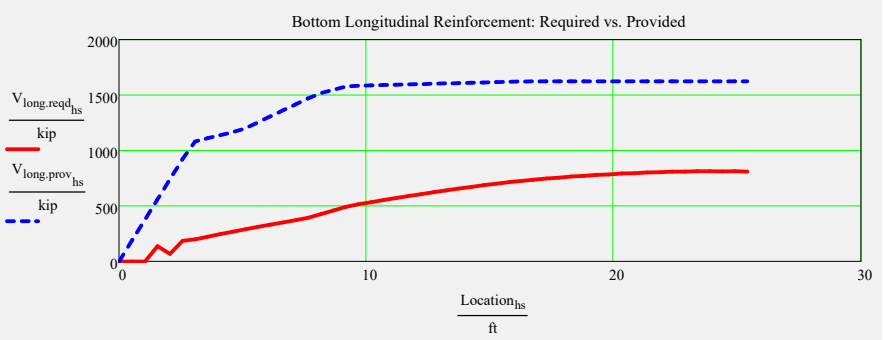


$$fCheckD/CShear = \begin{pmatrix} \text{"CheckType"} & \text{"Location(ft)"} & \text{"Vu (kip)"} & \text{"Vr (kip)"} \\ \text{"Shear"} & 2.06 & 67.32 & 514.28 \end{pmatrix}$$

$\max(D/C_{shear}) = 0.13$

Botom Longitudinal Reinf. (LRFD 5.7.3.5)

Longitutinal Reinforcement:



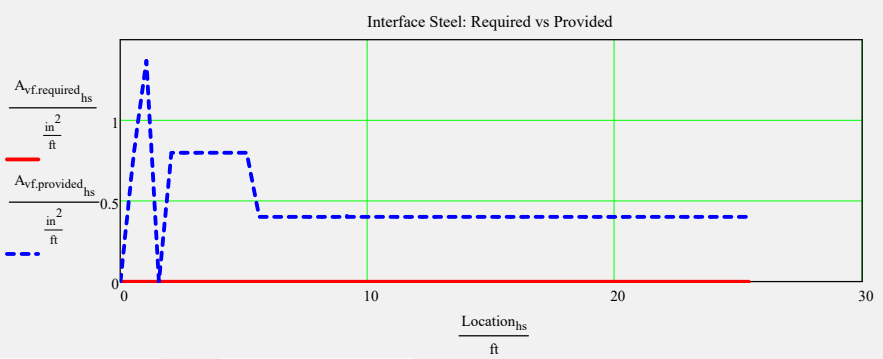
$$CheckD/CLong = \begin{pmatrix} \text{"CheckType"} & \text{"Location(ft)"} & \text{"V long Req'd (kip)"} & \text{"V long Prov (kip)"} \\ \text{"Longitudinal Reinf."} & 23.4 & 815.1 & 1625.5 \end{pmatrix}$$

$\max(D/C_{LongReinf}) = 0.50$ CheckLongReinf = "OK"

(LRFD 6.7.3.5)

Interface Shear Reinforcement

Interface Shear:



$TotalA_{vf.required} = 0 \cdot in^2$ $TotalA_{vf.provided} = 12.01 \cdot in^2$ $DC_{InterfaceReinf} = 0$ CheckInterfaceReinf = "OK"

Typically shear reinforcement is extended up into the deck slab.
These calculations are based on shear reinforcement functioning as interface reinforcing.
The interface_factor *can be used to adjust this assumption.*

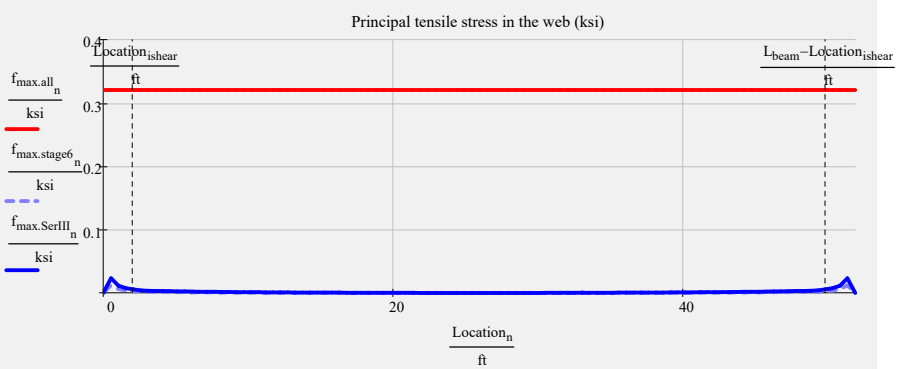
$\max(A_{vf.min.required}) = 0.000 \cdot \frac{in^2}{ft}$ $\max(A_{vf.des}) = 0.000 \cdot \frac{in^2}{ft}$

If max(Avf.min) or max(Avf.des) is greater than 0 in²/ft, interface reinforcement is required.

CheckInterfaceSpacing = "OK"

Pricipal Tensile Stresses in Webs - LRFD 5.9.2.3.3 and Splitting Resistance

Principal Stresses:



Crital Location for Principal Stress: $Location_{ishear} = 24.8 \cdot in$ $\max(DC_f_{max.web}) = 0.1$ $Check_f_{max.web} = "OK"$

Note: The check for pricipal tensile stresses in webs is only required for pretensioned girders with a compressive strength of concrete for use in design greater than 10 ksi per AASHTO BDS Article 5.9.2.3.3. The check is displayed for all concrete strengths for consideration when optimizing the strands design.

Splitting Reinforcement:

FDOT Splitting Rein Demand/Capacity Ratio and Check for FDOT Standard Beam End Reinforcement

$P_{r.standard} = 1460 \cdot kip$ $P_{prestress} = 1230 \cdot kip$ $D/C_{splitting.FDOT} = 0.84$

$CheckD/C_{splitting} = "OK"$

Confinement Bars:

$TotalNoConfineBars = 10$

Summary of Design Checks

$check_1 := "OK"$

$check_2 := CheckDeflectionOrSpanToDepth = "OK"$

$check_3 := CheckStrandFit = "OK"$

$check_4 := Check_f_{pt} = "OK"$

$check_5 := Check_f_{pe} = "OK"$

$check_6 := Check_f_{tension.rel} = "OK"$

$check_7 := Check_f_{comp.rel} = "OK"$

$check_8 := Check_f_{tension.ServiceIII} = "OK"$

$check_9 := Check_f_{comp.ServiceIa} = "OK"$

$check_{10} := Check_f_{comp.ServiceIb} = "OK"$

$check_{11} := Check_f_{comp.FatigueI} = "OK"$

$check_{12} := CheckMomentCapacity = "OK"$

$check_{13} := CheckMaxCapacity = "OK"$

$check_{14} := CheckStirArea = "OK"$

$check_{15} := CheckShearCapacity = "OK"$

$check_{16} := CheckMinStirArea = "OK"$

$check_{17} := CheckMaxStirSpacing = "OK"$

$check_{18} := CheckLongReinf = "OK"$

$check_{19} := CheckInterfaceSpacing = "OK"$

$check_{20} := CheckInterfaceReinf = "OK"$

$check_{21} := "OK"$

$check_{22} := CheckD/C_{splitting} = "OK"$

$check_{23} := CheckPattern_0 = "OK"$

$check_{24} := CheckPattern_1 = "OK"$

$check_{25} := CheckPattern_2 = "OK"$

$check_{26} := CheckPattern_3 = "OK"$

$check_{27} := CheckPattern_4 = "OK"$

$check_{28} := CheckWebPattern_0 = "OK"$

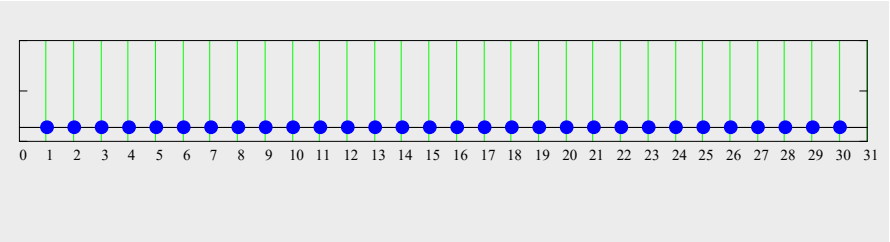
$check_{29} := CheckWebPattern_1 = "OK"$

$check_{30} := CheckBeamEndTopLongReinf = "OK"$

All Beam Checks

$check^T =$

	0	1	2	3	4	5	6	7	8	9	10
0	0	"OK"	"OK"	"OK"	"OK"	"OK"	"OK"	"OK"	"OK"	"OK"	...



$TotalCheck = "OK"$

Load Rating Analysis: Load and Resistance Factor Rating (LRFR)

ref.: Maintenance Office FDOT Bridge Load Rating Manual

Load Rating Computations

$$LRFR_{loadrating} = \begin{array}{c} \begin{array}{ccccccccc} & & & & & & \text{Moment (Strength) or Stress (Service)} & & \text{Shear (Strength)} \end{array} \\ \begin{pmatrix} 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 \\ 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 1.52 \\ 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 1.52 \\ 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 \\ 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 \\ 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 0.00 & 1.72 \times 10^{4} & 1.03 \times 10^{6} & 1.52 \end{pmatrix} \end{array} \quad \begin{pmatrix} \text{HL-93} \\ \text{HL-93} \\ \text{HL-93} \\ \text{HL-93*} \\ \text{Permit**} \end{pmatrix}$$

Notes:
** Not applicable for prestressed beams in good condition.*
*** FDOT Permit Rating truck is the FL120 .*
"Dim(ft)"*is measured from the centerline of bearings*

Longitudinal Reinforcement Check:

$DC_{LongReinf.HL93} = 0.50$ $DC_{LongReinf.Permit} = 0.29$ $CheckLongReinf_{loadrating} = \text{"OK"}$

QUICK CHECK: Factored Components of the Rating Factor Equation for the Design Operating Moment and Shear Ratings

$$HL93_{Operating.Moment} = \begin{pmatrix} \text{"}\varphi C\text{"} & 1493 & \text{"factored resistance (kip-ft)...."} \\ \text{"}\gamma DL\text{"} & 507 & \text{"factored dead load (kip-ft)...."} \\ \text{"}\gamma LL\text{"} & 281 & \text{"factored live load (kip-ft)....."} \\ \text{"RF"} & 3.51 & \text{"HL93 operating rating factor"} \end{pmatrix}$$

$$HL93_{Operating.Shear} = \begin{pmatrix} \text{"}\varphi C\text{"} & 514.3 & \text{"factored resistance (kip)....."} \\ \text{"}\gamma DL\text{"} & 38 & \text{"factored dead load (kip)....."} \\ \text{"}\gamma LL\text{"} & 22.6 & \text{"factored live load (kip)....."} \\ \text{"RF"} & 21.05 & \text{"HL93 operating rating factor"} \end{pmatrix}$$

2.3. Bearing Pad Design

Plain Elastomeric Pad (PEP) Design (LRFD 14.7.6)

Design is computed for all bearing locations on the Able Canal Pathway. FSB and bent cap designs are referenced for design input. FDOT SPI 450-450, AASHTO LRFD Bridge Design Specifications (LRFD), and FDOT Structures Design Guidelines (SDG) are referenced.

Pad Parameters

Pad Thickness: $t_{pad} := 1 \cdot in$

Pad Width: $w_{pad} := 42 \cdot in$

Pad Length: $b_{pad} := 8 \cdot in$

Shear Modulus of Elastomer: $G := 0.110 \cdot ksi$

Durometer Hardness: Durometer := 50

Shape Factor of Bearing: (LRFD 14.7.5.1) $S := \frac{b_{pad} \cdot w_{pad}}{2 \cdot t_{pad} \cdot (b_{pad} + w_{pad})}$ $S = 3.36$

Movement Parameters

Span Length: $Span := 53 \cdot ft$

Expansion Length: $L_{exp} := \frac{Span}{2}$ $L_{exp} = 318.00 \cdot in$

Concrete Compressive Strength: $f_c := 8.5$

Concrete Unit Weight: $w_c := 0.145$

Modulus of Elastcity: $E_c := 120000 \cdot (w_c)^2 \cdot f_c^{0.33} \cdot ksi$ $E_c = 5112 \cdot ksi$

Coefficient of Thermal Expansion: (LRFD 5.4.2.2) $\alpha := 6 \cdot 10^{-6} \cdot F^{-1}$

Thermal Gradient: (SDG 2.7) $\Delta T := 35 \cdot F$

Thermal Factor: (LRFD 14.7.5.3.2) $Factor := 65\%$

Thermal Load Factor: (LRFD Table 3.4.1-1) $\gamma_{TU} := 1.20$

Shrinkage Strain: (see 2.5 Joint Design for shrinkage calculation) $\varepsilon_{sh} := 0.00041$ OK. Negligible differences.

Load Data

Service Dead Load Reaction: (see following calculation, 2.2.2_Interior Beam) $DL := 31.5 \cdot kip$

Service Live Load Reaction: (see following calculation, 2.2.2_Interior Beam) $LL := 21.1 \cdot kip$

Service Temperature Load Reaction: (see following calculation, End Bent Loads) $TU := 3.64 \cdot kip$

Compressive Stress (LRFD 14.7.6.3.2)

Avg. Compressive Stress due to Service Load Combination: $\sigma_s := \frac{DL + LL}{w_{pad} \cdot b_{pad}}$ $\sigma_s = 0.157 \cdot ksi$

Allowable Compressive Stress: *LRFD 14.7.6.3.2* $\sigma_{allow} := \min(1.0 \cdot G \cdot S, 0.80 \cdot ksi)$ $\sigma_{allow} = 0.370 \cdot ksi$

Check Compressive Stress: $Check_\sigma_s := \begin{cases} \text{"OK"} & \text{if } \sigma_s \le \sigma_{allow} \\ \text{"No Good"} & \text{otherwise} \end{cases}$ $Check_\sigma_s = \text{"OK"}$

Compressive Deflection (LRFD 14.7.6.3.3)

Initial Dead Load Compressive Stress: $\sigma_D := \frac{DL}{w_{pad} \cdot b_{pad}}$ $\sigma_D = 0.094 \cdot ksi$

Initial Dead Load Compressive Strain: *LRFD 14.7.5.3.6* $\varepsilon_D := \frac{\sigma_D}{4.8 \cdot G \cdot S^2}$ $\varepsilon_D = 0.016 \cdot \frac{in}{in}$

Initial Dead Load Deflection: *LRFD C14.7.6.3.3* $\delta_D := 3 \cdot \varepsilon_D \cdot t_{pad}$ $\delta_D = 0.047 \cdot in$

Initial Live Load Compressive Stress: $\sigma_L := \frac{LL}{w_{pad} \cdot b_{pad}}$ $\sigma_D = 0.094 \cdot ksi$

Initial Live Load Compressive Strain: *LRFD 14.7.5.3.6* $\varepsilon_L := \frac{\sigma_L}{4.8 \cdot G \cdot S^2}$ $\varepsilon_L = 0.011 \cdot \frac{in}{in}$

Initial Dead Load Deflection: *LRFD C14.7.6.3.3* $\delta_L := 3 \cdot \varepsilon_L \cdot t_{pad}$ $\delta_L = 0.032 \cdot in$

Compressive Deflection (LRFD 14.7.6.3.3) Cont'd

Allowable Deflection: $\delta_{allow} := 0.09 \cdot t_{pad}$ $\delta_{allow} = 0.09 \cdot in$

Check Compressive Deflection: $Check_\delta_c := \begin{cases} \text{"OK"} & \text{if } \delta_D + \delta_L \le \delta_{allow} \\ \text{"No Good"} & \text{otherwise} \end{cases}$ $Check_\delta_c = \text{"OK"}$

Shear (LRFD 14.7.6.3.4, 14.7.5.3.2)

Shear Deformation due to Temperature: $\Delta_T := \left[(Factor \cdot \alpha \cdot \Delta T) + \varepsilon_{sh}\right] \cdot L_{exp}$ $\Delta_T = 0.174 \cdot in$

Maximum Shear Deformation of Bearing (LRFD 14.4): $\Delta_s := \gamma_{TU} \cdot \Delta_T$ $\Delta_s = 0.209 \cdot in$

Check Shear: $Check_\Delta_s := \begin{cases} \text{"OK"} & \text{if } 2 \cdot \Delta_s \le t_{pad} \\ \text{"No Good"} & \text{otherwise} \end{cases}$ $Check_\Delta_s = \text{"OK"}$

Stability (LRFD 14.7.6.3.6)

Check Stability: $Check_t_{pad} := \begin{cases} \text{"OK"} & \text{if } \frac{w_{pad}}{3} \ge t_{pad} \wedge \frac{b_{pad}}{3} \ge t_{pad} \\ \text{"No Good"} & \text{otherwise} \end{cases}$ $Check_t_{pad} = \text{"OK"}$

2.4. Build-Up and Deflection

KISINGER CAMPO & ASSOCIATES

LEE COUNTY
ABLE CANAL PATHWAY

DESIGNED BY: JHJ 03/22
CHECKED BY: LM 03/2022

Build-Up & Deflection Table

BUILD-UP AND DEFLECTION DATA TABLE FOR PRESTRESSED I-BEAMS

LOCATION		REQUIRED THEORETICAL BUILD-UP OVER CL BEAM			NET BEAM CAMBER (PRESTRESS - DEAD LOAD OF BEAM) @ 120 DAYS	DEAD LOAD DEFLECTION DURING DECK POUR @ 120 DAYS "DIM A"	BUILD-UP CASE NO.
SPAN NO.	BEAM NO.	AT BEGIN SPAN "DIM B"	AT CL SPAN "DIM C"	AT END SPAN "DIM D"			
1 through 5 •	1 & 3 •	0'-7 1/8" •	0'-6" •	0'-7 1/8" •	2.169 •	1.071 •	1
	2 •	0'-7 1/8" •	0'-6 1/8" •	0'-7 1/8" •	2.169 •	1.124 •	1

KISINGER CAMPO & ASSOCIATES

LEE COUNTY
ABLE CANAL PATHWAY

DESIGNED BY: JHJ 03/22
CHECKED BY: LM 03/2022

Build-Up & Deflection Table

Spans 1 through 5
Index 450-199: Build-Up Case 1: "B" = "C" + Net Camber - GSO

Beam Number	Location Along Span	PGL FGE (ft.)	Grade Sep. Ordinate (in.)	Camber @ 120 days (in.)	Non-Comp. DL Defl. @ Deck Pour (in.)	Net Camber @ Deck Pour (in.)	Dim. B, C, D	Calculated Build-Up (in.)	Recorded Build-Up (in.)
1	Begin	**22.790**	0.0000	2.169	1.071	1.098	B	7.0980	0'-7 1/8"
	Middle	**22.841**					C	**6.0000**	0'-6"
	End	**22.892**					D	7.0980	0'-7 1/8"
2	Begin	**22.790**	0.0000	2.169	1.124	1.044	B	7.0443	0'-7"
	Middle	**22.841**					C	**6.0000**	0'-6"
	End	**22.892**					D	7.0443	0'-7"
3	Begin	**22.790**	0.0000	2.169	1.071	1.098	B	7.0980	0'-7 1/8"
	Middle	**22.841**					C	**6.0000**	0'-6"
	End	**22.892**					D	7.0980	0'-7 1/8"

*Minimum build-up = 6.0 in per index

$t_{deck.delta}$ **Input**		
Spans	Beam	Difference in Build-Up at Ends to Mid-Span**
1-5	1	1.0980 in
	2	1.0443 in
	3	1.0980 in

** Difference in Build-Up at Ends to Mid-Span = (B - C)

2.5. Expansion Joint Design

SUPERSTRUCTURE DESIGN

JOINT DESIGN

CALCULATION QUALITY CONTROL		
RESPONSIBLE PROFESSIONAL / ORIGINATOR	INITIALS: JHJ	DATE: 02/22
QUALITY REVIEWER	INITIALS: LM	DATE: 02/2022

SUMMARY OF SECTION

Design of the ***Poured Joint with Backer Rod*** expansion joint is completed in accordance with FDOT Index 458-110 using supplemental calculations for the creep and shrinkage coefficient.

Poured Joint with Backer Rod Expansion Joint			
Expansion Joint Location	*DIM. "A" @ 70°F*	*Total Design Movement*	*DIM. "A" Adjustment Per 10°F **
End Bent 1	2.00"	0.110"	0.000"
Pier 2	2.00"	0.160"	0.000"
Pier 3	2.00"	0.160"	0.000"
Pier 4	2.00"	0.160"	0.000"
Pier 5	2.00"	0.160"	0.000"
End Bent 6	2.00"	0.110"	0.000"

* Index 450-450 (SPI) directs the Dimension "A" Adjustment Per 10°F to be set to 0" for FSB superstructures.

Shrinkage Calculation (LRFD 5.4.2.3.3)

AASHTO LRFD Bridge Design Specifications (LRFD) and FDOT Structures Design Guidelines (SDG) are referenced.

Bridge Information

Topping Thickness: $t_{top} := 6 \cdot in$

Pocket Area: $A_{pocket} := 132 \cdot in^2$ OK

Number of Pockets: $N_{pocket} := 3$

Concrete Compressive Strength: $f_c := 4.5 \cdot ksi$

Concrete Strength at Time of Initial Loading: $f_{ci} := 0.80 \cdot f_c$ $\quad f_{ci} = 3.6 \cdot ksi$

Time Dependent Information

Relative Humidity: (SDG 4.6.6) $H := 75$

Age of concrete when section becomes composite (days): $T_1 := 0$

Age of concrete for long term losses (days): $T_2 := 10000$

Volume-to-Surface Ratio Calculation

Deck Length: *(maximum use)* $L_{deck} := 26 \cdot ft + 6 \cdot in$

Deck Width: $b_{deck} := 13 \cdot ft + 7 \cdot in$ $\quad b_{deck} = 13.6 \cdot ft$

Deck Volume: $Volume := L_{deck} \cdot \left(b_{deck} \cdot t_{top} + N_{pocket} \cdot A_{pocket}\right)$ $\quad Volume = 253 \cdot ft^3$

Deck Surface Area: $Surface := 2 \cdot L_{deck} \cdot \left(b_{deck} + t_{top} + 44 \cdot in\right)$ $\quad Surface = 941\ ft^2$

Volume-to-Surface Ratio: $VS := \frac{Volume}{Surface}$ $\quad VS = 3.23 \cdot in$

Shrinkage Coefficient

- Factor for the effect of the volume-to-surface ratio (Eq. 5.4.2.3.2-2):

$k_s := \max(1.45 - 0.13 \cdot VS \div in, 1.0)$ $k_s = 1.03$

- Humidity factor for shrinkage:

$k_{hs} := 2.00 - 0.014 \cdot H$ $k_{hs} = 0.95$

- Factor for the effect of concrete strength:

$$k_f := \frac{5}{1 + \frac{f_{ci}}{ksi}}$$ $k_f = 1.09$

- Time development factor (*t* is concrete maturity in days):

$$k_{td}(t) := \frac{t}{12 \cdot \left[\frac{100 - 4 \cdot (f_{ci} \div ksi)}{(f_{ci} \div ksi) + 20}\right] + t}$$

$k_{td}(T_1) = 0.00$

$k_{td}(T_2) = 1.00$

- Shrinkage strain:

$\varepsilon_{sh}(t) := (0.00048) \cdot k_s \cdot k_{hs} \cdot k_f \cdot k_{td}(t)$

$\varepsilon_{sh}(T_1) = 0.00000$

$\varepsilon_{sh}(T_2) = 0.00051$

- Shrinkage strain on section from T_1 to T_2:

$\varepsilon_{SH} := \varepsilon_{sh}(T_2) - \varepsilon_{sh}(T_1)$ $\varepsilon_{SH} = 0.00051$

Expansion Joint Design

Expansion joints are designed in accordance with FDOT Indexes 458-100 and 458-110. This worksheet assumes a *straight bridge with direction of movement matching the direction of travel*. AASHTO LRFD Bridge Design Specifications (LRFD) and FDOT Structures Design Guidelines (SDG) are referenced.

Bridge Geometry

Continuous Deck/Slab Length Contributing to Joint Movement (Back): $Span_{Bk} := 10 \cdot ft + 0 \cdot in$

Continuous Deck/Slab Length Contributing to Joint Movement (Ahead): $Span_{Ah} := 26 \cdot ft + 6 \cdot in$

Skew Angle: $\phi := 0.00 \cdot deg$

Temperature, Shrinkage, and Creep Information

Temperature Rise:
(SDG Table 2.7.1-1) $\Delta T_{rise} := 35 \cdot F$

Temperature Fall:
(SDG Table 2.7.1-1) $\Delta T_{fall} := 35 \cdot F$

Coefficient of Thermal Expansion:
(LRFD 5.4.2.2) $\alpha := 6 \cdot 10^{-6} \cdot F^{-1}$

Shrinkage & Creep Strain:
(see independent calculation) $\varepsilon_{SC} := 0.00051$

Design Joint Information

Type of Expansion Joint: Joint := Poured Joint / Strip Seal — Joint = "Poured Joint"

Design Joint Opening, Dim. "A": $DimA := 2.00 \cdot in$

Joint Opening, Direction of Movement/Travel: $W := DimA \div \cos(\phi)$ — $W = 2.00 \cdot in$

Maximum Joint Opening: $W_{max} := \begin{cases} 4.0 \cdot in & \text{if Joint = "Strip Seal"} \\ 3.0 \cdot in & \text{otherwise} \end{cases}$ — $W_{max} = 3.00 \cdot in$

Minimum Joint Opening: $W_{min} := \begin{cases} 0.5 \cdot in & \text{if Joint = "Strip Seal"} \\ \max(0.5 \cdot W, 0.5 \cdot in) & \text{otherwise} \end{cases}$ — $W_{min} = 1.00 \cdot in$

Movement Calculations

Due to Shrinkage and Creep: $\Delta z_{SC} := \varepsilon_{SC} \cdot (0.5 \cdot Span_{Bk} + 0.5 \cdot Span_{Ah})$ $\Delta z_{SC} = 0.11 \cdot in$

Due to Temperature Rise: $\Delta z_{rise} := \alpha \cdot \Delta T_{rise} \cdot (0.5 \cdot Span_{Bk} + 0.5 \cdot Span_{Ah})$ $\Delta z_{rise} = 0.05 \cdot in$

Due to Temperature Fall: $\Delta z_{fall} := \alpha \cdot \Delta T_{fall} \cdot (0.5 \cdot Span_{Bk} + 0.5 \cdot Span_{Ah})$ $\Delta z_{fall} = 0.05 \cdot in$

Check Joint Openings

Movement tending to "open" the joint, due to the combined *factored* temperature, creep and shrinkage effects:

$W_{open} := W + 1.2 \cdot \Delta z_{fall} + \Delta z_{SC}$ $W_{open} = 2.17 \cdot in$

$$Check_{open} := \begin{cases} \text{"OK"} & \text{if } W_{open} \le W_{max} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$Check_{open} = \text{"OK"}$

Movement tending to "close" the joint, due to *factored* temperature effect:

$W_{close} := W - 1.2 \cdot \Delta z_{rise}$ $W_{close} = 1.94 \cdot in$

$$Check_{close} := \begin{cases} \text{"OK"} & \text{if } W_{close} \ge W_{min} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$Check_{close} = \text{"OK"}$

Dimension "A" Adjustment per 10°F

Index 458-110 (SPI): "Dimension A adjustments for 10°F shall be based on the *unfactored* movements perpendicular to the centerline of the joint."

$$T_{adjust} := \frac{\Delta z_{rise} \cdot \cos(\phi)}{\Delta T_{rise}} \cdot (10 \cdot F)$$

$T_{adjust} = 0.013 \cdot in$

Note: Index 450-450 (SPI) directs the Dimension "A" adjustment to be set to 0" for FSB superstructures.

Total Design Movement

Index 458-110 (SPI): "Total design movement is the absolute value of the total *factored* temperature movement (expansion and contraction) measured in the direction of movement. Note that creep and shrinkage may be neglected."

$Design_{total} := 1.2 \cdot (\Delta z_{rise} + \Delta z_{fall})$ $Design_{total} = 0.110 \cdot in$

Shrinkage Calculation (LRFD 5.4.2.3.3)

AASHTO LRFD Bridge Design Specifications (LRFD) and FDOT Structures Design Guidelines (SDG) are referenced.

Bridge Information

Topping Thickness:		$t_{top} := 6 \cdot in$
Pocket Area:		$A_{pocket} := 132 \cdot in^2$
Number of Pockets:		$N_{pocket} := 3$
Concrete Compressive Strength:		$f_c := 4.5 \cdot ksi$
Concrete Strength at Time of Initial Loading:	$f_{ci} := 0.80 \cdot f_c$	$f_{ci} = 3.6 \cdot ksi$

Time Dependent Information

Relative Humidity: (SDG 4.6.6)	$H := 75$
Age of concrete when section becomes composite (days):	$T_1 := 0$
Age of concrete for long term losses (days):	$T_2 := 10000$

Volume-to-Surface Ratio Calculation

Deck Length: *(maximum use)*		$L_{deck} := 53 \cdot ft$
Deck Width:	$b_{deck} := 13 \cdot ft + 7 \cdot in$	$b_{deck} = 13.6 \cdot ft$
Deck Volume:	$Volume := L_{deck} \cdot (b_{deck} \cdot t_{top} + N_{pocket} \cdot A_{pocket})$	$Volume = 506 \cdot ft^3$
Deck Surface Area:	$Surface := 2 \cdot L_{deck} \cdot (b_{deck} + t_{top} + 44 \cdot in)$	$Surface = 1882\ ft^2$
Volume-to-Surface Ratio:	$VS := \frac{Volume}{Surface}$	$VS = 3.23 \cdot in$

Shrinkage Coefficient

- Factor for the effect of the volume-to-surface ratio (Eq. 5.4.2.3.2-2):

$k_s := \max(1.45 - 0.13 \cdot VS \div in, 1.0)$ $k_s = 1.03$

- Humidity factor for shrinkage:

$k_{hs} := 2.00 - 0.014 \cdot H$ $k_{hs} = 0.95$

- Factor for the effect of concrete strength:

$$k_f := \frac{5}{1 + \frac{f_{ci}}{ksi}}$$ $k_f = 1.09$

- Time development factor (*t* is concrete maturity in days):

$$k_{td}(t) := \frac{t}{12 \cdot \left[\frac{100 - 4 \cdot (f_{ci} \div ksi)}{(f_{ci} \div ksi) + 20}\right] + t}$$

$k_{td}(T_1) = 0.00$

$k_{td}(T_2) = 1.00$

- Shrinkage strain:

$\varepsilon_{sh}(t) := (0.00048) \cdot k_s \cdot k_{hs} \cdot k_f \cdot k_{td}(t)$

$\varepsilon_{sh}(T_1) = 0.00000$

$\varepsilon_{sh}(T_2) = 0.00051$

- Shrinkage strain on section from T_1 to T_2:

$\varepsilon_{SH} := \varepsilon_{sh}(T_2) - \varepsilon_{sh}(T_1)$ $\varepsilon_{SH} = 0.00051$

Expansion Joint Design

Expansion joints are designed in accordance with FDOT Indexes 458-100 and 458-110. This worksheet assumes a *straight bridge with direction of movement matching the direction of travel*. AASHTO LRFD Bridge Design Specifications (LRFD) and FDOT Structures Design Guidelines (SDG) are referenced.

Bridge Geometry

Continuous Deck/Slab Length Contributing to Joint Movement (Back): $Span_{Bk} := 26 \cdot ft + 6 \cdot in$

Continuous Deck/Slab Length Contributing to Joint Movement (Ahead): $Span_{Ah} := 26 \cdot ft + 6 \cdot in$

Skew Angle: $\phi := 0.00 \cdot deg$

Temperature, Shrinkage, and Creep Information

Temperature Rise:
(SDG Table 2.7.1-1) $\Delta T_{rise} := 35 \cdot F$

Temperature Fall:
(SDG Table 2.7.1-1) $\Delta T_{fall} := 35 \cdot F$

Coefficient of Thermal Expansion:
(LRFD 5.4.2.2) $\alpha := 6 \cdot 10^{-6} \cdot F^{-1}$

Shrinkage & Creep Strain:
(see independent calculation) $\varepsilon_{SC} := 0.00051$

Design Joint Information

Type of Expansion Joint: Joint := Poured Joint / Strip Seal — Joint = "Poured Joint"

Design Joint Opening, Dim. "A": $DimA := 2.00 \cdot in$

Joint Opening, Direction of Movement/Travel: $W := DimA \div \cos(\phi)$ — $W = 2.00 \cdot in$

Maximum Joint Opening: $W_{max} := \begin{cases} 4.0 \cdot in & \text{if Joint = "Strip Seal"} \\ 3.0 \cdot in & \text{otherwise} \end{cases}$ — $W_{max} = 3.00 \cdot in$

Minimum Joint Opening: $W_{min} := \begin{cases} 0.5 \cdot in & \text{if Joint = "Strip Seal"} \\ \max(0.5 \cdot W, 0.5 \cdot in) & \text{otherwise} \end{cases}$ — $W_{min} = 1.00 \cdot in$

Movement Calculations

Due to Shrinkage and Creep: $\Delta z_{SC} := \varepsilon_{SC} \cdot (0.5 \cdot Span_{Bk} + 0.5 \cdot Span_{Ah})$ $\Delta z_{SC} = 0.16 \cdot in$

Due to Temperature Rise: $\Delta z_{rise} := \alpha \cdot \Delta T_{rise} \cdot (0.5 \cdot Span_{Bk} + 0.5 \cdot Span_{Ah})$ $\Delta z_{rise} = 0.07 \cdot in$

Due to Temperature Fall: $\Delta z_{fall} := \alpha \cdot \Delta T_{fall} \cdot (0.5 \cdot Span_{Bk} + 0.5 \cdot Span_{Ah})$ $\Delta z_{fall} = 0.07 \cdot in$

Check Joint Openings

Movement tending to "open" the joint, due to the combined *factored* temperature, creep and shrinkage effects:

$W_{open} := W + 1.2 \cdot \Delta z_{fall} + \Delta z_{SC}$ $W_{open} = 2.24 \cdot in$

$$Check_{open} := \begin{cases} \text{"OK"} & \text{if } W_{open} \le W_{max} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$Check_{open} = \text{"OK"}$

Movement tending to "close" the joint, due to *factored* temperature effect:

$W_{close} := W - 1.2 \cdot \Delta z_{rise}$ $W_{close} = 1.92 \cdot in$

$$Check_{close} := \begin{cases} \text{"OK"} & \text{if } W_{close} \ge W_{min} \\ \text{"NG"} & \text{otherwise} \end{cases}$$

$Check_{close} = \text{"OK"}$

Dimension "A" Adjustment per 10°F

Index 458-110 (SPI): "Dimension A adjustments for 10°F shall be based on the *unfactored* movements perpendicular to the centerline of the joint."

$$T_{adjust} := \frac{\Delta z_{rise} \cdot \cos(\phi)}{\Delta T_{rise}} \cdot (10 \cdot F)$$

$T_{adjust} = 0.019 \cdot in$

Note: Index 450-450 (SPI) directs the Dimension "A" adjustment to be set to 0" for FSB superstructures.

Total Design Movement

Index 458-110 (SPI): "Total design movement is the absolute value of the total *factored* temperature movement (expansion and contraction) measured in the direction of movement. Note that creep and shrinkage may be neglected."

$Design_{total} := 1.2 \cdot (\Delta z_{rise} + \Delta z_{fall})$ $Design_{total} = 0.160 \cdot in$

3. Substructure Design

3.1. End Bent Design

PROJECT DATA

PROJECT DATA

Project:	Able Canal Pathway
User Job No.:	
Designer:	BAH
Date:	11/2021
Checker:	
Checked date:	
State:	FL
State Job No.:	
Structure type:	Abutment.
State Specification:	None
Code:	AASHTO LRFD 8th Edition
Comments:	End bent design.

ABUTMENT GEOMETRY

Abutment Shape

Abutment Shape: Pile Cap
Frame Analysis Method: Cap as continuous beam
Top Elevations: start = 21.00 ft end = 21.00 ft
Skew angle = 0.00 Reduction of I = 1.000
Length = 15.75 ft
BW = 0.00 in WD = 0.00 in
SW = 42.00 in SD = 30.00 in

SUPERSTRUCTURE INFO

Superstructure info:

Total number of spans:	1	
Span number rear to current pier:	0	
Number of traffic lanes:	1	
Barrier height :	32.00 in	75% railing height to account for wind reduction due to open picket railing.
Depth of slab :	12.00 in	Includes height of railing support.
Curb to curb distance:	12.000 ft	

Beam info:

Height in	Section area in^2	Inertia (Ixx) in^4	Inertia (Iyy) in^4	Beam CG in
15.00	671.76	12950.00	116990.00	7.09

Span #	Span length ft	Bridge Width ft
1	53.000	13.583
		13.583

BEARING POINTS

Number of bearing lines: 1

First bearing line **Eccentricity = 0.46 ft**

Point	Distance ft
1	3.29
2	7.87
3	12.46

MATERIAL PROPERTIES

MATERIAL PROPERTIES

	Cap	Column	Footing
Concrete Type	normal	normal	normal
Concrete Strength (psi)	5500.00	6000.00	3400.00
Concrete Density (lb/ft3)	150.00	150.00	150.00
Concrete Modulus Ec (ksi)	4428.30	4557.30	3778.38
Steel Strength Fy (ksi)	60.00	60.00	60.00

DESIGN PARAMETERS

Resistance factors for reinf. concrete

Flexure and tension	0.90
Shear and torsion (normal)	0.90
Shear and torsion (lightweight)	0.90
Axial compression (ties)	0.75
Axial compression (spiral)	0.75
Compression in STM	0.70

Multi presence factors for live load

1 Lane	1.00
2 Lanes	1.00
3 Lanes	0.85
4 Lanes	0.65
5 Lanes	0.65
6 Lanes	0.65

Dynamic load allowance IM

	Truck	Lane	Fatigue
Cap	0.00	0.00	0.15
Column	0.00	0.00	0.15
Footing	0.00	0.00	0.00

	Exposure factors	Clear cover in	Clear side cover in
Cap	1.00	3.00	3.00
Column	1.00	3.00	
Footing	1.00	3.00	3.00

Degree of fixity in foundations for Moment Magnify Method: Ga = 5.00

SEISMIC DESIGN PARAMETERS

Strength Reduction factors for reinf. Concrete Seismic Design

Tension controlled	0.90
Shear and torsion (normal)	0.90
Shear and torsion (lightweight)	0.70
Compression Controlled (ties)	0.75
Compression Controlled (spiral)	0.75

Seismic Overstrength

Flexure and tension	1.30
Axial compression (ties)	1.30
Axial compression (spiral)	1.30

Response Modification Factor	1.00

Use core area for plastic hinging calculations.

Design Factors

Cap Design Factor	1.20
Footing Design Factor	1.20

Plastic Hinge Moment

Use actual computed Plastic Hinging Moment for each column in all combinations.

LOADS

Pier Info:

Pier View:	Downstation.

Load Cases: 15

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:	H-10

Tranverse Positioning

Number of loaded lanes	1 Lane Loaded
Live Load Positions	Constant Spacing
Minimum Distance from Curb	2.000000
Center to Center Spacing	10.000000

Generate Braking/Longitudinal Force	Not Selected
Generate Centrifugal Force	Not Selected

Number of single regular truck positions	2
Number of all possible combinations of regular truck positions	2

Loadcase ID: DC1 Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	Y	-1.75	-4.64 kips	0.00		
Force	Y	-1.75	-4.64 kips	1.00		
UDL	Y		-0.94 klf	0.07		0.93
Moment	X		-16.10 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	-33.40
1	2	Y	-33.40
1	3	Y	-33.40

Loadcase ID: TU1 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Z	-3.64
1	2	Z	-3.64
1	3	Z	-3.64

Loadcase ID: LL1 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips	Lane Load
1	1	Y	-8.27	
1	2	Y	-6.54	
1	3	Y	-4.13	
1	1	Y	0.00	L
1	2	Y	0.00	L
1	3	Y	0.00	L

Auto generation details:

Generated Live Load

TRUCK IDENTIFICATION : H-10
Positions of the center of each load measured from the left of the bridge :

TRUCK TYPE : Regular
Truck No. 1 - center line at: 5.79 - 2 axle on bearing line No. 1
Multi lane reduction factor used : 1.0000

Maximum effects in members :

Member 1	node 1	Fx = 11.3660
Member 1	node 2	Fx = -11.3660
Member 5	node 2	Fy = 11.3660
Member 5	node 7	Mz = 4.7359
Member 5	node 7	Fy = -11.3660
Member 6	node 7	Mz = -4.7359
Member 6	node 8	Mz = 5.7692
Member 7	node 8	Mz = -5.7692
Member 8	node 9	Fy = -3.4442
Member 8	node 10	Fy = 3.4442
Member 9	node 10	Fy = -3.4442
Member 9	node 11	Fy = 3.4442

Loadcase ID: LL2 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips	Lane Load
1	1	Y	-4.13	
1	2	Y	-6.54	
1	3	Y	-8.27	
1	1	Y	0.00	L
1	2	Y	0.00	L
1	3	Y	0.00	L

Auto generation details:

Generated Live Load

TRUCK IDENTIFICATION : H-10
Positions of the center of each load measured from the left of the bridge :

TRUCK TYPE : Regular
Truck No. 1 - center line at: 7.79 - 2 axle on bearing line No. 1
Multi lane reduction factor used : 1.0000

Maximum effects in members :

Member 2	node 3	Fx = 11.3660
Member 2	node 4	Fx = -11.3660
Member 6	node 7	Fy = 3.4443
Member 6	node 8	Fy = -3.4443
Member 7	node 8	Fy = 3.4443
Member 7	node 9	Mz = 18.9435
Member 7	node 9	Fy = -3.4443
Member 8	node 9	Mz = -18.9435
Member 8	node 10	Mz = 5.7696
Member 9	node 10	Mz = -5.7696
Member 9	node 11	Mz = 4.7366
Member 10	node 11	Fy = -11.3660
Member 10	node 11	Mz = -4.7366
Member 10	node 4	Fy = 11.3660

Loadcase ID: LL3 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips	Lane Load
1	1	Y	-9.54	
1	2	Y	-9.54	
1	3	Y	-9.54	

Note: Vertical Wind Loads per AASHTO LRFD 3.8.2 not included within Leap Bridge Concrete Substructure calculation in order to provide conservative pile reactions.

Loadcase ID: WS1 Name: STR III-Angle: 0

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	3.55
1	1	Y	2.86
1	1	Z	-0.00
1	2	X	3.55
1	2	Y	-0.00
1	2	Z	-0.00
1	3	X	3.55
1	3	Y	-2.86
1	3	Z	-0.00

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 0.00 deg Elevation above which wind load acts = 10.00 ft

Default wind pressure:

For STR III

Wind pressure for superstructure: Pz = 81.750 psf

Wind pressure for substructure: Pz = 100.620 psf

Overturning not considered

For SER IV

Wind pressure for superstructure: Pz = 45.990 psf

Wind pressure for substructure: Pz = 56.600 psf

Overturning not considered

For STR V

Wind pressure for superstructure: Pz = 21.190 psf

Wind pressure for substructure: Pz = 26.080 psf

For SER I

Wind pressure for superstructure: Pz = 16.250 psf

Wind pressure for substructure: Pz = 20.000 psf

Note: Vertical Wind Loads per AASHTO LRFD 3.8.2 not included within Leap Bridge Concrete Substructure calculation in order to provide conservative pile reactions.

Loadcase ID: WS2 Name: STR III-Angle: 15

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	3.12
1	1	Y	2.51
1	1	Z	-0.43
1	2	X	3.12
1	2	Y	-0.00
1	2	Z	-0.43
1	3	X	3.12
1	3	Y	-2.51
1	3	Z	-0.43

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 15.00 deg Elevation above which wind load acts = 10.00 ft
Default wind pressure:

	Wind pressure for superstructure:	Wind pressure for substructure:
For STR III	Pz = 81.750 psf	Pz = 100.620 psf
Overturning not considered		
For SER IV	Pz = 45.990 psf	Pz = 56.600 psf
Overturning not considered		
For STR V	Pz = 21.190 psf	Pz = 26.080 psf
For SER I	Pz = 16.250 psf	Pz = 20.000 psf

Loadcase ID: WS3 Name: STR III-Angle: 30

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	2.91
1	1	Y	2.34
1	1	Z	-0.85
1	2	X	2.91
1	2	Y	-0.00
1	2	Z	-0.85
1	3	X	2.91
1	3	Y	-2.34
1	3	Z	-0.85

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 30.00 deg Elevation above which wind load acts = 10.00 ft	
Default wind pressure:	
For STR III	
Wind pressure for superstructure: Pz = 81.750 psf	Wind pressure for substructure: Pz = 100.620 psf
Overturning not considered	
For SER IV	
Wind pressure for superstructure: Pz = 45.990 psf	Wind pressure for substructure: Pz = 56.600 psf
Overturning not considered	
For STR V	
Wind pressure for superstructure: Pz = 21.190 psf	Wind pressure for substructure: Pz = 26.080 psf
For SER I	
Wind pressure for superstructure: Pz = 16.250 psf	Wind pressure for substructure: Pz = 20.000 psf

Loadcase ID: WS4 Name: STR III-Angle: 45

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	2.34
1	1	Y	1.89
1	1	Z	-1.14
1	2	X	2.34
1	2	Y	-0.00
1	2	Z	-1.14
1	3	X	2.34
1	3	Y	-1.89
1	3	Z	-1.14

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 45.00 deg Elevation above which wind load acts = 10.00 ft	
Default wind pressure:	
For STR III	
Wind pressure for superstructure: Pz = 81.750 psf	Wind pressure for substructure: Pz = 100.620 psf
Overturning not considered	
For SER IV	
Wind pressure for superstructure: Pz = 45.990 psf	Wind pressure for substructure: Pz = 56.600 psf
Overturning not considered	
For STR V	
Wind pressure for superstructure: Pz = 21.190 psf	Wind pressure for substructure: Pz = 26.080 psf
For SER I	
Wind pressure for superstructure: Pz = 16.250 psf	Wind pressure for substructure: Pz = 20.000 psf

Loadcase ID: WS5 Name: STR III-Angle: 60

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	1.21
1	1	Y	0.97
1	1	Z	-1.35
1	2	X	1.21
1	2	Y	-0.00
1	2	Z	-1.35
1	3	X	1.21
1	3	Y	-0.97
1	3	Z	-1.35

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 60.00 deg Elevation above which wind load acts = 10.00 ft Default wind pressure:	
For STR III Wind pressure for superstructure: Pz = 81.750 psf Overturning not considered	Wind pressure for substructure: Pz = 100.620 psf
For SER IV Wind pressure for superstructure: Pz = 45.990 psf Overturning not considered	Wind pressure for substructure: Pz = 56.600 psf
For STR V Wind pressure for superstructure: Pz = 21.190 psf	Wind pressure for substructure: Pz = 26.080 psf
For SER I Wind pressure for superstructure: Pz = 16.250 psf	Wind pressure for substructure: Pz = 20.000 psf

Loadcase ID: WS16 Name: SER I-Angle: 0

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	0.71
1	1	Y	0.57
1	1	Z	-0.00
1	2	X	0.71
1	2	Y	-0.00
1	2	Z	-0.00
1	3	X	0.71
1	3	Y	-0.57
1	3	Z	-0.00

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 0.00 deg Elevation above which wind load acts = 10.00 ft

Default wind pressure:

	Wind pressure for superstructure:	Wind pressure for substructure:
For STR III	Pz = 81.750 psf	Pz = 100.620 psf
Overturning not considered		
For SER IV	Pz = 45.990 psf	Pz = 56.600 psf
Overturning not considered		
For STR V	Pz = 21.190 psf	Pz = 26.080 psf
For SER I	Pz = 16.250 psf	Pz = 20.000 psf

Loadcase ID: WS17 Name: SER I-Angle: 15

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	0.62
1	1	Y	0.50
1	1	Z	-0.08
1	2	X	0.62
1	2	Y	-0.00
1	2	Z	-0.08
1	3	X	0.62
1	3	Y	-0.50
1	3	Z	-0.08

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 15.00 deg Elevation above which wind load acts = 10.00 ft

Default wind pressure:

For STR III

Wind pressure for superstructure: Pz = 81.750 psf

Wind pressure for substructure: Pz = 100.620 psf

Overturning not considered

For SER IV

Wind pressure for superstructure: Pz = 45.990 psf

Wind pressure for substructure: Pz = 56.600 psf

Overturning not considered

For STR V

Wind pressure for superstructure: Pz = 21.190 psf

Wind pressure for substructure: Pz = 26.080 psf

For SER I

Wind pressure for superstructure: Pz = 16.250 psf

Wind pressure for substructure: Pz = 20.000 psf

Loadcase ID: WS18 Name: SER I-Angle: 30

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	0.58
1	1	Y	0.47
1	1	Z	-0.17
1	2	X	0.58
1	2	Y	-0.00
1	2	Z	-0.17
1	3	X	0.58
1	3	Y	-0.47
1	3	Z	-0.17

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 30.00 deg Elevation above which wind load acts = 10.00 ft
Default wind pressure:

	Wind pressure for superstructure:	Wind pressure for substructure:
For STR III	Pz = 81.750 psf	Pz = 100.620 psf
Overturning not considered		
For SER IV	Pz = 45.990 psf	Pz = 56.600 psf
Overturning not considered		
For STR V	Pz = 21.190 psf	Pz = 26.080 psf
For SER I	Pz = 16.250 psf	Pz = 20.000 psf

Loadcase ID: WS19 Name: SER I-Angle: 45

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	0.47
1	1	Y	0.37
1	1	Z	-0.23
1	2	X	0.47
1	2	Y	-0.00
1	2	Z	-0.23
1	3	X	0.47
1	3	Y	-0.37
1	3	Z	-0.23

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 45.00 deg Elevation above which wind load acts = 10.00 ft	
Default wind pressure:	
For STR III	
Wind pressure for superstructure: Pz = 81.750 psf	Wind pressure for substructure: Pz = 100.620 psf
Overturning not considered	
For SER IV	
Wind pressure for superstructure: Pz = 45.990 psf	Wind pressure for substructure: Pz = 56.600 psf
Overturning not considered	
For STR V	
Wind pressure for superstructure: Pz = 21.190 psf	Wind pressure for substructure: Pz = 26.080 psf
For SER I	
Wind pressure for superstructure: Pz = 16.250 psf	Wind pressure for substructure: Pz = 20.000 psf

Loadcase ID: WS20 Name: SER I-Angle: 60

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	0.24
1	1	Y	0.19
1	1	Z	-0.27
1	2	X	0.24
1	2	Y	-0.00
1	2	Z	-0.27
1	3	X	0.24
1	3	Y	-0.19
1	3	Z	-0.27

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 60.00 deg Elevation above which wind load acts = 10.00 ft
Default wind pressure:

For STR III

Wind pressure for superstructure:
Pz = 81.750 psf

Wind pressure for substructure:
Pz = 100.620 psf

Overturning not considered

For SER IV

Wind pressure for superstructure:
Pz = 45.990 psf

Wind pressure for substructure:
Pz = 56.600 psf

Overturning not considered

For STR V

Wind pressure for superstructure:
Pz = 21.190 psf

Wind pressure for substructure:
Pz = 26.080 psf

For SER I

Wind pressure for superstructure:
Pz = 16.250 psf

Wind pressure for substructure:
Pz = 20.000 psf

Selected load groups

STRENGTH GROUP I
STRENGTH GROUP III
SERVICE GROUP I
SERVICE GROUP III

STRUCTURE MODEL

Frame Model Structure:

	Member	Node	Hinge	Check Pt	Dist ft	Memb length ft
Column No. 1						
	1	1	-		0.00	
		2	YZ		21.25	21.25
Column No. 2						
	2	3	-		0.00	
		4	YZ		21.25	21.25
Cap						
	3	5	-		0.00	
		6	-	f	2.13	2.13
	4	6	-	f	2.13	
		2	-		2.88	0.75
	5	2	-		2.88	
		7	-		3.29	0.42
	6	7	-		3.29	
		8	-	f	3.63	0.33
	7	8	-	f	3.63	
		9	-		7.87	4.25
	8	9	-		7.87	
		10	-	f	12.13	4.25
	9	10	-	f	12.13	
		11	-		12.46	0.33
	10	11	-		12.46	
		4	-		12.88	0.42
	11	4	-		12.88	
		12	-	f	13.63	0.75
	12	12	-	f	13.63	
		13	-		15.75	2.13

Node coordinates

Number	X ft	Y ft	Node type
1	2.88	-1.50	fixed at ground
2	2.88	19.75	column-cap
3	12.88	-1.50	fixed at ground
4	12.88	19.75	column-cap
5	0.00	19.75	
6	2.13	19.75	face of support
7	3.29	19.75	bearing
8	3.63	19.75	face of support
9	7.87	19.75	bearing
10	12.13	19.75	face of support
11	12.46	19.75	bearing
12	13.63	19.75	face of support
13	15.75	19.75	

CAP DESIGN

Code: AASHTO LRFD 8
Units: US
Abutment View: Downstation.

DESIGN PARAMETERS

f'c = 5500.0 psi	
Fy flex = 60000.0 psi	Fy shear = 60000.0 psi
phi tens = 0.90	
phi comp = 0.75	phi shear = 0.90
Tens above = 0.005	Comp below = 0.002
Ec = 4428.3 ksi	Es = 29000.0 ksi
Crack check as per current LRFD	
Crack control Exposure = 1.00	
Concrete Type : Normal Weight.	
Design of cap at face of stem.	

CAP GEOMETRY

Straight Cap : Length(X) = 15.75 ft Depth(Z) = 42.00 in

Cap Section Properties

Sec.	Area ft ^2	Iz in ^4	Iy in ^4
1	8.75	94500.00	185220.00

MAIN REINFORCEMENT

	Bar size	Quantity	Bar dist. in	As total in^2	From ft	To ft	Hook
TOP							
	US#5[M16]	3	4.00	0.930	0.00	15.75	Both 90
BOTTOM							
	US#6[M19]	4	5.00	1.760	0.00	15.75	Both 90
	US#6[M19]	4	9.00	1.760	0.00	15.75	Both 90

STIRRUPS

From ft	To ft	Stirrup Size	n legs	Spacing in	Aprv/s in^2 / ft	Bar Type
0.00	15.75	US#4[M13]	2	12.00	0.40	Stirrup

Clear Cover on Sides = 3.00 in

FLEXURE DESIGN

Span 1: From 0.00 ft To 2.88 ft

Loc ft	AbsLoc ft	H in	Mmax Mmin kips-ft	Mr kips-ft	Comb	CL	c	dt in	eps_t	Phi	Asb-req in^2	Asb-prv in^2	Asb-eff in^2	Ast-req in^2	Ast-prv in^2	Ast-eff in^2
2.1	2.1	30	0.0	367.4	0	T	1.75	23.00	0.036	0.90	0.66	3.52	3.52	0.66	0.93	0.93
			-16.6	-206.1	1	T	1.75	26.00	0.041	0.90	0.66	3.52	3.52	0.66	0.93	0.93

Span 2: From 2.88 ft To 12.88 ft

Loc ft	AbsLoc ft	H in	Mmax Mmin kips-ft	Mr kips-ft	Comb	CL	c	dt in	eps_t	Phi	Asb-req in^2	Asb-prv in^2	Asb-eff in^2	Ast-req in^2	Ast-prv in^2	Ast-eff in^2
0.8	3.6	30	30.7	367.4	3	T	1.75	23.00	0.036	0.90	0.66	3.52	3.52	0.66	0.93	0.93
			0.0	-206.1	0	T	1.75	26.00	0.041	0.90	0.66	3.52	3.52	0.66	0.93	0.93
5.0	7.9	30	180.4	367.4	3	T	1.75	23.00	0.036	0.90	2.35	3.52	3.52	0.66	0.93	0.93
			0.0	-206.1	0	T	1.75	26.00	0.041	0.90	0.66	3.52	3.52	0.66	0.93	0.93
9.3	12.1	30	30.7	367.4	3	T	1.75	23.00	0.036	0.90	0.66	3.52	3.52	0.66	0.93	0.93
			0.0	-206.1	0	T	1.75	26.00	0.041	0.90	0.66	3.52	3.52	0.66	0.93	0.93

Span 3: From 12.88 ft To 15.75 ft

Loc ft	AbsLoc ft	H in	Mmax Mmin kips-ft	Mr kips-ft	Comb	CL	c	dt in	eps_t	Phi	Asb-req in^2	Asb-prv in^2	Asb-eff in^2	Ast-req in^2	Ast-prv in^2	Ast-eff in^2
0.8	13.6	30	0.0	367.4	0	T	1.75	23.00	0.036	0.90	0.66	3.52	3.52	0.66	0.93	0.93
			-16.6	-206.1	1	T	1.75	26.00	0.041	0.90	0.66	3.52	3.52	0.66	0.93	0.93

Flexure Design : Notes

CL: Section classification as per LRFD 2006 interims for provided reinforcement.
C = Compression controlled, I = In-Transition, T = Tension controlled.

SHEAR AND TORSION DESIGN:

Span 1: From 0.00 ft To 2.88 ft

Loc ft	AbsLoc ft	Pos	Vu kips	Comb	Tu kips-ft	Comb	phi*Vn kips	T-lim kips-ft	Avs/s in^2/ft	Ats/s in^2/ft	Av/s in^2/ft	Aprv/s in^2/ft	Alx in^2	Vc kips	Vs kips	Beta	Theta deg	b in	dv in	Eps_s
2.13	2.13	L	10.5	1	0.0	0	288.9	61.1	0.00	0.00	0.00	0.40	0.00	239.10	81.92	3.03	31.72	42.00	25.32	0.0008

Span 2: From 2.88 ft To 12.88 ft

Loc ft	AbsLoc ft	Pos	Vu kips	Comb	Tu kips-ft	Comb	phi*Vn kips	T-lim kips-ft	Avs/s in^2/ft	Ats/s in^2/ft	Av/s in^2/ft	Aprv/s in^2/ft	Alx in^2	Vc kips	Vs kips	Beta	Theta deg	b in	dv in	Eps_s
0.75	3.63	R	41.2	3	0.0	0	252.7	61.2	0.00	0.00	0.00	0.40	0.00	208.74	72.01	3.00	31.79	42.00	22.32	0.0008
5.00	7.87	L	29.2	3	0.0	0	217.5	61.2	0.00	0.00	0.00	0.40	0.00	173.75	67.97	2.50	33.30	42.00	22.32	0.0012
		R	29.2	3	0.0	0	216.4	60.9	0.00	0.00	0.00	0.40	0.00	172.58	67.81	2.48	33.36	42.00	22.32	0.0012
9.25	12.13	L	41.2	3	0.0	0	251.0	60.9	0.00	0.00	0.00	0.40	0.00	207.05	71.84	2.98	31.85	42.00	22.32	0.0008

Span 3: From 12.88 ft To 15.75 ft

Loc ft	AbsLoc ft	Pos	Vu kips	Comb	Tu kips-ft	Comb	phi*Vn kips	T-lim kips-ft	Avs/s in^2/ft	Ats/s in^2/ft	Av/s in^2/ft	Aprv/s in^2/ft	Alx in^2	Vc kips	Vs kips	Beta	Theta deg	b in	dv in	Eps_s
0.75	13.63	R	10.5	1	0.0	0	288.9	61.1	0.00	0.00	0.00	0.40	0.00	239.10	81.92	3.03	31.72	42.00	25.32	0.0008

Shear and Torsion Design : Notes

- Pos is the design position. L suggests the calculation is done at immediate left of "Loc" and R suggests at immediate right of it.
- T-lim is the limiting value of torsion for the concrete section. If actual torsion is higher than this value, torsional steel has to be provided.
- Avs/s is the required area of steel per unit length for shear force.
- Ats/s is the required area of steel per unit length for one leg of torsional reinforcement.
- Av/s is the total required area of steel per unit length due to shear plus torsion.
- Aprvs/s is the total provided area of transverse steel reinforcement.
- Alx is the EFFECTIVE longitudinal steel required in addition to the PROVIDED EFFECTIVE flexural steel.
- Vc is the nominal shear resistence of concrete.
- Vs is the nominal shear resistence of transverse reinforcement.
- Beta is the factor indicating ability of diagonally cracked concrete to transmit tension and shear.
- Theta is the angle of inclination of diagonal compressive stress.
- # Vu is greater tha phi*Vn.

CRACKING/FATIGUE CHECK

Span 1: From 0.00 ft To 2.88 ft

Loc ft	AbsLoc ft	H in	Cracking Comb	Cracking fs-t fs-b ksi	Cracking dc in	Cracking Srqt Srqb in	Cracking Sprt Sprb in	Fatigue stress range fs-t fs-b ksi	Fatigue threshold FTH-t FTH-b ksi
2.13	2.1	30.0	27	5.6	2.3	42.0	17.2	0.00	0.00
			0	0.0	2.4	5.0	11.4	0.00	0.00

Span 2: From 2.88 ft To 12.88 ft

Loc ft	AbsLoc ft	H in	Cracking Comb	Cracking fs-t fs-b ksi	Cracking dc in	Cracking Srqt Srqb in	Cracking Sprt Sprb in	Fatigue stress range fs-t fs-b ksi	Fatigue threshold FTH-t FTH-b ksi
0.75	3.6	30.0	0	0.0	2.3	5.0	17.2	0.00	0.00
			29	3.5	2.4	42.0	11.4	0.00	0.00
5.00	7.9	30.0	0	0.0	2.3	5.0	17.2	0.00	0.00
			29	21.2	2.4	24.6	11.4	0.00	0.00
9.25	12.1	30.0	0	0.0	2.3	5.0	17.2	0.00	0.00
			44	3.5	2.4	42.0	11.4	0.00	0.00

Span 3: From 12.88 ft To 15.75 ft

Loc ft	AbsLoc ft	H in	Cracking Comb	Cracking fs-t fs-b ksi	Cracking dc in	Cracking Srqt Srqb in	Cracking Sprt Sprb in	Fatigue stress range fs-t fs-b ksi	Fatigue threshold FTH-t FTH-b ksi
0.75	13.6	30.0	27	5.6	2.3	42.0	17.2	0.00	0.00
			0	0.0	2.4	5.0	11.4	0.00	0.00

Cracking and fatigue Check : Notes

* Cracking / fatigue checking failed.
Required bar spacing, Sreq, should not be less than 5 in per Art. 5.7.3.4.

Note:

Please note the analysis is based on the assumption that the cap is infinitely rigid which results into only axial forces in the piles.

Pile Reactions, Service **Load Effect @ Pile Cap Bot.**

Pile	X in	Z in	Batter X degree	Batter Z degree	comb	Ovs	P kips	Mxx kft	Mzz kft	Pile Reac. kips
1	34.5	0.0	0	0	44	1.000	-171.50	0.00	7.86	86.54
					58	1.000	-158.04	0.00	-15.15	77.50
2	154.5	0.0	0	0	29	1.000	-171.50	0.00	-7.85	86.54
					57	1.000	-158.04	0.00	15.16	77.50

Pile Reactions, Factored **Load Effect @ Pile Cap Bot.**

Pile	X in	Z in	Batter X degree	Batter Z degree	comb	Ovs	P kips	Mxx kft	Mzz kft	Pile Reac. kips
1	34.50	0.0	0	0	3	---	-228.69	0.00	0.01	114.35
					17	---	-128.60	0.00	-39.50	60.35
2	154.50	0.0	0	0	3	---	-228.69	0.00	0.01	114.34
					22	---	-128.60	0.00	39.50	60.35

Nominal Bearing Resistance = 114.34 kip / 0.65 = 175.91 kip = 88 ton

Pile Reactions: Notes

Pile reaction calculation is based on the assumption that pile cap is infinitely rigid. Load effects on pile are calculated at CG of pile group.
Both the max. and min. pile reaction are reported for each individual pile.
Positive pile reaction represents pile subject to compression load; negative pile reaction represents pile subject to uplift.
Coordinate system of pile layout see Geometry Tab>Footing Pile>Edit Pile.

Pile Lateral Resistance Check X

Controlling load combination number	17
Lateral loads on all piles(kips)	10.651
Batter angle(deg)	0.000
Available lateral resistance due to batter(kips)	0.000
Direct shear resistance of all piles(kips)	780.000
Total pile lateral resistance(kips)	780.000
Is the lateral resistance due to batter greater than lateral load on all piles?	NO
Is the total pile lateral resistance greater than the lateral load on all piles?	YES

Pile Lateral Resistance Check Z

Controlling load combination number	17
Lateral loads on all piles(kips)	0.000
Batter angle(deg)	0.000
Available lateral resistance due to batter(kips)	0.000
Direct shear resistance of all piles(kips)	780.000
Total pile lateral resistance(kips)	780.000
Is the lateral resistance due to batter greater than lateral load on all piles?	NO
Is the total pile lateral resistance greater than the lateral load on all piles?	YES

End Bent Design

AASHTO LRFD Bridge Design Specifications (LRFD) and FDOT Structures Design Guidelines (SDG) are referenced.

Bridge Geometry

Span Length: $Span := 53 \cdot ft + 0 \cdot in$

Bridge Width: $b_{bridge} := 13 \cdot ft + 7 \cdot in$

Bridge Skew: $\phi := 0 \cdot deg$

Approach Slab Length: $L_{as} := (10 \cdot ft) \div \cos(\phi)$ $\quad L_{as} = 10.00 \cdot ft$

Number of Beams: $NoBeam := 3$

Average Beam Tributary Widths: $j := 1 .. 3$

$BeamTA_j :=$

$4 \cdot ft + 6 \cdot in$
$4 \cdot ft + 7 \cdot in$
$4 \cdot ft + 6 \cdot in$

OK

Average Exterior Beam Overhang: $k := 1 .. 2$

$OH_k :=$

$2 \cdot ft + 2.5 \cdot in$
$2 \cdot ft + 2.5 \cdot in$

2ft + 3in. Not Used. Say OK.

FSB Height: $h_{FSB} := 15 \cdot in$

Superstructure Details

Topping Thickness: $Deck := 6 \cdot in$

FSB Width: $b_{FSB} := 4 \cdot ft + 6 \cdot in$

Number of Traffic Railings: $No_{t.rail} := 0$

Number of Pedestrian Railings: $No_{p.rail} := 2$

Approach Slab Thickness: (approx. slab + overlay) $t_{as} := 13.0 \cdot in$ OK

Miscellaneous Dead Loads (SDG Table 2.2-1)

Concrete Unit Weight: $\gamma_c := 150 \cdot pcf$

Future Wearing Surface Weight: $w_{fws} := 0 \cdot psf$

Soil Unit Weight, Compacted: $\gamma_{soil} := 115 \cdot pcf$

Beam Weight: $w_{beam} := 0 \cdot plf$

NOTE: Superstructure dead load evaluated below in combination with slab

Traffic Railing Weight: $w_{t.rail} := 0 \cdot plf$

Pedestrian Railing Weight: $w_{p.rail} := 80 \cdot plf$

Note(s):
Weight increased to include concrete pedestal

Dead Loads (DC)

BEARING LOADS

BEAM

Beam Weight: (per bearing)

$$Beam := w_{beam} \cdot \left(\frac{Span}{2}\right)$$

$$Beam = 0.0 \cdot kip$$

SLAB

Slab Weight: (per bearing)

$$Slab := \gamma_c \cdot \left(\sum_{k=1}^{NoBeam} BeamTA_k\right) \cdot \left(h_{FSB} + Deck\right) \cdot \left(\frac{Span}{2}\right) \div NoBeam$$

$$Slab = 31.5 \cdot kip$$

BEAM BUILD-UP

Build-Up Dimensions: (estimate, SPI 450-450, case 1-3 eq. assumed)

Dimension B: $B := 1.0 \cdot in$

Dimension C: $C := 0 \cdot in$

Dimension D: $D := 1.0 \cdot in$

Build-Up Weight: (per bearing)

$$BuildUp := \gamma_c \cdot b_{FSB} \cdot \left(C + \frac{B + D - 2C}{6}\right) \cdot \left(\frac{Span}{2}\right)$$

$$BuildUp = 0.5 \cdot kip$$

Dead Loads (DC) Cont'd...

BEARING LOADS CONT'D...

BARRIER

Barrier Weight: (per bearing)

$$Rail := \frac{No_{t.rail} \cdot w_{t.rail} + No_{p.rail} \cdot w_{p.rail}}{NoBeam} \cdot \left(\frac{Span}{2}\right)$$

$$Rail = 1.4 \cdot kip$$

TOTAL LOADS

Total DC Bearing Load: (per bearing)

$$DC := Beam + Slab + BuildUp + Rail$$

$$-DC = -33.4 \cdot kip$$

Note(s):
n/a

Dead Loads (DC) Cont'd...

CAP GEOMETRY

Bent Cap Length:	For load calculation only.	$L_{cap} := 13 \cdot ft + 9 \cdot in$
Bent Cap Width:		$b_{cap} := 3 \cdot ft + 6 \cdot in$
Bent Cap Height:		$h_{cap} := 2 \cdot ft + 6 \cdot in$
Backwall Height:		$h_{bw} := 0 \cdot ft + 6 \cdot in$
Backwall Width:		$b_{bw} := 1 \cdot ft + 0 \cdot in$
Wingwall Length: (from front face of cap)		$L_{ww} := 7 \cdot ft + 0 \cdot in$
Wingwall Cap Width:		$b_{ww} := 0 \cdot ft + 0 \cdot in$
Wingwall Height:	*(estimate)*	$h_{ww} := 4 \cdot ft + 5 \cdot in$
Wingwall Thickness:		$t_{ww} := 12 \cdot in$
Percentage of Wingwall Length (from bfbw) applied to Primary Cap: (to allow for consideration of pile supported wingwalls)		$\%_{ww} := 100 \cdot \%$

CAP LOADS

Consider a point load on the cap due to the cheekwall and/or wingwall self-weights.

Wingwall Load: *(apply at x1/L = n/a)*	$DC_{ww} := \%_{ww} \cdot \gamma_c \cdot L_{ww} \cdot h_{ww} \cdot t_{ww}$	$-DC_{ww} = -4.64 \cdot kip$
Wingwall Moment Arm:	$a_{ww} := \frac{1}{2} \cdot (\%_{ww} \cdot L_{ww}) - \frac{1}{2} \cdot b_{cap}$	$a_{ww} = 1.75 \cdot ft$

Consider a uniform line load along the centerline of the backwall due to the backwall and approach slab reaction. Reference SDG 3.1I for distribution of load notes. Transform to a line load at the centerline of the cap and a moment about the cap.

Moment Arm: (CL Backwall to CL Cap)	$a_{bw} := 0.5 \cdot b_{cap} - 0.5 \cdot b_{bw}$	$a_{bw} = 1.25 \cdot ft$
Backwall Weight:	$w_{bw} := \gamma_c \cdot (b_{bw} \cdot h_{bw})$	$w_{bw} = 0.08 \cdot \frac{kip}{ft}$
Approach Slab Weight:	$w_{as} := \gamma_c \cdot (t_{as} \cdot b_{bridge} \cdot L_{as}) \div L_{cap}$	$w_{as} = 1.61 \cdot \frac{kip}{ft}$
Barrier Weight:	$w_{Rail} := (No_{t.rail} \cdot w_{t.rail} + No_{p.rail} \cdot w_{p.rail}) \cdot L_{as} \div L_{cap}$	$w_{Rail} = 0.12 \cdot \frac{kip}{ft}$

Dead Loads (DC) Cont'd...

CAP LOADS CONT'D...

Total Uniform Dead Load: $DC_{bw} := w_{bw} + \frac{1}{2} \cdot (w_{as} + w_{Rail})$ $\quad -DC_{bw} = -0.94 \cdot \frac{kip}{ft}$

Total Dead Load Moment: $MDC_{bw} := (DC_{bw} \cdot L_{cap}) \cdot a_{bw}$ $\quad MDC_{bw} = 16.1 \cdot kip \cdot ft$

Note:
n/a

Dead Loads (DW)

BEARING LOADS

WEARING SURFACE

Wearing Surface:
(per bearing)

$FWS := w_{fws} \cdot BeamTA \cdot \left(\frac{Span}{2}\right)$

$FWS^T = (0.0 \quad 0.0 \quad 0.0) \cdot kip$

UTILITIES

Utility Weight:
(per linear foot of bridge)

$w_{util} := 0.0 \cdot plf$

Utility Weight:
(per bearing)

$Util := w_{util} \cdot \left(\frac{Span}{2}\right) \div NoBeam$ $\quad Util = 0.0 \cdot kip$

TOTAL LOADS

Total DW Bearing Load:
(per bearing)

$DW := FWS + Util$

$$-DW = \begin{pmatrix} 0.00 \\ 0.00 \\ 0.00 \end{pmatrix} \cdot kip$$

Note(s):
n/a

Uniform Temperature Loads (TU)

Uniform temperature loads include the forces from the superstructure to the substructure due to temperature, creep, and shrinkage. Bearing pad properties are per FDOT Index 450-450 Standard Plans Instructions. Creep is ignored assuming that the composite deck will offer restraint.

BEARING LOADS

CST Length:		$CST := 26.5 \cdot ft$
Coefficient of Thermal Expansion: (LRFD 5.4.2.2)		$\alpha := 6 \cdot 10^{-6} \cdot F^{-1}$
Temperature Rise/Fall: (SDG 2.7.1)		$\Delta T := 35 \cdot F$
Bearing Pad Area:		$Pad_{area} := (8 \cdot in) \cdot (42 \cdot in)$
Bearing Pad Elastomeric Thickness:		$Pad_{thick} := 1 \cdot in$
Shear Modulus:		$G := 110 \cdot psi$
Shrinkage Strain:	*(Assumed)*	$\varepsilon_{sh} := 0.0001$
Temperature Strain:	$\varepsilon_{temp} := \alpha \cdot \Delta T$	$\varepsilon_{temp} = 0.00021$
Total Strain:	$\varepsilon_{total} := \varepsilon_{sh} + \varepsilon_{temp}$	$\varepsilon_{total} = 0.00031$
Displacement Due to Strain:	$\Delta_{pier} := CST \cdot (\varepsilon_{sh} + \varepsilon_{temp})$	$\Delta_{pier} = 0.099 \cdot in$
Force Transferred by Pad: (per bearing)	$TU_{CST} := \Delta_{pier} \cdot \frac{G \cdot Pad_{area}}{Pad_{thick}}$	$TU_{CST} = 3.64 \cdot kip$
Longitudinal Force: (per bearing)	$TU_{long} := TU_{CST} \cdot \cos(\phi)$	$TU_{long} = 3.64 \cdot kip$
Transverse Force: (per bearing)	$TU_{trans} := TU_{CST} \cdot \sin(\phi)$	$TU_{trans} = 0.00 \cdot kip$

Note(s):
Apply in both the (+/-) directions.

Live Loads (LL)

LIve load is generated for the H10 truck loading in Leap Substructure. Pedestrian loading is calculated below per AASHTO LRFD for Ped. Bridges.

Truck Loads

Generated in RCPier.

Pedestrian Loads

Pedestrian loading: $Ped := 90psf$

Trail Width on Bridge: $W_{trl} := 12ft + 0in$

Ped load per bearing: $PL := \frac{Span}{2} \cdot Ped \cdot W_{trl} \div 3$ $\quad -PL = -9.54 \cdot kip$

Braking Loads (BR)

n/a

Centrifugal Loads (CE)

n/a

Live Loads (LLf)

n/a

Dynamic Load Allowance (IM)

n/a

Wind Load on Structure (WS)

Wind load on structure forces are generated in Leap Substructure per FDOT SDG 2.4 and LRFD 3.8. The wind angle varies from 0 to 60 deg. in 15 deg. intervals. The height of the barrier input into Leap Substructure is modified to account for the effect of bridge cross-slope. The Leap Substructure bearing pad depth is modified to match the approx. build-up height for superstructure depth estimate purposes.

Design Wind Speed: (SDG Table 2.4.1-1) — *(Lee County - assume Exposure C, G = 0.85)* — $V_{wind} := 170 \cdot mph$

Controlling Railing Height: — *Note(s):* *Use effective railing height to account for reduced area subject to wind pressure of pedestrian rail.* — $h_{rail} := 32 \cdot in$

Avg. Height to Top of Superstructure: — *(height <33 ft.)* — $h_{avg} := 33 \cdot ft$ OK

Superstructure Height: — $Z := \max(h_{avg}, 33 \cdot ft)$ — $Z = 33.0 \cdot ft$

Wind on Live Load (WL)

n/a

3.2. End Bent Wingwall Length

Vertical Alignment:

VPC Station:	$PC1_{Sta} := 44620.00\,ft$	VPC Elevation:	$PC1_{EL} := 21.6831\,ft$
VPI Station:	$PI2_{Sta} := 44650.00\,ft$	VPI Elevation:	$PI2_{EL} := 22.6100\,ft$
VPT Station:	$PT3_{Sta} := 44680.00\,ft$	VPT Elevation:	$PT3_{EL} := 22.7000\,ft$
VPI Station:	$PI4_{Sta} := 44790.82\,ft$	VPI Elevation:	$PI4_{EL} := 23.0325\,ft$
VPC Station:	$PC5_{Sta} := 44950.00\,ft$	VPC Elevation:	$PC5_{EL} := 22.7141\,ft$
VPI Station:	$PI6_{Sta} := 44980.00\,ft$	VPI Elevation:	$PI6_{EL} := 22.6541\,ft$
VPT Station:	$PT7_{Sta} := 45010.00\,ft$	VPT Elevation:	$PT7_{EL} := 22.2665\,ft$

Grade 1: $g_1 := (PI2_{EL} - PC1_{EL}) \div (PI2_{Sta} - PC1_{Sta})$ $\quad g_1 = 3.0897 \cdot \%$

Grade 2: $g_2 := (PT3_{EL} - PI2_{EL}) \div (PT3_{Sta} - PI2_{Sta})$ $\quad g_2 = 0.3 \cdot \%$

Grade 3: $g_3 := (PI4_{EL} - PT3_{EL}) \div (PI4_{Sta} - PT3_{Sta})$ $\quad g_3 = 0.3 \cdot \%$

Grade 4: $g_4 := (PC5_{EL} - PI4_{EL}) \div (PC5_{Sta} - PI4_{Sta})$ $\quad g_4 = -0.2 \cdot \%$

Grade 5: $g_5 := (PI6_{EL} - PC5_{EL}) \div (PI6_{Sta} - PC5_{Sta})$ $\quad g_5 = -0.2 \cdot \%$

Grade 6: $g_6 := (PT7_{EL} - PI6_{EL}) \div (PT7_{Sta} - PI6_{Sta})$ $\quad g_6 = -1.292 \cdot \%$

Profile Length Considered: $LP := PT7_{Sta} - PC1_{Sta}$ $\quad LP = 390\,ft$

Vertical Curve Rate of Grade Change:

$$R1 := \frac{(g_2 - g_1)}{(PT3_{Sta} - PC1_{Sta})}$$

$$R2 := \frac{(g_6 - g_5)}{(PT7_{Sta} - PC5_{Sta})}$$

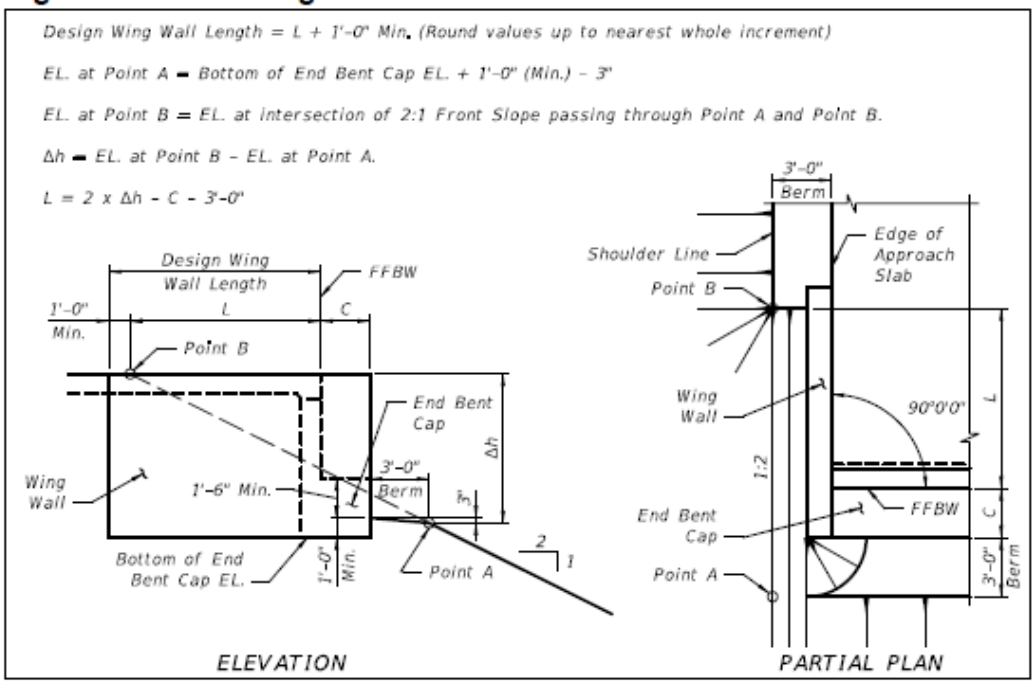
Vertical Profile Eqn.:

$$PGL(x) := \begin{cases} PC1_{EL} + (x - PC1_{Sta}) \cdot g_1 + \frac{R1}{2} \cdot (x - PC1_{Sta})^2 & \text{if } x < PT3_{Sta} \\ PT3_{EL} + g_3 \cdot (x - PT3_{Sta}) & \text{if } PT3_{Sta} \le x < PI4_{Sta} \\ PI4_{EL} + g_4 \cdot (x - PI4_{Sta}) & \text{if } PI4_{Sta} \le x < PC5_{Sta} \\ PC5_{EL} + (x - PC5_{Sta}) \cdot g_5 + \frac{R2}{2} \cdot (x - PC5_{Sta})^2 & \text{otherwise} \end{cases}$$

Bridge Geometry - Abutment 1

Begin Bridge Station:		$Sta_{bb} := 44684.82ft$
PGL Elevation at Beg. Bridge:	$EL_{bb} := PGL(Sta_{bb})$	$EL_{bb} = 22.714\ ft$
Deck Thickness:		$t_{deck} := 6in$
Beam Height:		$h_{beam} := 15in$
Buildup on Beams (Assumed):		$BU := 1.0in$
Beam Pedestal Height:		$h_p := 0in$
Bearing Height:		$h_{bear} := 1in$
Superstructure Depth:	$D_{SS} := t_{deck} + BU + h_{beam} + h_{bear} + h_p$	$D_{SS} = 1.917\ ft$
End Bent 1 Cap Elevation at PGL:	$El_{cap1} := EL_{bb} - D_{SS}$	$El_{cap1} = 20.798\ ft$
Length of front Berm:		$berm := 3ft$
Width of Bent Cap:		$b_{cap} := 3.5ft$
Width of Back Wall (Approach Slab):		$b_{bw} := 1ft$
Height of Bent Cap:		$h_{cap} := 2.5ft$
Offset to Left Coping:		$off_{Lt} := -9.5in$
Offset to Right Coping:		$off_{Rt} := 12ft + 9.5in$
Cross slope:		$xs := 2\%$
Skew:		$\phi := 0deg$
End Bent 1 Cap Elevation at Left Coping:	$El_{cap1Lt} := PGL(Sta_{bb} + off_{Lt} \cdot \tan(\phi)) \ldots + off_{Lt} \cdot xs - D_{SS}$	$El_{cap1Lt} = 20.782\ ft$
End Bent 1 Cap Elevation at Right Coping:	$El_{cap1Rt} := PGL(Sta_{bb} - off_{Rt} \cdot \tan(\phi)) \ldots + off_{Rt} \cdot xs - D_{SS}$	$El_{cap1Rt} = 21.054\ ft$

Figure 12.6-3 Wingwall Calculation Detail



Design Wingwall Length - Abutment 1 Left:

Elevation of Point 'A':	$EL_A := El_{cap1Lt} - h_{cap} + 1ft - 3in$	$EL_A = 19.032\ ft$
Dimension 'C':	$C := b_{cap} - b_{bw}$	$C = 2.5\ ft$
Initial Guess:		$x := Sta_{bb} - 6ft$
Given	$PGL\left(x + off_{Lt} \cdot \tan(\phi)\right) + off_{Lt} \cdot xs = EL_A + \frac{1}{2} \cdot \left(Sta_{bb} + C + berm - x\right)$	
Location of Point 'B':	$Sta_B := Find(x)$	$Sta_B = 44682.998\ ft$
Elevation at Point 'B':	$EL_B := EL_A + \frac{1}{2} \cdot \left(Sta_{bb} + C + berm - Sta_B\right)$	$EL_B = 22.693\ ft$
Change in Elevation:	$\Delta h := EL_B - EL_A$	$\Delta h = 3.661\ ft$
Dimension 'L':	$L := 2 \cdot \Delta h - C - berm$	$L = 1.822\ ft$
Design Wingwall Length:	$L_{des} := Ceil(L \div in + 12, 1) \cdot in$	$L_{des} = 2.833\ ft$

Design Wingwall Length - Abutment 1 Right:

Elevation of Point 'A': $EL_A := El_{cap1Rt} - h_{cap} + 1ft - 3in$

$EL_A = 19.304\ ft$

Initial Guess: $x := Sta_{bb} - 6ft$

Given $PGL\left(x - off_{Rt} \cdot \tan(\phi)\right) + off_{Rt} \cdot xs = EL_A + \frac{1}{2} \cdot \left(Sta_{bb} + C + berm - x\right)$

Location of Point 'B': $Sta_B := Find(x)$ $Sta_B = 44682.998\ ft$

Elevation at Point 'B': $EL_B := EL_A + \frac{1}{2} \cdot \left(Sta_{bb} + C + berm - Sta_B\right)$ $EL_B = 22.965\ ft$

Change in Elevation: $\Delta h := EL_B - EL_A$ $\Delta h = 3.661\ ft$

Dimension 'L': $L := 2 \cdot \Delta h - C - berm$ $L = 1.822\ ft$

Design Wingwall Length: $L_{des} := Ceil(L \div in + 12, 1) \cdot in$ $L_{des} = 2.833\ ft$

Bridge Geometry - Abutment 6

End Bridge Station: $Sta_{eb} := 44949.82\,ft$

PGL Elevation at End. Bridge: $EL_{eb} := PGL(Sta_{eb})$ $EL_{eb} = 22.714\,ft$

End Bent 2 Cap Elevation at PGL: $El_{cap2} := EL_{eb} - D_{SS}$ $El_{cap2} = 20.798\,ft$

End Bent 2 Cap Elevation at Left Coping: $El_{.cap2Lt} := PGL(Sta_{.eb} + off_{.Lt} \cdot \tan(\phi)) + off_{.Lt} \cdot xs - D_{.SS}$

$El_{cap2Lt} = 20.782\,ft$

End Bent 1 Cap Elevation at Right Coping: $El_{cap2Rt} := PGL(Sta_{eb} - off_{Rt} \cdot \tan(\phi)) + off_{Rt} \cdot xs - D_{SS}$

$El_{cap2Rt} = 21.054\,ft$

Design Wingwall Length - Abutment 6 Left:

Elevation of Point 'A': $EL_A := El_{cap2Lt} - h_{cap} + 1ft - 3in$ $EL_A = 19.032\,ft$

Initial Guess: $x := Sta_{eb} + 6ft$

Given $PGL(x + off_{Lt} \cdot \tan(\phi)) + off_{Lt} \cdot xs = EL_A + \frac{1}{2} \cdot [x - (Sta_{eb} - C - berm)]$

Location of Point 'B': $Sta_B := Find(x)$ $Sta_B = 44951.646\,ft$

Elevation at Point 'B': $EL_B := EL_A + \frac{1}{2} \cdot [Sta_B - (Sta_{eb} - C - berm)]$ $EL_B = 22.695\,ft$

Change in Elevation: $\Delta h := EL_B - EL_A$ $\Delta h = 3.663\,ft$

Dimension 'L': $L := 2 \cdot \Delta h - C - berm$ $L = 1.826\,ft$

Design Wingwall Length: $L_{des} := Ceil(L \div in + 12, 1) \cdot in$ $L_{des} = 2.833\,ft$

Design Wingwall Length - Abutment 6 Right:

Elevation of Point 'A': $EL_A := El_{cap2Rt} - h_{cap} + 1ft - 3in$ $EL_A = 19.304\ ft$

Initial Guess: $x := Sta_{eb} + 6ft$

Given $PGL\left(x - off_{Rt} \cdot \tan(\phi)\right) + off_{Rt} \cdot xs = EL_A + \frac{1}{2} \cdot \left[x - \left(Sta_{eb} - C - berm\right)\right]$

Location of Point 'B': $Sta_B := Find(x)$ $Sta_B = 44951.646\ ft$

Elevation at Point 'B': $EL_B := EL_A + \frac{1}{2} \cdot \left[Sta_B - \left(Sta_{eb} - C - berm\right)\right]$ $EL_B = 22.966\ ft$

Change in Elevation: $\Delta h := EL_B - EL_A$ $\Delta h = 3.663\ ft$

Dimension 'L': $L := 2 \cdot \Delta h - C - berm$ $L = 1.826\ ft$

Design Wingwall Length: $L_{des} := Ceil(L \div in + 12, 1) \cdot in$ $L_{des} = 2.833\ ft$

3.3. Intermediate Bent Design

PROJECT DATA

PROJECT DATA

Project:	Able Canal Pathway
User Job No.:	
Designer:	BAH
Date:	11/2021
Checker:	
Checked date:	
State:	FL
State Job No.:	
Structure type:	Pier.
Pier View:	Downstation.
State Specification:	None
Code:	AASHTO LRFD 8th Edition
Comments:	Intermediate bent design. FSB Superstructure

PIER GEOMETRY

Pier Info:

Pier View:	Downstation.
Pier Type:	Pile Bent

Cap Shape

Cap Shape: Straight
Top Elevations: start = 21.00 ft end = 21.00 ft
Depth(Z) = 44.00 in Skew angle = 0.00 Reduction of I = 1.000
Length(X) = 13.67 ft Height(Y) = 30.00 in

Column Shape : Rectangular

Number of columns: 3

Column number 1

Location from the left edge of the cap(X): 1.75 ft
Elevations: bottom = -1.50 ft top = 19.75 ft Reduction of I = 1.000
Column Bottom is Fixed

Column Section Dimensions

Width(X) = 18.00 in Depth(Z) = 18.00 in

Column number 2

Location from the left edge of the cap(X): 6.83 ft
Elevations: bottom = -1.50 ft top = 19.75 ft Reduction of I = 1.000
Column Bottom is Fixed

Column Section Dimensions

Width(X) = 18.00 in Depth(Z) = 18.00 in

Column number 3

Location from the left edge of the cap(X): 11.92 ft
Elevations: bottom = -1.50 ft top = 19.75 ft Reduction of I = 1.000
Column Bottom is Fixed

Column Section Dimensions

Width(X) = 18.00 in Depth(Z) = 18.00 in

SUPERSTRUCTURE INFO

Superstructure info:

Total number of spans:	2	
Span number rear to current pier:	1	
Number of traffic lanes:	1	
Barrier height :	32.00 in	75% railing height to account for wind reduction due to open picket railing.
Depth of slab :	12.00 in	Includes height of railing support.
Curb to curb distance:	12.000 ft	

Beam info:

Height in	Section area in^2	Inertia (Ixx) in^4	Inertia (Iyy) in^4	Beam CG in
15.00	671.76	12950.00	116990.00	7.09

Span #	Span length ft	Bridge Width ft
1	53.000	13.667
2	53.000	13.667
		13.667

BEARING POINTS

Number of bearing lines: 2

First bearing line **Eccentricity = 1.13 ft**

Point	Distance ft
1	2.25
2	6.83
3	11.42

Second bearing line **Eccentricity = -1.13 ft**

Point	Distance ft
1	2.25
2	6.83
3	11.42

MATERIAL PROPERTIES

MATERIAL PROPERTIES

	Cap	Column	Footing
Concrete Type	normal	normal	normal
Concrete Strength (psi)	5500.00	6000.00	3400.00
Concrete Density (lb/ft3)	150.00	150.00	150.00
Concrete Modulus Ec (ksi)	4428.30	4557.30	3778.38
Steel Strength Fy (ksi)	60.00	60.00	60.00

DESIGN PARAMETERS

Resistance factors for reinf. concrete

Flexure and tension	0.90
Shear and torsion (normal)	0.90
Shear and torsion (lightweight)	0.90
Axial compression (ties)	0.75
Axial compression (spiral)	0.75
Compression in STM	0.70

Multi presence factors for live load

1 Lane	1.00
2 Lanes	1.00
3 Lanes	0.85
4 Lanes	0.65
5 Lanes	0.65
6 Lanes	0.65

Dynamic load allowance IM

	Truck	Lane	Fatigue
Cap	0.00	0.00	0.00
Column	0.00	0.00	0.00
Footing	0.00	0.00	0.00

	Exposure factors	Clear cover in	Clear side cover in
Cap	1.00	3.00	3.00
Column	1.00	2.00	
Footing	1.00	3.00	3.00

Degree of fixity in foundations for Moment Magnify Method: Ga = 5.00

SEISMIC DESIGN PARAMETERS

Strength Reduction factors for reinf. Concrete Seismic Design

Tension controlled	0.90
Shear and torsion (normal)	0.90
Shear and torsion (lightweight)	0.70
Compression Controlled (ties)	0.75
Compression Controlled (spiral)	0.75

Seismic Overstrength

Flexure and tension	1.30
Axial compression (ties)	1.30
Axial compression (spiral)	1.30

Response Modification Factor	1.00

Use core area for plastic hinging calculations.

Design Factors

Cap Design Factor	1.20
Footing Design Factor	1.20

Plastic Hinge Moment

Use actual computed Plastic Hinging Moment for each column in all combinations.

LOADS

Pier Info:

Pier View:	Downstation.

Load Cases: 16

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:	H-10

Tranverse Positioning

Number of loaded lanes	1 Lane Loaded
Live Load Positions	Constant Spacing
Minimum Distance from Curb	2.000000
Center to Center Spacing	10.000000

Generate Braking/Longitudinal Force	Not Selected
Generate Centrifugal Force	Not Selected

Number of single regular truck positions	4
Number of all possible combinations of regular truck positions	8

Loadcase ID: DC1 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	-33.40
1	2	Y	-33.40
1	3	Y	-33.40
2	1	Y	-33.40
2	2	Y	-33.40
2	3	Y	-33.40

Loadcase ID: LL1 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips	Lane Load
1	1	Y	-6.98	
1	2	Y	-5.53	
1	3	Y	-3.49	
2	1	Y	-1.51	
2	2	Y	-1.20	
2	3	Y	-0.76	
1	1	Y	0.00	L
1	2	Y	0.00	L
1	3	Y	0.00	L
2	1	Y	0.00	L
2	2	Y	0.00	L
2	3	Y	0.00	L

Auto generation details:

Generated Live Load

TRUCK IDENTIFICATION : H-10
Positions of the center of each load measured from the left of the bridge :

TRUCK TYPE : Regular
Truck No. 1 - center line at: 5.83 - 2 axle on bearing line No. 1
Multi lane reduction factor used : 1.0000

Maximum effects in members :

Member 1	node 1	Fx = 7.3488
Member 1	node 2	Fx = -7.3488
Member 6	node 2	Fy = 7.3488
Member 6	node 9	Mz = 3.6744
Member 6	node 9	Fy = -7.3488
Member 7	node 9	Fy = -1.1460
Member 7	node 9	Mz = -3.6744
Member 7	node 10	Fy = 1.1460
Member 7	node 10	Mz = 3.3879
Member 8	node 10	Fy = -1.1460
Member 8	node 10	Mz = -3.3879
Member 8	node 11	Fy = 1.1460
Member 9	node 11	Fy = -1.1460
Member 9	node 4	Fy = 1.1460
Member 10	node 12	Mz = -1.0315
Member 11	node 12	Mz = 1.0315

Loadcase ID: LL2 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips	Lane Load
1	1	Y	-3.49	
1	2	Y	-5.53	
1	3	Y	-6.98	
2	1	Y	-0.76	
2	2	Y	-1.20	
2	3	Y	-1.51	
1	1	Y	0.00	L
1	2	Y	0.00	L
1	3	Y	0.00	L
2	1	Y	0.00	L
2	2	Y	0.00	L
2	3	Y	0.00	L

Auto generation details:

Generated Live Load

TRUCK IDENTIFICATION : H-10
Positions of the center of each load measured from the left of the bridge :

TRUCK TYPE : Regular
Truck No. 1 - center line at: 7.83 - 2 axle on bearing line No. 1
Multi lane reduction factor used : 1.0000

Maximum effects in members :

Member 2	node 3	Fx = 8.6003
Member 2	node 4	Fx = -8.6003
Member 3	node 5	Fx = 7.3495
Member 3	node 6	Fx = -7.3495
Member 8	node 11	Mz = -1.0327
Member 9	node 11	Mz = 1.0327
Member 9	node 4	Mz = -1.5789
Member 10	node 4	Fy = 1.1475
Member 10	node 4	Mz = 1.5789
Member 10	node 12	Fy = -1.1475
Member 11	node 12	Fy = 1.1475
Member 11	node 13	Mz = 3.3935
Member 11	node 13	Fy = -1.1475
Member 12	node 13	Fy = 1.1475
Member 12	node 13	Mz = -3.3935
Member 12	node 14	Mz = 3.6796
Member 12	node 14	Fy = -1.1475
Member 13	node 14	Fy = -7.3495
Member 13	node 14	Mz = -3.6796
Member 13	node 6	Fy = 7.3495

Loadcase ID: LL3 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips	Lane Load
1	1	Y	-9.54	
1	2	Y	-9.54	
1	3	Y	-9.54	
2	1	Y	-9.54	
2	2	Y	-9.54	
2	3	Y	-9.54	

Loadcase ID: LL4 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips	Lane Load
1	1	Y	-9.54	
1	2	Y	-9.54	
1	3	Y	-9.54	
2	1	Y	0.00	
2	2	Y	0.00	
2	3	Y	0.00	

Loadcase ID: LL5 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips	Lane Load
1	1	Y	0.00	
1	2	Y	0.00	
1	3	Y	0.00	
2	1	Y	-9.54	•
2	2	Y	-9.54	•
2	3	Y	-9.54	•

Note: Vertical Wind Loads per AASHTO LRFD 3.8.2 not included within Leap Bridge Concrete Substructure calculation in order to provide conservative pile reactions.

Loadcase ID: WS1 Name: STR III-Angle: 0

Multiplier = 1.000 •

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	0.92 kips	0.50		

Column loads

Col #	Type	Dir	Mag1	y1/L	Mag2	y2/L
1	UDL	X	0.151 klf	0.54		0.94
2	UDL	X	0.151 klf	0.54		0.94
3	UDL	X	0.151 klf	0.54		0.94

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	3.55
1	1	Y	2.86
1	1	Z	-0.00
1	2	X	3.55
1	2	Y	-0.00
1	2	Z	-0.00
1	3	X	3.55
1	3	Y	-2.86
1	3	Z	-0.00
2	1	X	3.55
2	1	Y	2.86
2	1	Z	-0.00
2	2	X	3.55
2	2	Y	-0.00
2	2	Z	-0.00
2	3	X	3.55
2	3	Y	-2.86
2	3	Z	-0.00

Note: Vertical Wind Loads per AASHTO LRFD 3.8.2 not included within Leap Bridge Concrete Substructure calculation in order to provide conservative pile reactions.

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 0.00 deg Elevation above which wind load acts = 10.00 ft Default wind pressure:		
For STR III		
	Wind pressure for superstructure: Pz = 81.750 psf	Wind pressure for substructure: Pz = 100.620 psf
Overturning not considered		
For SER IV		
	Wind pressure for superstructure: Pz = 45.990 psf	Wind pressure for substructure: Pz = 56.600 psf
Overturning not considered		
For STR V		
	Wind pressure for superstructure: Pz = 21.190 psf	Wind pressure for substructure: Pz = 26.080 psf
For SER I		
	Wind pressure for superstructure: Pz = 16.250 psf	Wind pressure for substructure: Pz = 20.000 psf

Loadcase ID: WS2 Name: STR III-Angle: 15

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	1.72 kips	0.50		
UDL	Z		-0.03 klf	0.00		1.00

Column loads

Col #	Type	Dir	Mag1	y1/L	Mag2	y2/L
1	UDL	X	0.179 klf	0.54		0.94
1	UDL	Z	-0.048 klf	0.54		0.94
2	UDL	X	0.179 klf	0.54		0.94
2	UDL	Z	-0.048 klf	0.54		0.94
3	UDL	X	0.179 klf	0.54		0.94
3	UDL	Z	-0.048 klf	0.54		0.94

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	3.12
1	1	Y	2.51
1	1	Z	-0.43
1	2	X	3.12
1	2	Y	-0.00
1	2	Z	-0.43
1	3	X	3.12
1	3	Y	-2.51
1	3	Z	-0.43
2	1	X	3.12
2	1	Y	2.51
2	1	Z	-0.43
2	2	X	3.12
2	2	Y	-0.00
2	2	Z	-0.43
2	3	X	3.12
2	3	Y	-2.51
2	3	Z	-0.43

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 15.00 deg Elevation above which wind load acts = 10.00 ft Default wind pressure:	
For STR III Wind pressure for superstructure: Pz = 81.750 psf Overturning not considered	Wind pressure for substructure: Pz = 100.620 psf
For SER IV Wind pressure for superstructure: Pz = 45.990 psf Overturning not considered	Wind pressure for substructure: Pz = 56.600 psf
For STR V Wind pressure for superstructure: Pz = 21.190 psf	Wind pressure for substructure: Pz = 26.080 psf
For SER I Wind pressure for superstructure: Pz = 16.250 psf	Wind pressure for substructure: Pz = 20.000 psf

Loadcase ID: WS3 Name: STR III-Angle: 30

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	2.18 kips	0.50		
UDL	Z		-0.09 klf	0.00		1.00

Column loads

Col #	Type	Dir	Mag1	y1/L	Mag2	y2/L
1	UDL	X	0.179 klf	0.54		0.94
1	UDL	Z	-0.103 klf	0.54		0.94
2	UDL	X	0.179 klf	0.54		0.94
2	UDL	Z	-0.103 klf	0.54		0.94
3	UDL	X	0.179 klf	0.54		0.94
3	UDL	Z	-0.103 klf	0.54		0.94

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	2.91
1	1	Y	2.34
1	1	Z	-0.85
1	2	X	2.91
1	2	Y	-0.00
1	2	Z	-0.85
1	3	X	2.91
1	3	Y	-2.34
1	3	Z	-0.85
2	1	X	2.91
2	1	Y	2.34
2	1	Z	-0.85
2	2	X	2.91
2	2	Y	-0.00
2	2	Z	-0.85
2	3	X	2.91
2	3	Y	-2.34
2	3	Z	-0.85

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 30.00 deg Elevation above which wind load acts = 10.00 ft

Default wind pressure:

For STR III

Wind pressure for superstructure: Pz = 81.750 psf

Wind pressure for substructure: Pz = 100.620 psf

Overturning not considered

For SER IV

Wind pressure for superstructure: Pz = 45.990 psf

Wind pressure for substructure: Pz = 56.600 psf

Overturning not considered

For STR V

Wind pressure for superstructure: Pz = 21.190 psf

Wind pressure for substructure: Pz = 26.080 psf

For SER I

Wind pressure for superstructure: Pz = 16.250 psf

Wind pressure for substructure: Pz = 20.000 psf

Loadcase ID: WS4 Name: STR III-Angle: 45

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	2.18 kips	0.50		
UDL	Z		-0.16 klf	0.00		1.00

Column loads

Col #	Type	Dir	Mag1	y1/L	Mag2	y2/L
1	UDL	X	0.151 klf	0.54		0.94
1	UDL	Z	-0.151 klf	0.54		0.94
2	UDL	X	0.151 klf	0.54		0.94
2	UDL	Z	-0.151 klf	0.54		0.94
3	UDL	X	0.151 klf	0.54		0.94
3	UDL	Z	-0.151 klf	0.54		0.94

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	2.34
1	1	Y	1.89
1	1	Z	-1.14
1	2	X	2.34
1	2	Y	-0.00
1	2	Z	-1.14
1	3	X	2.34
1	3	Y	-1.89
1	3	Z	-1.14
2	1	X	2.34
2	1	Y	1.89
2	1	Z	-1.14
2	2	X	2.34
2	2	Y	-0.00
2	2	Z	-1.14
2	3	X	2.34
2	3	Y	-1.89
2	3	Z	-1.14

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 45.00 deg Elevation above which wind load acts = 10.00 ft
Default wind pressure:

For STR III

Wind pressure for superstructure:
Pz = 81.750 psf

Wind pressure for substructure:
Pz = 100.620 psf

Overturning not considered

For SER IV

Wind pressure for superstructure:
Pz = 45.990 psf

Wind pressure for substructure:
Pz = 56.600 psf

Overturning not considered

For STR V

Wind pressure for superstructure:
Pz = 21.190 psf

Wind pressure for substructure:
Pz = 26.080 psf

For SER I

Wind pressure for superstructure:
Pz = 16.250 psf

Wind pressure for substructure:
Pz = 20.000 psf

Loadcase ID: WS5 Name: STR III-Angle: 60

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	1.72 kips	0.50		
UDL	Z		-0.22 klf	0.00		1.00

Column loads

Col #	Type	Dir	Mag1	y1/L	Mag2	y2/L
1	UDL	X	0.103 klf	0.54		0.94
1	UDL	Z	-0.179 klf	0.54		0.94
2	UDL	X	0.103 klf	0.54		0.94
2	UDL	Z	-0.179 klf	0.54		0.94
3	UDL	X	0.103 klf	0.54		0.94
3	UDL	Z	-0.179 klf	0.54		0.94

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	1.21
1	1	Y	0.97
1	1	Z	-1.35
1	2	X	1.21
1	2	Y	-0.00
1	2	Z	-1.35
1	3	X	1.21
1	3	Y	-0.97
1	3	Z	-1.35
2	1	X	1.21
2	1	Y	0.97
2	1	Z	-1.35
2	2	X	1.21
2	2	Y	-0.00
2	2	Z	-1.35
2	3	X	1.21
2	3	Y	-0.97
2	3	Z	-1.35

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 60.00 deg Elevation above which wind load acts = 10.00 ft Default wind pressure:	
For STR III Wind pressure for superstructure: Pz = 81.750 psf Overturning not considered	Wind pressure for substructure: Pz = 100.620 psf
For SER IV Wind pressure for superstructure: Pz = 45.990 psf Overturning not considered	Wind pressure for substructure: Pz = 56.600 psf
For STR V Wind pressure for superstructure: Pz = 21.190 psf	Wind pressure for substructure: Pz = 26.080 psf
For SER I Wind pressure for superstructure: Pz = 16.250 psf	Wind pressure for substructure: Pz = 20.000 psf

Loadcase ID: WS16 Name: SER I-Angle: 0

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	0.18 kips	0.50		

Column loads

Col #	Type	Dir	Mag1	y1/L	Mag2	y2/L
1	UDL	X	0.030 klf	0.54		0.94
2	UDL	X	0.030 klf	0.54		0.94
3	UDL	X	0.030 klf	0.54		0.94

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	0.71
1	1	Y	0.57
1	1	Z	-0.00
1	2	X	0.71
1	2	Y	-0.00
1	2	Z	-0.00
1	3	X	0.71
1	3	Y	-0.57
1	3	Z	-0.00
2	1	X	0.71
2	1	Y	0.57
2	1	Z	-0.00
2	2	X	0.71
2	2	Y	-0.00
2	2	Z	-0.00
2	3	X	0.71
2	3	Y	-0.57
2	3	Z	-0.00

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 0.00 deg Elevation above which wind load acts = 10.00 ft

Default wind pressure:

For STR III

Wind pressure for superstructure: Pz = 81.750 psf

Wind pressure for substructure: Pz = 100.620 psf

Overturning not considered

For SER IV

Wind pressure for superstructure: Pz = 45.990 psf

Wind pressure for substructure: Pz = 56.600 psf

Overturning not considered

For STR V

Wind pressure for superstructure: Pz = 21.190 psf

Wind pressure for substructure: Pz = 26.080 psf

For SER I

Wind pressure for superstructure: Pz = 16.250 psf

Wind pressure for substructure: Pz = 20.000 psf

Loadcase ID: WS17 Name: SER I-Angle: 15

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	0.34 kips	0.50		
UDL	Z		-0.01 klf	0.00		1.00

Column loads

Col #	Type	Dir	Mag1	y1/L	Mag2	y2/L
1	UDL	X	0.035 klf	0.54		0.94
1	UDL	Z	-0.010 klf	0.54		0.94
2	UDL	X	0.035 klf	0.54		0.94
2	UDL	Z	-0.010 klf	0.54		0.94
3	UDL	X	0.035 klf	0.54		0.94
3	UDL	Z	-0.010 klf	0.54		0.94

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	0.62
1	1	Y	0.50
1	1	Z	-0.08
1	2	X	0.62
1	2	Y	-0.00
1	2	Z	-0.08
1	3	X	0.62
1	3	Y	-0.50
1	3	Z	-0.08
2	1	X	0.62
2	1	Y	0.50
2	1	Z	-0.08
2	2	X	0.62
2	2	Y	-0.00
2	2	Z	-0.08
2	3	X	0.62
2	3	Y	-0.50
2	3	Z	-0.08

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 15.00 deg Elevation above which wind load acts = 10.00 ft
Default wind pressure:

For STR III

Wind pressure for superstructure:
Pz = 81.750 psf

Wind pressure for substructure:
Pz = 100.620 psf

Overturning not considered

For SER IV

Wind pressure for superstructure:
Pz = 45.990 psf

Wind pressure for substructure:
Pz = 56.600 psf

Overturning not considered

For STR V

Wind pressure for superstructure:
Pz = 21.190 psf

Wind pressure for substructure:
Pz = 26.080 psf

For SER I

Wind pressure for superstructure:
Pz = 16.250 psf

Wind pressure for substructure:
Pz = 20.000 psf

Loadcase ID: WS18 Name: SER I-Angle: 30

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	0.43 kips	0.50		
UDL	Z		-0.02 klf	0.00		1.00

Column loads

Col #	Type	Dir	Mag1	y1/L	Mag2	y2/L
1	UDL	X	0.035 klf	0.54		0.94
1	UDL	Z	-0.020 klf	0.54		0.94
2	UDL	X	0.035 klf	0.54		0.94
2	UDL	Z	-0.020 klf	0.54		0.94
3	UDL	X	0.035 klf	0.54		0.94
3	UDL	Z	-0.020 klf	0.54		0.94

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	0.58
1	1	Y	0.47
1	1	Z	-0.17
1	2	X	0.58
1	2	Y	-0.00
1	2	Z	-0.17
1	3	X	0.58
1	3	Y	-0.47
1	3	Z	-0.17
2	1	X	0.58
2	1	Y	0.47
2	1	Z	-0.17
2	2	X	0.58
2	2	Y	-0.00
2	2	Z	-0.17
2	3	X	0.58
2	3	Y	-0.47
2	3	Z	-0.17

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 30.00 deg Elevation above which wind load acts = 10.00 ft Default wind pressure:	
For STR III Wind pressure for superstructure: Pz = 81.750 psf Overturning not considered	Wind pressure for substructure: Pz = 100.620 psf
For SER IV Wind pressure for superstructure: Pz = 45.990 psf Overturning not considered	Wind pressure for substructure: Pz = 56.600 psf
For STR V Wind pressure for superstructure: Pz = 21.190 psf	Wind pressure for substructure: Pz = 26.080 psf
For SER I Wind pressure for superstructure: Pz = 16.250 psf	Wind pressure for substructure: Pz = 20.000 psf

Loadcase ID: WS19 Name: SER I-Angle: 45

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	0.43 kips	0.50		
UDL	Z		-0.03 klf	0.00		1.00

Column loads

Col #	Type	Dir	Mag1	y1/L	Mag2	y2/L
1	UDL	X	0.030 klf	0.54		0.94
1	UDL	Z	-0.030 klf	0.54		0.94
2	UDL	X	0.030 klf	0.54		0.94
2	UDL	Z	-0.030 klf	0.54		0.94
3	UDL	X	0.030 klf	0.54		0.94
3	UDL	Z	-0.030 klf	0.54		0.94

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	0.47
1	1	Y	0.37
1	1	Z	-0.23
1	2	X	0.47
1	2	Y	-0.00
1	2	Z	-0.23
1	3	X	0.47
1	3	Y	-0.37
1	3	Z	-0.23
2	1	X	0.47
2	1	Y	0.37
2	1	Z	-0.23
2	2	X	0.47
2	2	Y	-0.00
2	2	Z	-0.23
2	3	X	0.47
2	3	Y	-0.37
2	3	Z	-0.23

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 45.00 deg Elevation above which wind load acts = 10.00 ft

Default wind pressure:

For STR III

Wind pressure for superstructure: Pz = 81.750 psf

Wind pressure for substructure: Pz = 100.620 psf

Overturning not considered

For SER IV

Wind pressure for superstructure: Pz = 45.990 psf

Wind pressure for substructure: Pz = 56.600 psf

Overturning not considered

For STR V

Wind pressure for superstructure: Pz = 21.190 psf

Wind pressure for substructure: Pz = 26.080 psf

For SER I

Wind pressure for superstructure: Pz = 16.250 psf

Wind pressure for substructure: Pz = 20.000 psf

Loadcase ID: WS20 Name: SER I-Angle: 60

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	0.34 kips	0.50		
UDL	Z		-0.04 klf	0.00		1.00

Column loads

Col #	Type	Dir	Mag1	y1/L	Mag2	y2/L
1	UDL	X	0.020 klf	0.54		0.94
1	UDL	Z	-0.035 klf	0.54		0.94
2	UDL	X	0.020 klf	0.54		0.94
2	UDL	Z	-0.035 klf	0.54		0.94
3	UDL	X	0.020 klf	0.54		0.94
3	UDL	Z	-0.035 klf	0.54		0.94

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	0.24
1	1	Y	0.19
1	1	Z	-0.27
1	2	X	0.24
1	2	Y	-0.00
1	2	Z	-0.27
1	3	X	0.24
1	3	Y	-0.19
1	3	Z	-0.27
2	1	X	0.24
2	1	Y	0.19
2	1	Z	-0.27
2	2	X	0.24
2	2	Y	-0.00
2	2	Z	-0.27
2	3	X	0.24
2	3	Y	-0.19
2	3	Z	-0.27

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 60.00 deg Elevation above which wind load acts = 10.00 ft
Default wind pressure:

For STR III

Wind pressure for superstructure:
Pz = 81.750 psf

Wind pressure for substructure:
Pz = 100.620 psf

Overturning not considered

For SER IV

Wind pressure for superstructure:
Pz = 45.990 psf

Wind pressure for substructure:
Pz = 56.600 psf

Overturning not considered

For STR V

Wind pressure for superstructure:
Pz = 21.190 psf

Wind pressure for substructure:
Pz = 26.080 psf

For SER I

Wind pressure for superstructure:
Pz = 16.250 psf

Wind pressure for substructure:
Pz = 20.000 psf

Selected load groups

STRENGTH GROUP I
STRENGTH GROUP III
SERVICE GROUP I
SERVICE GROUP III

STRUCTURE MODEL

Frame Model Structure:

	Member	Node	Hinge	Check Pt	Dist ft	Memb length ft
Column No. 1						
	1	1	-		0.00	
		2	YZ		21.25	21.25
Column No. 2						
	2	3	-		0.00	
		4	YZ		21.25	21.25
Column No. 3						
	3	5	-		0.00	
		6	YZ		21.25	21.25
Cap						
	4	7	-		0.00	
		8	-	f	1.00	1.00
	5	8	-	f	1.00	
		2	-		1.75	0.75
	6	2	-		1.75	
		9	-		2.25	0.50
	7	9	-		2.25	
		10	-	f	2.50	0.25
	8	10	-	f	2.50	
		11	-	f	6.08	3.58
	9	11	-	f	6.08	
		4	-		6.83	0.75
	10	4	-		6.83	
		12	-	f	7.58	0.75
	11	12	-	f	7.58	
		13	-	f	11.17	3.58
	12	13	-	f	11.17	
		14	-		11.42	0.25
	13	14	-		11.42	
		6	-		11.92	0.50
	14	6	-		11.92	
		15	-	f	12.67	0.75
	15	15	-	f	12.67	
		16	-		13.67	1.00

Node coordinates

Number	X ft	Y ft	Node type
1	1.75	-1.50	fixed at ground
2	1.75	19.75	column-cap
3	6.83	-1.50	fixed at ground
4	6.83	19.75	column-cap bearing
5	11.92	-1.50	fixed at ground
6	11.92	19.75	column-cap
7	0.00	19.75	
8	1.00	19.75	face of support
9	2.25	19.75	bearing
10	2.50	19.75	face of support
11	6.08	19.75	face of support
12	7.58	19.75	face of support
13	11.17	19.75	face of support
14	11.42	19.75	bearing
15	12.67	19.75	face of support
16	13.67	19.75	

CAP DESIGN

Code: AASHTO LRFD 8
Units: US
Pier View: Downstation.

DESIGN PARAMETERS

f'c = 5500.0 psi	
Fy flex = 60000.0 psi	Fy shear = 60000.0 psi
phi tens = 0.90	
phi comp = 0.75	phi shear = 0.90
Tens above = 0.005	Comp below = 0.002
Ec = 4428.3 ksi	Es = 29000.0 ksi
Crack check as per current LRFD	
Crack control Exposure = 1.00	
Concrete Type : Normal Weight.	
Design of cap at face of column.	

CAP GEOMETRY

Straight Cap : Length(X) = 13.67 ft Depth(Z) = 44.00 in

Cap Section Properties

Sec.	Area ft ^2	Iz in ^4	Iy in ^4
1	9.17	99000.00	212960.00

MAIN REINFORCEMENT

	Bar size	Quantity	Bar dist. in	As total in^2	From ft	To ft	Hook
TOP	US#5[M16]	4	4.00 OK	1.240	0.00	13.67	None
BOTTOM	US#5[M16]	4	4.00 OK	1.240	0.00	13.67	None

Note: In accordance with LRFD Section 5.10.6, longitudinal reinforcement spacing shall not exceed 12". 2 #5 bars will be added to each vertical face of the intermediate bent caps. #4 hooked bars shall be included in the bottom layer between the piles. The bars noted here are conservatively excluded from end bent capacity calculations.

STIRRUPS

From ft	To ft	Stirrup Size	n legs	Spacing in	Aprv/s in^2 / ft	Bar Type
0.00	13.67	US#4[M13]	2	12.00	0.40	Stirrup

Clear Cover on Sides = 3.00 in

FLEXURE DESIGN

Span 1: From 0.00 ft To 1.75 ft

Loc ft	AbsLoc ft	H in	Mmax Mmin kips-ft	Mr kips-ft	Comb	CL	c	dt in	eps_t	Phi	Asb-req in^2	Asb-prv in^2	Asb-eff in^2	Ast-req in^2	Ast-prv in^2	Ast-eff in^2
1.0	1.0	30	0.0	163.4	0	T	0.94	26.00	0.080	0.90	0.71	1.24	1.24	0.71	1.24	1.24
			-0.9	-163.4	1	T	0.94	26.00	0.080	0.90	0.71	1.24	1.24	0.71	1.24	1.24

Span 2: From 1.75 ft To 6.83 ft

Loc ft	AbsLoc ft	H in	Mmax Mmin kips-ft	Mr kips-ft	Comb	CL	c	dt in	eps_t	Phi	Asb-req in^2	Asb-prv in^2	Asb-eff in^2	Ast-req in^2	Ast-prv in^2	Ast-eff in^2
0.8	2.5	30	45.5	163.4	3	T	0.94	26.00	0.080	0.90	0.71	1.24	1.24	0.71	1.24	1.24
			0.0	-163.4	0	T	0.94	26.00	0.080	0.90	0.71	1.24	1.24	0.71	1.24	1.24
4.3	6.1	30	0.0	163.4	0	T	0.94	26.00	0.080	0.90	0.71	1.24	1.24	0.71	1.24	1.24
			-17.3	-163.4	3	T	0.94	26.00	0.080	0.90	0.71	1.24	1.24	0.71	1.24	1.24

Span 3: From 6.83 ft To 11.92 ft

Loc ft	AbsLoc ft	H in	Mmax Mmin kips-ft	Mr kips-ft	Comb	CL	c	dt in	eps_t	Phi	Asb-req in^2	Asb-prv in^2	Asb-eff in^2	Ast-req in^2	Ast-prv in^2	Ast-eff in^2
0.8	7.6	30	0.0	163.4	0	T	0.94	26.00	0.080	0.90	0.71	1.24	1.24	0.71	1.24	1.24
			-17.3	-163.4	3	T	0.94	26.00	0.080	0.90	0.71	1.24	1.24	0.71	1.24	1.24
4.3	11.2	30	45.5	163.4	3	T	0.94	26.00	0.080	0.90	0.71	1.24	1.24	0.71	1.24	1.24
			0.0	-163.4	0	T	0.94	26.00	0.080	0.90	0.71	1.24	1.24	0.71	1.24	1.24

Span 4: From 11.92 ft To 13.67 ft

Loc ft	AbsLoc ft	H in	Mmax Mmin kips-ft	Mr kips-ft	Comb	CL	c	dt in	eps_t	Phi	Asb-req in^2	Asb-prv in^2	Asb-eff in^2	Ast-req in^2	Ast-prv in^2	Ast-eff in^2
0.8	12.7	30	0.0	163.4	0	T	0.94	26.00	0.080	0.90	0.71	1.24	1.24	0.71	1.24	1.24
			-0.9	-163.4	1	T	0.94	26.00	0.080	0.90	0.71	1.24	1.24	0.71	1.24	1.24

Flexure Design : Notes

CL: Section classification as per LRFD 2006 interims for provided reinforcement.
C = Compression controlled, I = In-Transition, T = Tension controlled.

SHEAR AND TORSION DESIGN:

Span 1: From 0.00 ft To 1.75 ft

Loc ft	AbsLoc ft	Pos	Vu kips	Comb	Tu kips-ft	Comb	phi*Vn kips	T-lim kips-ft	Avs/s in^2/ft	Ats/s in^2/ft	Av/s in^2/ft	Aprv/s in^2/ft	Alx in^2	Vc kips	Vs kips	Beta	Theta deg	b in	dv in	Eps_s
1.00	1.00	L	1.7	1	0.0	0	419.3	65.2	0.00	0.00	0.00	0.40	0.00	374.69	91.24	4.48	29.33	44.00	25.64	0.0001

Span 2: From 1.75 ft To 6.83 ft

Loc ft	AbsLoc ft	Pos	Vu kips	Comb	Tu kips-ft	Comb	phi*Vn kips	T-lim kips-ft	Avs/s in^2/ft	Ats/s in^2/ft	Av/s in^2/ft	Aprv/s in^2/ft	Alx in^2	Vc kips	Vs kips	Beta	Theta deg	b in	dv in	Eps_s
0.75	2.50	R	14.4	3	0.0	0	278.6	65.2	0.00	0.00	0.00	0.40	0.00	229.13	80.45	2.74	32.51	44.00	25.64	0.0010
4.33	6.08	L	20.6	3	0.0	0	264.6	65.2	0.00	0.00	0.00	0.40	0.00	215.14	78.82	2.57	33.04	44.00	25.64	0.0012

Span 3: From 6.83 ft To 11.92 ft

Loc ft	AbsLoc ft	Pos	Vu kips	Comb	Tu kips-ft	Comb	phi*Vn kips	T-lim kips-ft	Avs/s in^2/ft	Ats/s in^2/ft	Av/s in^2/ft	Aprv/s in^2/ft	Alx in^2	Vc kips	Vs kips	Beta	Theta deg	b in	dv in	Eps_s
0.75	7.58	R	20.6	3	0.0	0	266.3	65.3	0.00	0.00	0.00	0.40	0.00	216.83	79.03	2.59	32.98	44.00	25.64	0.0011
4.33	11.17	L	14.4	3	0.0	0	280.5	65.3	0.00	0.00	0.00	0.40	0.00	231.00	80.65	2.76	32.45	44.00	25.64	0.0010

Span 4: From 11.92 ft To 13.67 ft

Loc ft	AbsLoc ft	Pos	Vu kips	Comb	Tu kips-ft	Comb	phi*Vn kips	T-lim kips-ft	Avs/s in^2/ft	Ats/s in^2/ft	Av/s in^2/ft	Aprv/s in^2/ft	Alx in^2	Vc kips	Vs kips	Beta	Theta deg	b in	dv in	Eps_s
0.75	12.67	R	1.7	1	0.0	0	419.3	65.2	0.00	0.00	0.00	0.40	0.00	374.68	91.24	4.48	29.33	44.00	25.64	0.0001

Shear and Torsion Design : Notes

- Pos is the design position. L suggests the calculation is done at immediate left of "Loc" and R suggests at immediate right of it.
- T-lim is the limiting value of torsion for the concrete section. If actual torsion is higher than this value, torsional steel has to be provided.
- Avs/s is the required area of steel per unit length for shear force.
- Ats/s is the required area of steel per unit length for one leg of torsional reinforcement.
- Av/s is the total required area of steel per unit length due to shear plus torsion.
- Aprvs/s is the total provided area of transverse steel reinforcement.
- Alx is the EFFECTIVE longitudinal steel required in addition to the PROVIDED EFFECTIVE flexural steel.
- Vc is the nominal shear resistence of concrete.
- Vs is the nominal shear resistence of transverse reinforcement.
- Beta is the factor indicating ability of diagonally cracked concrete to transmit tension and shear.
- Theta is the angle of inclination of diagonal compressive stress.
- # Vu is greater tha phi*Vn.

CRACKING/FATIGUE CHECK

Span 1: From 0.00 ft To 1.75 ft

Loc ft	AbsLoc ft	H in	Cracking Comb	Cracking fs-t fs-b ksi	Cracking dc in	Cracking Srqt Srqb in	Cracking Sprt Sprb in	Fatigue stress range fs-t fs-b ksi	Fatigue threshold FTH-t FTH-b ksi
1.00	1.0	30.0	31	0.3	2.3	44.0	12.1	0.00	0.00
			0	0.0	2.3	5.0	12.1	0.00	0.00

Span 2: From 1.75 ft To 6.83 ft

Loc ft	AbsLoc ft	H in	Cracking Comb	Cracking fs-t fs-b ksi	Cracking dc in	Cracking Srqt Srqb in	Cracking Sprt Sprb in	Fatigue stress range fs-t fs-b ksi	Fatigue threshold FTH-t FTH-b ksi
0.75	2.5	30.0	0	0.0	2.3	5.0	12.1	0.00	0.00
			33	13.2	2.3	42.9	12.1	0.00	0.00
4.33	6.1	30.0	33	5.1	2.3	44.0	12.1	0.00	0.00
			0	0.0	2.3	5.0	12.1	0.00	0.00

Span 3: From 6.83 ft To 11.92 ft

Loc ft	AbsLoc ft	H in	Cracking Comb	Cracking fs-t fs-b ksi	Cracking dc in	Cracking Srqt Srqb in	Cracking Sprt Sprb in	Fatigue stress range fs-t fs-b ksi	Fatigue threshold FTH-t FTH-b ksi
0.75	7.6	30.0	58	5.1	2.3	44.0	12.1	0.00	0.00
			0	0.0	2.3	5.0	12.1	0.00	0.00
4.33	11.2	30.0	0	0.0	2.3	5.0	12.1	0.00	0.00
			58	13.2	2.3	42.8	12.1	0.00	0.00

Span 4: From 11.92 ft To 13.67 ft

Loc ft	AbsLoc ft	H in	Cracking Comb	Cracking fs-t fs-b ksi	Cracking dc in	Cracking Srqt Srqb in	Cracking Sprt Sprb in	Fatigue stress range fs-t fs-b ksi	Fatigue threshold FTH-t FTH-b ksi
0.75	12.7	30.0	31	0.3	2.3	44.0	12.1	0.00	0.00
			0	0.0	2.3	5.0	12.1	0.00	0.00

Cracking and fatigue Check : Notes

* Cracking / fatigue checking failed.
Required bar spacing, Sreq, should not be less than 5 in per Art. 5.7.3.4.

Note:

Please note the analysis is based on the assumption that the cap is infinitely rigid which results into only axial forces in the piles.

Pile Reactions, Service **Load Effect @ Pile Cap Bot.**

Pile	X in	Z in	Batter X degree	Batter Z degree	comb	Ovs	P kips	Mxx kft	Mzz kft	Pile Reac. kips
1	21.0	0.0	0	0	58	1.000	-276.43	0.00	15.76	93.69
					32	1.000	-238.66	0.00	-35.14	76.10
2	82.0	0.0	0	0	58	1.000	-276.43	0.00	15.76	92.14
					81	1.000	-234.77	0.00	15.61	78.26
3	143.0	0.0	0	0	33	1.000	-276.43	0.00	-15.65	93.68
					56	1.000	-238.66	0.00	35.21	76.09

Pile Reactions, Factored **Load Effect @ Pile Cap Bot.**

Pile	X in	Z in	Batter X degree	Batter Z degree	comb	Ovs	P kips	Mxx kft	Mzz kft	Pile Reac. kips
1	21.00	0.0	0	0	3	---	-374.16	0.00	0.07	124.73
					21	---	-197.27	0.00	-78.96	57.99
2	82.00	0.0	0	0	3	---	-374.16	0.00	0.07	124.72
					21	---	-197.27	0.00	-78.96	65.76
3	143.00	0.0	0	0	3	---	-374.16	0.00	0.07	124.71
					26	---	-197.27	0.00	79.03	57.98

Nominal Bearing Resistance = 124.72 kip / 0.65 = 191.88 kip = 96 ton

Pile Reactions: Notes

Pile reaction calculation is based on the assumption that pile cap is infinitely rigid. Load effects on pile are calculated at CG of pile group.
Both the max. and min. pile reaction are reported for each individual pile.
Positive pile reaction represents pile subject to compression load; negative pile reaction represents pile subject to uplift.
Coordinate system of pile layout see Geometry Tab>Footing Pile>Edit Pile.

Pile Lateral Resistance Check X

Controlling load combination number	21
Lateral loads on all piles(kips)	22.225
Batter angle(deg)	0.000
Available lateral resistance due to batter(kips)	0.000
Direct shear resistance of all piles(kips)	1170.000
Total pile lateral resistance(kips)	1170.000
Is the lateral resistance due to batter greater than lateral load on all piles?	NO
Is the total pile lateral resistance greater than the lateral load on all piles?	YES

Pile Lateral Resistance Check Z

Controlling load combination number	25
Lateral loads on all piles(kips)	2.978
Batter angle(deg)	0.000
Available lateral resistance due to batter(kips)	0.000
Direct shear resistance of all piles(kips)	1170.000
Total pile lateral resistance(kips)	1170.000
Is the lateral resistance due to batter greater than lateral load on all piles?	NO
Is the total pile lateral resistance greater than the lateral load on all piles?	YES

Intermediate Bent Design

AASHTO LRFD Bridge Design Specifications (LRFD) and FDOT Structures Design Guidelines (SDG) are referenced.

Bridge Geometry

	BACK SPAN	AHEAD SPAN
Span Length:	$Span_{Bk} := 53 \cdot ft + 0 \cdot in$	$Span_{Ah} := 53 \cdot ft + 0 \cdot in$
Number of Beams:	$NoBeam_{Bk} := 3$	$NoBeam_{Ah} := 3$
Average Beam Tributary Widths:	$i := 1 .. NoBeam_{Bk}$	$j := 1 .. NoBeam_{Ah}$
	$BeamTA_{Bk_i} :=$	$BeamTA_{Ah_j} :=$
	$4 \cdot ft + 6 \cdot in$	$4 \cdot ft + 6 \cdot in$
	$4 \cdot ft + 7 \cdot in$ OK	$4 \cdot ft + 7 \cdot in$ OK
	$4 \cdot ft + 6 \cdot in$	$4 \cdot ft + 6 \cdot in$
Bridge Skew:		$\phi := 0 \cdot deg$
FSB Height:		$h_{FSB} := 15 \cdot in$

Superstructure Details

Bridge Width:	$b_{bridge} := \sum BeamTA_{Bk}$	$b_{bridge} = 13.583\,ft$
Topping Thickness:		$Deck := 6 \cdot in$
FSB Width:		$b_{FSB} := 4 \cdot ft + 6 \cdot in$
Number of Traffic Railings:		$No_{t.rail} := 0$
Number of Pedestrian Railings:		$No_{p.rail} := 2$

Miscellaneous Dead Loads (SDG Table 2.2-1)

Concrete Unit Weight: $\gamma_c := 150 \cdot pcf$

Future Wearing Surface Weight: $w_{fws} := 0 \cdot psf$

Soil Unit Weight, Compacted: $\gamma_{soil} := 115 \cdot pcf$

Beam Weight: *(Bk & Ah)* $w_{beam.Bk} := 0 \cdot plf$ $w_{beam.Ah} := 0 \cdot plf$

NOTE: Superstructure dead load evaluated below in combination with slab

Traffic / Pedestrian Railing Weight: $w_{t.rail} := 0 \cdot plf$ $w_{p.rail} := 80 \cdot plf$

Note(s):
Weight increased to include concrete pedestal

Dead Loads (DC)

BEARING LOADS

BEAM AND SLAB (COMBINED)

Beam Weight - Back: (per bearing)

$$Beam_{Bk} := w_{beam.Bk} \cdot \left(\frac{Span_{Bk}}{2}\right)$$

$$Beam_{Bk} = 0.0 \cdot kip$$

Beam Weight - Ahead: (per bearing)

$$Beam_{Ah} := w_{beam.Ah} \cdot \left(\frac{Span_{Ah}}{2}\right)$$

$$Beam_{Ah} = 0.0 \cdot kip$$

SLAB

Slab Weight - Back: (per bearing)

$$Slab_{Bk} := \gamma_c \cdot \left(\sum_{k=1}^{NoBeam_{Bk}} BeamTA_{Bk_k}\right) \cdot \left(h_{FSB} + Deck\right) \cdot \left(\frac{Span_{Bk}}{2}\right) \div NoBeam_{Bk}$$

$$Slab_{Bk} = 31.5 \cdot kip$$

Slab Weight - Ahead: (per bearing)

$$Slab_{Ah} := \gamma_c \cdot \left(\sum_{k=1}^{NoBeam_{Ah}} BeamTA_{Ah_k}\right) \cdot \left(h_{FSB} + Deck\right) \cdot \left(\frac{Span_{Ah}}{2}\right) \div NoBeam_{Ah}$$

$$Slab_{Ah} = 31.5 \cdot kip$$

Dead Loads (DC) Cont'd...

BEARING LOADS CONT'D...

BEAM BUILD-UP

Build-Up Dimensions: (estimate, SPI 450-450, case 1-3 eq. assumed)

Dimension B (assumed): $B := 1.0 \cdot in$

Dimension C (assumed): $C := 0 \cdot in$

Dimension D (assumed): $D := 1.0 \cdot in$

BeamTA.Bk (Say OK)

Build-Up Weight - Back: (per bearing)

$$BuildUp_{Bk} := \gamma_c \cdot b_{FSB} \cdot \left(C + \frac{B + D - 2 \cdot C}{6}\right) \cdot \left(\frac{Span_{Bk}}{2}\right)$$

$$BuildUp_{Bk} = 0.5 \cdot kip$$

Build-Up Weight - Ahead: (per bearing)

$$BuildUp_{Ah} := \gamma_c \cdot b_{FSB} \cdot \left(C + \frac{B + D - 2 \cdot C}{6}\right) \cdot \left(\frac{Span_{Ah}}{2}\right)$$

$$BuildUp_{Ah} = 0.5 \cdot kip$$

BeamTA.Ah (Say OK)

Dead Loads (DC) Cont'd...

BEARING LOADS CONT'D...

BARRIER

Barrier Weight - Back: (per bearing)

$$Rail_{Bk} := \frac{No_{t.rail} \cdot w_{t.rail} + No_{p.rail} \cdot w_{p.rail}}{NoBeam_{Bk}} \cdot \left(\frac{Span_{Bk}}{2}\right) \qquad Rail_{Bk} = 1.4 \cdot kip$$

Barrier Weight - Ahead: (per bearing)

$$Rail_{Ah} := \frac{No_{t.rail} \cdot w_{t.rail} + No_{p.rail} \cdot w_{p.rail}}{NoBeam_{Ah}} \cdot \left(\frac{Span_{Ah}}{2}\right) \qquad Rail_{Ah} = 1.4 \cdot kip$$

TOTAL LOADS

Total DC Dead Load: (per bearing)

$$DC_{Bk} := Beam_{Bk} + Slab_{Bk} + BuildUp_{Bk} + Rail_{Bk}$$

$$DC_{Ah} := Beam_{Ah} + Slab_{Ah} + BuildUp_{Ah} + Rail_{Ah}$$

BACK SPAN

$$-DC_{Bk} = -33.4 \cdot kip$$

AHEAD SPAN

$$-DC_{Ah} = -33.4 \cdot kip$$

Note(s):
n/a

Dead Loads (DC) Cont'd...

CAP LOADS

Consider a point load on the cap due to the cheekwall self-weight.

Pier Cap Width: $b_{cap} := 3 \cdot ft + 8 \cdot in$

Cheekwall Height: $h_{cheek} := 0 \cdot ft + 0 \cdot in$

Cheekwall Thickness: $t_{cheek} := 0 \cdot in$

Cheekwall Load: $DC_{cap} := \gamma_c \cdot h_{cheek} \cdot t_{cheek} \cdot b_{cap}$ $-DC_{cap} = 0.00 \cdot kip$

Note(s):
Apply at x1/L = n/a

FOOTING LOADS

Consider a uniform line load along the z-axis of the footing due to the seal weight (assume 90% of the footing area given pile embedment).

Depth of Seal: $h_{seal} := 0.0 \cdot ft$

Uniform Seal Pressure: $p_{seal} := 0.9 \cdot \gamma_c \cdot h_{seal}$ $-p_{seal} = 0.000 \cdot ksf$

Note(s):
Convert pressure to a uniform line load given the specific footing size.

MISCELLANEOUS LOADS

n/a

Dead Loads (DW)

BEARING LOADS

WEARING SURFACE

Wearing Surface - Back: (per bearing)

$$FWS_{Bk} := w_{fws} \cdot BeamTA_{Bk} \cdot \left(\frac{Span_{Bk}}{2}\right)$$

$$FWS_{Bk}^{T} = \begin{pmatrix} 0.0 & 0.0 & 0.0 \end{pmatrix} \cdot kip$$

Wearing Surface - Ahead: (per bearing)

$$FWS_{Ah} := w_{fws} \cdot BeamTA_{Ah} \cdot \left(\frac{Span_{Ah}}{2}\right)$$

$$FWS_{Ah}^{T} = \begin{pmatrix} 0.0 & 0.0 & 0.0 \end{pmatrix} \cdot kip$$

UTILITIES

Utility Weight: (per linear foot of bridge)

$$w_{util} := 0.0 \cdot plf$$

Utility Weight - Back: (per bearing)

$$Util_{Bk} := w_{util} \cdot \left(\frac{Span_{Bk}}{2}\right) \div NoBeam_{Bk}$$

$$Util_{Bk} = 0.0 \cdot kip$$

Utility Weight - Ahead: (per bearing)

$$Util_{Ah} := w_{util} \cdot \left(\frac{Span_{Ah}}{2}\right) \div NoBeam_{Ah}$$

$$Util_{Ah} = 0.0 \cdot kip$$

TOTAL LOADS

Total DW Dead Load: (per bearing)

$$DW_{Bk} := FWS_{Bk} + Util_{Bk}$$

$$DW_{Ah} := FWS_{Ah} + Util_{Ah}$$

BACK SPAN

$$-DW_{Bk} = \begin{pmatrix} 0.00 \\ 0.00 \\ 0.00 \end{pmatrix} \cdot kip$$

AHEAD SPAN

$$-DW_{Ah} = \begin{pmatrix} 0.00 \\ 0.00 \\ 0.00 \end{pmatrix} \cdot kip$$

Note(s):
n/a

Uniform Temperature Loads (TU)

Uniform temperature loads include the forces from the superstructure to the substructure due to temperature, creep, and shrinkage. Bearing pad properties are per FDOT Index 450-450 Standard Plans Instructions. Creep is ignored assuming that the composite deck will offer restraint.

BEARING LOADS

CST Length: $CST := 0 \cdot ft$

Coefficient of Thermal Expansion: (LRFD 5.4.2.2) $\alpha := 6 \cdot 10^{-6} \cdot F^{-1}$

Temperature Rise/Fall: (SDG 2.7.1) $\Delta T := 35 \cdot F$

Bearing Pad Area: $Pad_{area} := (8 \cdot in) \cdot (42 \cdot in)$

Bearing Pad Elastomeric Thickness: $Pad_{thick} := 1 \cdot in$

Shear Modulus: $G := 110 \cdot psi$

Shrinkage Strain: *(assumed)* $\varepsilon_{sh} := 0.0001$

Temperature Strain: $\varepsilon_{temp} := \alpha \cdot \Delta T$ $\varepsilon_{temp} = 0.00021$

Total Strain: $\varepsilon_{total} := \varepsilon_{sh} + \varepsilon_{temp}$ $\varepsilon_{total} = 0.00031$

Displacement Due to Strain: $\Delta_{pier} := CST \cdot (\varepsilon_{sh} + \varepsilon_{temp})$ $\Delta_{pier} = 0 \cdot in$

Force Transferred by Pad: (per bearing) $TU_{CST} := \Delta_{pier} \cdot \frac{G \cdot Pad_{area}}{Pad_{thick}}$ $TU_{CST} = 0 \cdot kip$

Longitudinal Force: (per bearing) $TU_{long} := TU_{CST} \cdot \cos(\phi)$ $TU_{long} = 0.00 \cdot kip$

Transverse Force: (per bearing) $TU_{trans} := TU_{CST} \cdot \sin(\phi)$ $TU_{trans} = 0.00 \cdot kip$

Note(s):
Apply in both the (+/-) directions.

CAP STRAIN

Temperature Cap Strain: $TU_{\varepsilon.cap} := \varepsilon_{temp}$ $TU_{\varepsilon.cap} = 0.00021$

Note(s):
Applicable to multi-column piers. Apply in both the (+/-) directions.

Shrinkage Force Effects (SH)

Pier shrinkage effects are considered as a strain load.

Shrinkage Strain: $SH_{\varepsilon.cap} := \text{"N/A"}$

Note(s):
Applicable to multi-column piers. Apply in (-) direction (contraction).

Vertical Pressure from Dead Load of Earth Fill (EV)

Vertical pressure due to earth fill for buried structures is considered in Leap Substructure as a uniform line load on the footing.

Depth of Soil Fill: $h_{soil} := 0.0 \cdot ft$

Uniform Soil Pressure: $p_{soil} := \gamma_{soil} \cdot h_{soil}$ $\quad -p_{soil} = 0.000 \cdot ksf$

Note(s):
Convert pressure to a uniform line load given the specific footing size.

Water Load (WA)

Stream pressure is calculated per LRFD 3.7.3.

Longitudinal Drag Coefficient:
(LRFD Table 3.7.3.1-1) $C_D := 0.0$

Lateral Drag Coefficient:
(LRFD Table 3.7.3.2-1) $C_L := 0.0$

Design Velocity of Water: $V := 0.0 \cdot (ft \div s)$

Longitudinal Pressure:
(LRFD Eq. 3.7.3.1-1) $p_D := \dfrac{C_D \cdot [V \div (ft \div s)]^2}{1000} \cdot ksf$ $\quad p_D = 0.000 \cdot ksf$

Lateral Pressure:
(LRFD Eq. 3.7.3.2-1) $p_L := \dfrac{C_L \cdot [V \div (ft \div s)]^2}{1000} \cdot ksf$ $\quad p_L = 0.000 \cdot ksf$

Note(s):
Stream pressure force neglected, negligible.

Live Loads (LL)

LIve load is generated for the H10 truck loading in Leap Substructure. Pedestrian loading is calculated below per AASHTO LRFD for Ped. Bridges.

Truck Loads

Generated in RCPier.

Pedestrian Loads

Pedestrian loading: $Ped := 90psf$

Trail Width on Bridge: $W_{trl} := 12ft + 0in$

Ped load per bearing: $PL_{Bk} := \frac{Span_{Bk}}{2} \cdot Ped \cdot W_{trl} \div 3$ $\quad -PL_{Bk} = -9.54 \cdot kip$

$PL_{Ah} := \frac{Span_{Ah}}{2} \cdot Ped \cdot W_{trl} \div 3$ $\quad -PL_{Ah} = -9.54 \cdot kip$

Braking Loads (BR)

n/a

Centrifugal Loads (CE)

n/a

Live Loads (LLf)

n/a

Dynamic Load Allowance (IM)

n/a

Wind Load on Structure (WS)

Wind load on structure forces are generated in Leap Substructure per FDOT SDG 2.4 and LRFD 3.8. The wind angle varies from 0 to 60 deg. in 15 deg. intervals. The height of the barrier input into Leap Substructure is modified to account for the effect of bridge cross-slope. The Leap Substructure bearing pad depth is modified to match the approx. build-up height for superstructure depth estimate purposes.

Design Wind Speed: (SDG Table 2.4.1-1) — *(Lee County - assume Exposure C, G = 0.85)* — $V_{wind} := 170 \cdot mph$

Controlling Railing Height: — *Note(s): Use effective railing height to account for reduced area subject to wind pressure of pedestrian rail.* — $h_{rail} := 32 \cdot in$

Avg. Height to Top of Superstructure: — *(height < 33 ft.)* — $h_{avg} := 33 \cdot ft$ OK

Superstructure Height: — $Z := \max(h_{avg}, 33 \cdot ft)$ — $Z = 33.0 \cdot ft$

Vertical Wind Load

Apply a vertical upward wind load at the windward quarter-point of the deck width in accordance with AASHTO 3.8.2. This load is applied in FB-Pier in conjunction with the STRIII wind loads for evaluation of lateral stability.

Vertical Wind Pressure: (LRFD 3.8.2) — $VP := 0.020ksf$

Vertical Wind Force: — $P_{VP} := VP \cdot b_{bridge} \cdot Span_{Bk} \div 2$ — $P_{VP} = 7.20 \cdot kip$

Vertical Wind Force: (Distributed to exterior beams assuming rigid behavior)

$$M := P_{VP} \cdot \left(\frac{1}{4} \cdot b_{bridge}\right) \qquad d := \begin{bmatrix} (b_{FSB} + 1in) \\ -(b_{FSB} + 1in) \end{bmatrix}$$

$$P_{VP} := \frac{P_{VP}}{2} + \frac{M \cdot d}{\sum d^2} \qquad P_{VP} = \begin{pmatrix} 6.27 \\ 0.93 \end{pmatrix} \cdot kip$$

Wind on Live Load (WL)

Wind on live load is not considered.

Vehicular Collision Force (CT)

n/a

Vessel Collision Force (CV)

n/a

3.4. End Bent Wingwall Design

Wingwall Design

Design the wingwall for cantilever moment and soil forces.

References

AASHTO LRFD Bridge Design Specifications (LRFD), 9th Edition

Material Properties

Concrete Strength: $f_c := 5.5ksi$ •

Concrete Modulus: (LRFD C5.4.2.4) $E_c := 120000 \cdot 0.145^{2.0} \cdot \left(\frac{f_c}{ksi}\right)^{0.33} \cdot ksi$ $E_c = 4428.30 \cdot ksi$ •

Concrete Unit Weight: $\gamma_c := 150pcf$ •

Steel Yield Strength: $f_y := 60ksi$ •

Steel Modulus: $E_s := 29000ksi$ •

Soil Unit Weight: (assumed) $\gamma_{soil} := 112pcf$ OK

Load and Reistance Factors

Flexure: (LRFD 5.5.4.2) $\phi_f := 0.9$ •

Dead Load: $\lambda_{DL} := 1.25$ •

Live Load: $\lambda_{LL} := 1.75$ •

Earth Pressure: $\lambda_{EH} := 1.50$ •

Uniform Surcharge: $\lambda_{ES} := 1.50$ •

Live Load Surcharge: $\lambda_{LS} := 1.75$ •

Passive Pressure: $\lambda_p := 0.9$ •

Geometry

Max Wingwall Height:		$h_{ww} := 4.45 \cdot ft$
Wingwall Thickness:		$t_{ww} := 1ft$
Wingwall Length:		$l_{ww} := 3.5ft$
Wingwall Surface Area:	$A_{ww} := l_{ww} \cdot h_{ww}$	$A_{ww} = 15.575\,ft^2$
Centroid Location from Left:	$NA_x := l_{ww} \div 2$	$NA_x = 1.75\,ft$
Centroid Location from Bottom:	$NA_y := h_{ww} \div 2$	$NA_y = 2.225\,ft$
Approach Slab Thickness:		$t_{as} := 13.75in$

Proposed Reinforcement

Clear Cover: $cover := 4in$ Bottom cover

Reinforcing Bar Size: $bar := 4$

$$\phi(bar) := \begin{cases} \phi \leftarrow 0.500in & \text{if } bar = 4 \\ \phi \leftarrow 0.625in & \text{if } bar = 5 \\ \phi \leftarrow 0.750in & \text{if } bar = 6 \\ \phi \leftarrow 0.875in & \text{if } bar = 7 \end{cases}$$

$$A(bar) := \begin{cases} A \leftarrow 0.20in^2 & \text{if } bar = 4 \\ A \leftarrow 0.31 \cdot in^2 & \text{if } bar = 5 \\ A \leftarrow 0.44 \cdot in^2 & \text{if } bar = 6 \\ A \leftarrow 0.60 \cdot in^2 & \text{if } bar = 7 \end{cases}$$

Reinforcing Bar Diameter: $dia_{bar} := \phi(bar)$ $dia_{bar} = 0.5 \cdot in$

Reinforcing Bar Area: $area_{bar} := A(bar)$ $area_{bar} = 0.2 \cdot in^2$

Analyze Cantilever Moment

Wingwall Weight Moment: $M_{DL} := \lambda_{DL} \cdot A_{ww} \cdot t_{ww} \cdot \gamma_c \cdot NA_x$ $\boxed{M_{DL} = 5.111 \cdot kip \cdot ft}$

Assumed Maintenance Equipment Loads:
Note: To be conservitive, point load placed at end of wingwall.

Maintenance Surcharge:
(assumed 12.5 kip point load) $P := 12.5 \cdot kip$

Maint. Equip. Weight Moment: $M_{LL} := \lambda_{LL} \cdot P \cdot l_{ww}$ $\boxed{M_{LL} = 76.562 \cdot kip \cdot ft}$

Depth to Centroid of Tension Reinforcement from Extreme Compression Fiber:
(assume top two bars) $d_p := h_{ww} - cover - 0.5 \cdot dia_{bar}$ $d_p = 49.15 \cdot in$

Wingwall Top Reinforcement: $A_s := 2 \cdot area_{bar}$ $A_s = 0.40 \cdot in^2$

Equivalent Stress Block Depth: $a_p := A_s \cdot f_y \div (0.85 \cdot f_c \cdot t_{ww})$ $a_p = 0.43 \cdot in$

Nominal Flexural Resistance: $M_n := (A_s \cdot f_y) \cdot (d_p - 0.5 \cdot a_p)$ $M_n = 97.87 \cdot kip \cdot ft$

Factored Flexural Resistance: $M_r := \phi_f \cdot M_n$ $\boxed{M_r = 88.08 \cdot kip \cdot ft}$

Check Flexural Resistance: $Chk_M_r := if\left[(M_{DL} + M_{LL}) < M_r, \text{"OK"}, \text{"N.G"}\right]$ $Chk_M_r = \text{"OK"}$

Analyze Earth Pressure Moment

Design Height: $h_{des} := h_{ww}$ $h_{des} = 4.45\,ft$

Slope of Tapered Section: $m := \frac{10ft - 6in}{4.75ft}$ $m = 2$

Length of Wing Wall at Design Height: $L_{des} := 10ft - m \cdot \left(h_{des} - FIF(3, 10.75)\right)$ $L_{des} = 8.892\,ft$

Active Earth Pressure Coefficient: *(assumed)* $k_a := 0.32$

Approach Slab Surcharge: $\sigma_{AS} := t_{as} \cdot \gamma_c \cdot k_a$ $\sigma_{AS} = 0.382\,psi$

Point Load: $P_{AS} := \lambda_{ES} \cdot \sigma_{AS} \cdot l_{ww} \cdot h_{ww}$ $P_{AS} = 1.285 \cdot kip$

Lever Arm Length: $LA_{AS} := NA_x$ $LA_{AS} = 1.75\,ft$

Assumed Maintenance Equipment Loads:
Note: Elastic method used to determine load effects. LRFD 3.11.6.2

Maintenance Surcharge: *(assumed 12.5 kip point load)* $P = 12.5\,kip$

Radial Distance from Point Load to a Point on the Wall: *(assumed 2 ft from edge of wingwall)*

$X := 2.0 \cdot ft \quad Y := 1.0 \cdot ft \quad z := 0 .. h_{des} \div ft$

$R(z) := \sqrt{X^2 + Y^2 + (z \cdot ft)^2}$

Poisson's Ratio: *(assumed)* $\nu := 0.30$

Horizontal Pressure from Point Load: $\Delta(z) := \frac{P}{\pi \cdot (R(z))^2} \cdot \left[\frac{3 \cdot z \cdot ft \cdot X^2}{(R(z))^3} - \frac{R(z) \cdot (1 - 2 \cdot \nu)}{R(z) + z \cdot ft}\right]$

Resultant Force: $P_{Maint} := \lambda_{ES} \cdot \int_0^{h_{des} \div ft} \Delta(z) \cdot l_{ww} \cdot z \cdot ft \, dz$ $P_{Maint} = 7.881\,kip$

Lever Arm: $LA_{Maint} := Y$ $LA_{Maint} = 1\,ft$

Uniform Surcharge Moment: $M_{ES} := P_{AS} \cdot LA_{AS} + P_{Maint} \cdot LA_{Maint}$ $\boxed{M_{ES} = 10.13 \cdot kip \cdot ft}$

Live Load Surcharge:
(LRFD Table 3.11.6.4-2)

$\sigma_{LS} := k_a \cdot \gamma_{soil} \cdot 2ft$ $\quad \sigma_{LS} = 0.498\,psi$

Point Load: $P_{LS} := \lambda_{LS} \cdot \sigma_{LS} \cdot l_{ww} \cdot h_{ww}$ $\quad P_{LS} = 1.954 \cdot kip$

Lever Arm Length: $LA_{LS} := NA_x$ $\quad LA_{LS} = 1.75\,ft$

Uniform Live Load Surcharge Moment: $M_{LS} := P_{LS} \cdot LA_{LS}$ $\quad \boxed{M_{LS} = 3.42 \cdot kip \cdot ft}$

Active Earth Pressure:

Pressure: $\sigma_{EH} := \frac{1}{2} \cdot k_a \cdot \gamma_{soil} \cdot (h_{ww} - t_{as})$ $\quad \sigma_{EH} = 0.411\,psi$

Point Load: $P_{EH} := \lambda_{EH} \cdot \sigma_{EH} \cdot l_{ww} \cdot (h_{ww} - t_{as})$ $\quad P_{EH} = 1.027 \cdot kip$

Lever Arm Length: $LA_{EH} := NA_x$ $\quad LA_{EH} = 1.75\,ft$

Active Earth Pressure Moment: $M_{EH} := P_{EH} \cdot LA_{EH}$ $\quad \boxed{M_{EH} = 1.8 \cdot kip \cdot ft}$

Total Moment from Active Side of Wing Wall: $M_a := M_{ES} + M_{LS} + M_{EH}$ $\quad M_a = 15.35 \cdot kip \cdot ft$

Passive Earth Pressure:

Passive Pressure Coefficient:		$k_p := 3.12$
Height of Fill from Bottom of Wing Wall:		$h_{fill} := 0 \cdot ft$
Pressure:	$\sigma_p := \frac{1}{2} \cdot k_p \cdot \gamma_{soil} \cdot h_{fill}$	$\sigma_p = 0 \cdot psi$
Point Load:	$P_p := \lambda_p \cdot \sigma_p \cdot l_{ww} \cdot h_{fill}$	$P_p = 0 \cdot kip$
Lever Arm Length:	$LA_p := NA_x$	$LA_p = 1.75\,ft$
Passive Earth Pressure Moment:	$M_p := P_p \cdot LA_p$	$\boxed{M_p = 0 \cdot kip \cdot ft}$

Analyze Moment Capacity at the Design Height:

Number of Bars Used in Design Height: $No := 8$

Reinforcing Bar Size: $bar := 4$

$$\phi(bar) := \begin{cases} \phi \leftarrow 0.500in & \text{if } bar = 4 \\ \phi \leftarrow 0.625in & \text{if } bar = 5 \\ \phi \leftarrow 0.750in & \text{if } bar = 6 \\ \phi \leftarrow 0.875in & \text{if } bar = 7 \end{cases}$$

$$A(bar) := \begin{cases} A \leftarrow 0.20in^2 & \text{if } bar = 4 \\ A \leftarrow 0.31 \cdot in^2 & \text{if } bar = 5 \\ A \leftarrow 0.44 \cdot in^2 & \text{if } bar = 6 \\ A \leftarrow 0.60 \cdot in^2 & \text{if } bar = 7 \end{cases}$$

Reinforcing Bar Diameter: $dia_{bar} := \phi(bar)$ — $dia_{bar} = 0.5 \cdot in$

Reinforcing Bar Area: $area_{bar} := A(bar)$ — $area_{bar} = 0.2 \cdot in^2$

Area of Steel: $A_s := No \cdot area_{bar}$ — $A_s = 1.6 \cdot in^2$

Depth to Centroid of Tension Reinforcement from Extreme Compression Fiber: $d := t_{ww} - cover - dia_{bar} - \frac{dia_{bar}}{2}$ — $d = 7.25 \cdot in$

Equivalent Stress Block Depth: $a := \frac{A_s \cdot f_y}{0.85 \cdot f_c \cdot h_{des}}$ — $a = 0.385 \cdot in$

Nominal Flexural Resistance: $M_n := A_s \cdot f_y \cdot \left(d - \frac{a}{2}\right)$ — $M_n = 56.46 \cdot kip \cdot ft$

Factored Flexural Resistance: $M_r := \phi_f \cdot M_n$ — $M_r = 50.82 \cdot kip \cdot ft$

Check Flexural Resistance: $Chk_M_r := if\left(M_a - M_p < M_r, \text{"OK"}, \text{"N.G"}\right)$ — $Chk_M_r = \text{"OK"}$

Summary

This analysis utilizes conserevitive soil surcharge pressures. It is known that passive pressure will be applied to the wing wall during construction before backfill loads are applied. Due to the uncertainty of the passive pressure loading, the passive pressure loads were neglected.

4. Foundation Design

4.1. Minimum Tip

KISINGER CAMPO & ASSOCIATES

LEE COUNTY
ABLE CANAL PATHWAY

FOUNDATION MINIMUM TIP DESIGN

MINIMUM TIP – INTERMEDIATE BENT

CALCULATION QUALITY CONTROL		
RESPONSIBLE PROFESSIONAL / ORIGINATOR	INITIALS: JHJ	DATE: 08/22
QUALITY REVIEWER	INITIALS: LM	DATE: 08/2022

SUMMARY OF SECTION

Minimum tip analysis is completed in accordance with SDG 3.5.9. The analysis employs FB-MultiPier v5.9.0 The FB-MultiPier analyses are organized as follows:

- *Intermediate Bent Minimum Tip Model with boring B-5 parameters*
- *Intermediate Bent Minimum Tip Model with boring B-6 parameters*– Includes only the soil data for the model with all other parameters equal to the B-5 model.

The FB MultiPier lateral stability feature is utilized for obtaining the minimum tip embedment.

LATERAL STABILITY ELEVATION

Boring	*Scour Elevation*	*Embedment*	*Additional Embedment**	*Minimum Tip EL.***
B-5 •	2.2 •	29.0 •	5.0 •	-32.0 •
B-6 •	2.2 •	25.5 •	5.0 •	-29.0 •

*SDG 3.5.8: Add 5 feet or 20% of the penetration, whichever is less.
**Round down to nearest whole foot elevation.

BHRS shows updated scour elevation of 2.3. Say OK and consider conservative.

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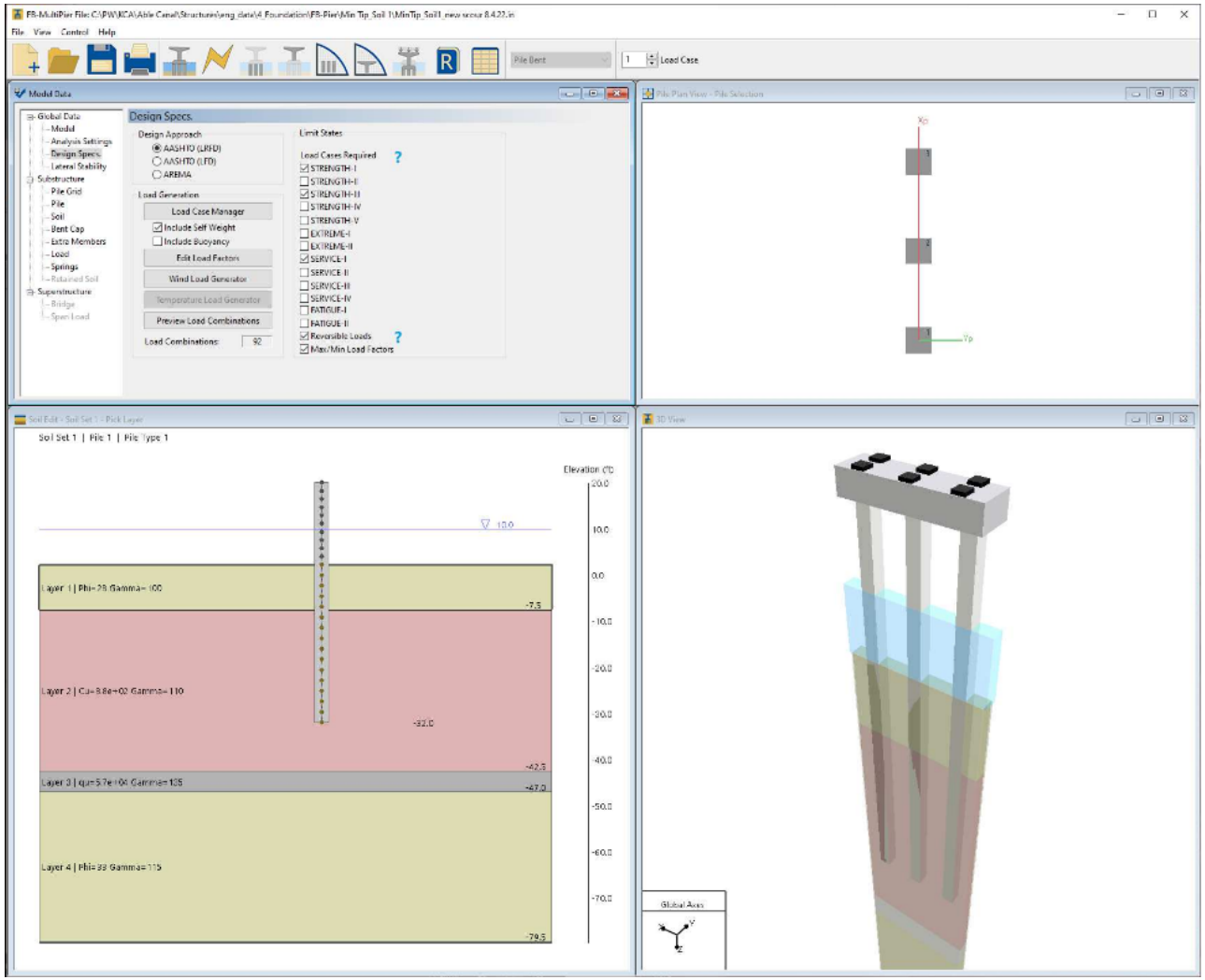
JOHN YARBROUGH LINEAR PARK

INTERMEDIATE BENT LOAD CASES



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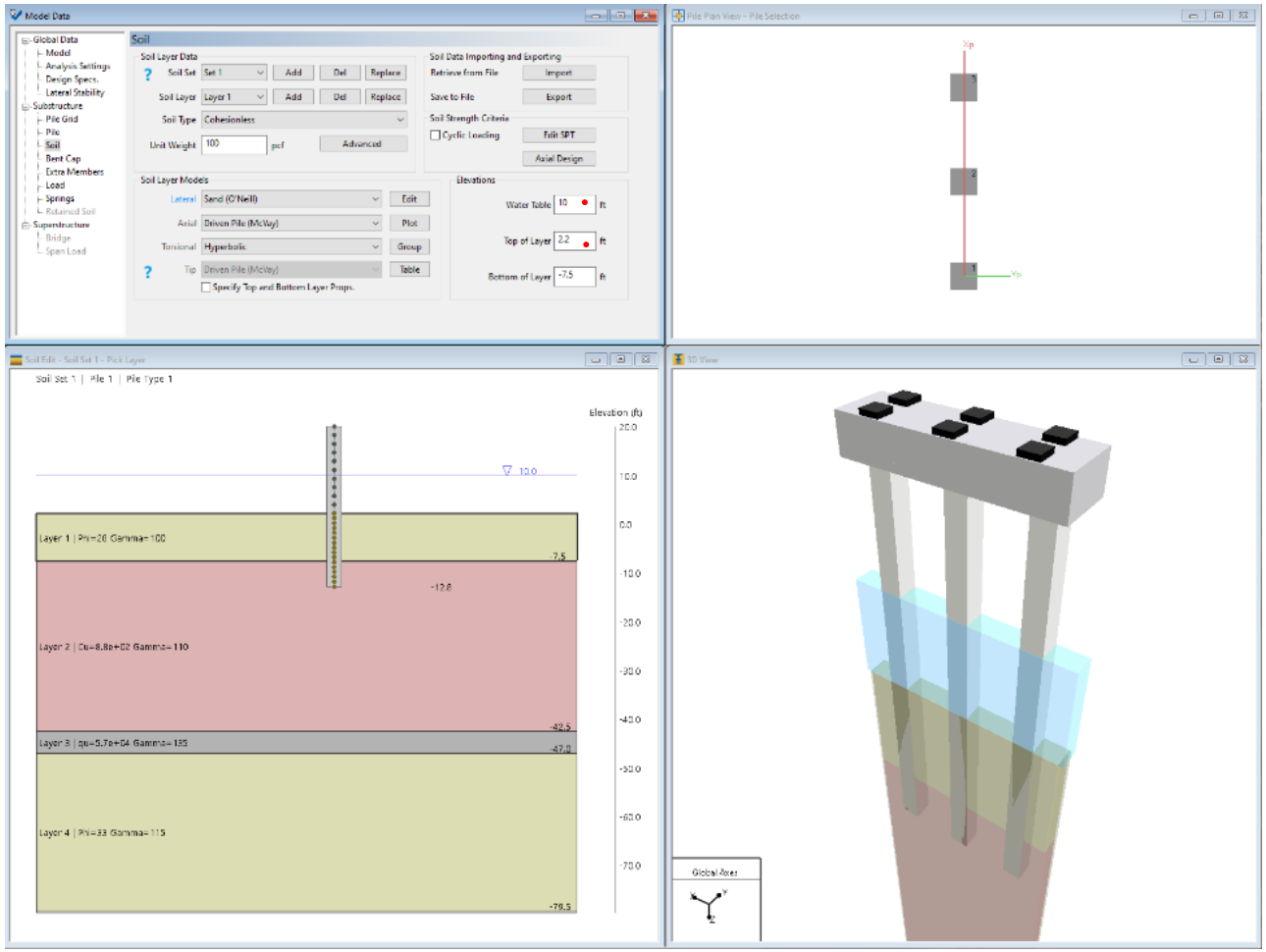
INTERMEDIATE BENT SOIL LAYERING – B-5



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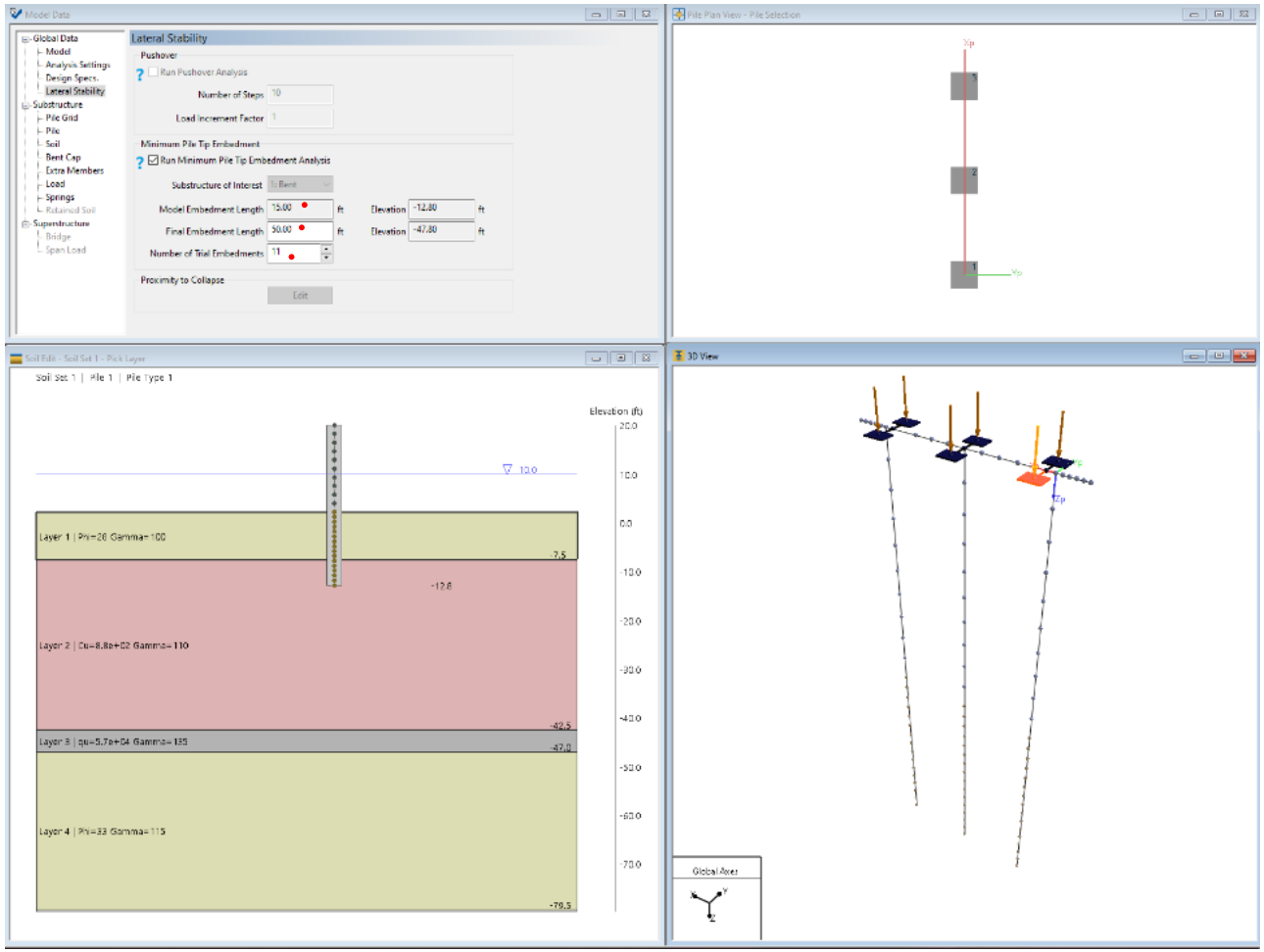
INTERMEDIATE BENT LATERAL STABILITY ANALYSIS – B-5



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INTERMEDIATE BENT MINIMUM TIP TABLE – B-5

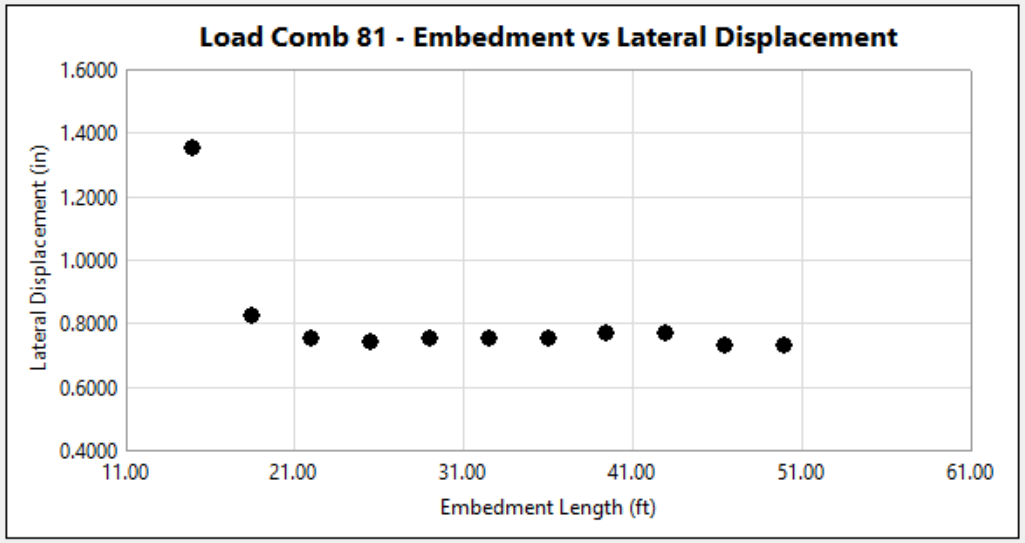
Minimum Pile Tip Embedment - Substructure 1

(●) Individual Load Combination
( ) Envelope (All Load Combs)

Load Comb 81

Plot Lateral Displacement

Max Envelope



Trial	Load Comb	Pile	Embedment (ft)	Displacement (in)
1	81	3	15.00	1.348300
2	81	1	18.50	0.823065
3	81	3	22.00	0.755681
4	81	3	25.50	0.740042
5	81	3	29.00	0.754428
6	81	3	32.50	0.752052
7	81	3	36.00	0.753718
8	81	3	39.50	0.767680

Notes

1. Results are presented for each load combination and each embedment.

2. The 'Lateral Displacement', 'Lateral X-Displacement', and 'Lateral Y-Displacement' plots display maximums of resultant Lateral Displacement, X-displacement, and Y-displacement across pile heads.

3. The 'Max Axial Force' plot displays maximum axial force for all pile elements. The 'Min Axial Force' plot displays minimum axial force for all pile elements.

Close

Excel

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FB-MultiPier

XML Report

XML File: MinTip_Soil1_new scour 8.4.22.XML

Project Information

Version	5.9.0
Client	Lee County
Project Name	Able Canal Pathway
Project Manager	
Description	Intermediate Bent - Min Tip - Soil 1 **Boring B-5**
Computed By	BAH
Project Date	11/04/21

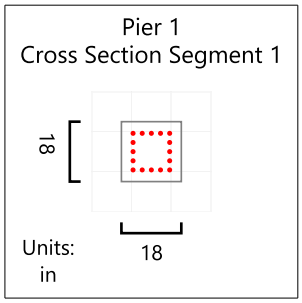
Control Information

Control Settings

Units	English	
Maximum Iterations	100	
Tolerance	1	(kip) •
Soil Behavior	0	
user supplied P-Y Multipliers must be given		
Tip Spring Option	0	
no linear tip springs on piles		
Pile Tip Stiffness	0	(kip/in)
Analysis	Static	
Design Code	AASHTO-LRFD	

Pile Geometry - Substructure 1

Segment Number 1



Dimensions

Width	18	(in) •
Depth	18	(in) •
Web thickness	0	(in)
Flange thickness	0	(in)
Orientation	0	
Void type	None	
Section void diameter	0	(in)
Section void width	0	(in)
Section void depth	0	(in)
Embedded h-pile width	0	(in)
Embedded h-pile depth	0	(in)
Embedded h-pile web thickness	0	(in)
Embedded h-pile flange thickness	0	(in)

Embedded h-pile orientation	0	
Shell thickness	0	(in)
Shear steel	spiral	
Length	393.6	(in)

Material Properties

Concrete compressive strength	6	(ksi)
Concrete elastic modulus	4557	(ksi)
Mild steel yield length	0	(ksi)
Mild steel elastic modulus	0	(ksi)
Prestressing steel yield length	270	(ksi)
Prestressing steel elastic modulus	28500	(ksi)
H-pile steel yield strength	0	(ksi)
H-pile steel elastic modulus	0	(ksi)
Steel casing yield strength	0	(ksi)
Steel casing elastic modulus	0	(ksi)

Gross Section Properties

Area	324	(in^2)
Elastic modulus	4557	(ksi)
Inertia 2	8748	(in^4)
Inertia 3	8748	(in^4)
Shear modulus	1899	(ksi)
Torsional inertia	14780	(in^4)
Unit weight	8.6806E-05	(k/in^3)

Steel Groups

Bar Group #1

Bars	5	
Layer diameter	0	(in)
Starting local "2" coordinate	5.5	(in)
Starting local "3" coordinate	5.5	(in)
Bar area	0.153	(in^2)
Prestress	140	(ksi)
Orientation	2	

Bar Group #2

Bars	5	
Layer diameter	0	(in)
Starting local "2" coordinate	5.5	(in)
Starting local "3" coordinate	-5.5	(in)
Bar area	0.153	(in^2)
Prestress	140	(ksi)
Orientation	2	

Bar Group #3

Bars	3	
Layer diameter	0	(in)
Starting local "2" coordinate	5.5	(in)
Starting local "3" coordinate	2.75	(in)
Bar area	0.153	(in^2)
Prestress	140	(ksi)
Orientation	3	

Bar Group #4

Bars	3	
Layer diameter	0	(in)

Starting local "2" coordinate	-5.5	(in)
Starting local "3" coordinate	2.75	(in)
Bar area	0.153	(in^2)
Prestress	140	(ksi)
Orientation	3	

Stress-Strain Curves

Concrete

Point	Stress (ksi)	Strain
1	-4.335	-0.0038
2	-5.1	-0.0022383
3	-4.996	-0.0019186
4	-4.684	-0.0015988
5	-4.163	-0.001279
6	-3.435	-0.00095928
7	-2.498	-0.00063952
8	-1.457	-0.00031976
9	0	0
10	0.5996	0.00013158
11	0.2998	0.0003
12	0	0.002
13	0	0.005

Prestressing steel

Point	Stress (ksi)	Strain
1	-267.8	-0.03
2	-267.8	-0.0256
3	-266.7	-0.019
4	-262.6	-0.0124
5	-257.7	-0.0102
6	-245.1	-0.0086
7	0	0
8	245.1	0.0086
9	257.7	0.0102
10	262.6	0.0124
11	266.7	0.019
12	267.8	0.0256
13	267.8	0.03

Pile Group - Substructure 1

Bearing Capacity	0
Axial Efficiency (Uplift)	1
Axial Efficiency (Skin)	1
Axial Efficiency (Tip)	1

X Grid Spacings (in)

Space 1	61
Space 2	61

Y Grid Spacings ()

Number of Piles	3

Pile to Cap Fixity

Pile #	Fixity
1	1

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2	1
3	1

Pile Sets

Set #1 Piles: 1 2 3

Soil Input Data

Lateral

Layer 1 Soil Set # 1 Model # 1

	Phi (deg)	Rk (lbs/in^3)	Gamma (pcf)
Top	28	6	100
Bottom	28	6	100

8. Say OK. Results unaffected.

Layer 2 Soil Set # 1 Model # 4

	Gamma (pcf)	Cu (psf)	E50
Top	110	875	0.015
Bottom	110	875	0.015

Layer 3 Soil Set # 1 Model # 9

	Qu (psf)
Top	57060
Bottom	57060

Layer 4 Soil Set # 1 Model # 1

	Phi (deg)	Rk (lbs/in^3)	Gamma (pcf)
Top	33	67	115
Bottom	33	67	115

Layer 5 Soil Set # 1 Model # 9

	Qu (psf)
Top	80000
Bottom	80000

Axial

Layer 1 Soil Set # 1 Model # 1

	G (ksi)	Poisson's ratio	Tau max (psf)	Elevation (ft)	Piezometric elevation (ft)
Top	1	0.25	76	2.2	10
Bottom	1	0.25	76	-7.5	10

Layer 2 Soil Set # 1 Model # 1

	G (ksi)	Poisson's ratio	Tau max (psf)	Elevation (ft)	Piezometric elevation (ft)
Top	3.5	0.45	720	-7.5	10
Bottom	3.5	0.45	720	-42.5	10

Layer 3 Soil Set # 1 Model # 1

	G (ksi)	Poisson's ratio	Tau max (psf)	Elevation (ft)	Piezometric elevation (ft)
Top	28.5	0.5	1140	-42.5	10
Bottom	28.5	0.5	1140	-47	10

Layer 4 Soil Set # 1 Model # 1

	G (ksi)	Poisson's ratio	Tau max (psf)	Elevation (ft)	Piezometric elevation (ft)
Top	12	0.3	912	-47	10

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Bottom	12	0.3	912	-79.5	10

Layer 5 Soil Set # 1 Model # 1

	G (ksi)	Poisson's ratio	Tau max (psf)	Elevation (ft)	Piezometric elevation (ft)
Top	50	0.5	1200	-79.5	10
Bottom	50	0.5	1200	-79.7	10

Torsion

Layer 1 Soil Set # 1 Model # 1

	G (ksi)	Tau max (psf)
Top	1	76
Bottom	1	76

G = 1.5 and Tau.max = 114.
Say OK. Conservative as is.

Layer 2 Soil Set # 1 Model # 1

	G (ksi)	Tau max (psf)
Top	3.5	720
Bottom	3.5	720

Layer 3 Soil Set # 1 Model # 1

	G (ksi)	Tau max (psf)
Top	28.5	1140
Bottom	28.5	1140

Layer 4 Soil Set # 1 Model # 1

	G (ksi)	Tau max (psf)
Top	12	912
Bottom	12	912

Layer 5 Soil Set # 1 Model # 1

	G (ksi)	Tau max (psf)
Top	50	1200
Bottom	50	1200

Tip

	Model #	G (ksi)	Poisson's ratio	Ultimate bearing force (kips)
Soil Set #1	1	3.5	0.45	9999

Pile Element Segments

Pile #1

Element #	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
Segment #	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1

Pile #2

Element #	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
Segment #	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1

Pile #3

Element #	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
Segment #	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1

Pile Coordinates

Pile #1

Point	X (in)	Y (in)	Z (in)
1	0	0	-240
2	0	0	-218.6
3	0	0	-197.3
4	0	0	-175.9
5	0	0	-154.6
6	0	0	-133.2
7	0	0	-111.8
8	0	0	-90.48
9	0	0	-69.12
10	0	0	-47.76
11	0	0	-26.4
12	0	0	-14.4
13	0	0	-2.4
14	0	0	9.6
15	0	0	21.6
16	0	0	33.6
17	0	0	45.6
18	0	0	57.6
19	0	0	69.6
20	0	0	81.6
21	0	0	93.6
22	0	0	105.6
23	0	0	117.6
24	0	0	129.6
25	0	0	141.6
26	0	0	153.6

Pile #2

Point	X (in)	Y (in)	Z (in)
1	61	0	-240
2	61	0	-218.6
3	61	0	-197.3
4	61	0	-175.9
5	61	0	-154.6
6	61	0	-133.2
7	61	0	-111.8
8	61	0	-90.48
9	61	0	-69.12
10	61	0	-47.76
11	61	0	-26.4
12	61	0	-14.4
13	61	0	-2.4
14	61	0	9.6
15	61	0	21.6
16	61	0	33.6
17	61	0	45.6
18	61	0	57.6
19	61	0	69.6
20	61	0	81.6
21	61	0	93.6
22	61	0	105.6
23	61	0	117.6
24	61	0	129.6
25	61	0	141.6
26	61	0	153.6

Pile #3

Point	X (in)	Y (in)	Z (in)
1	122	0	-240
2	122	0	-218.6
3	122	0	-197.3

4	122	0	-175.9
5	122	0	-154.6
6	122	0	-133.2
7	122	0	-111.8
8	122	0	-90.48
9	122	0	-69.12
10	122	0	-47.76
11	122	0	-26.4
12	122	0	-14.4
13	122	0	-2.4
14	122	0	9.6
15	122	0	21.6
16	122	0	33.6
17	122	0	45.6
18	122	0	57.6
19	122	0	69.6
20	122	0	81.6
21	122	0	93.6
22	122	0	105.6
23	122	0	117.6
24	122	0	129.6
25	122	0	141.6
26	122	0	153.6

Pier Geometry - Substructure 1

Modeling

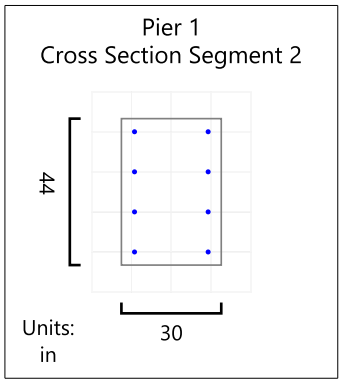
Structure type	Pile bent	
Number of columns	3	
Number of elements per span	5	
Cantilever length	1.75	(ft)
Number of elements per cantilever	5	
Bearing pads		
	Pad # 1	Located @ Node 9
	Pad # 2	Located @ Node 2
	Pad # 3	Located @ Node 16
Column taper	no	
Cantilever taper	no	
Beam taper	no	

Nodal Coordinates

		Boundary Conditions						Coordinates		
Node #	Location	X	Y	Z	Xx	Yy	Zz	X (in)	Y (in)	Z (in)
4	Pier	A	A	A	A	A	A	-21	0	-240
5	Pier	A	A	A	A	A	A	-16.8	0	-240
6	Pier	A	A	A	A	A	A	-12.6	0	-240
7	Pier	A	A	A	A	A	A	-8.4	0	-240
8	Pier	A	A	A	A	A	A	-4.2	0	-240
9	Pier	A	A	A	A	A	A	6	0	-240
10	Pier	A	A	A	A	A	A	24.4	0	-240
11	Pier	A	A	A	A	A	A	36.6	0	-240
12	Pier	A	A	A	A	A	A	48.8	0	-240
13	Pier	A	A	A	A	A	A	73.2	0	-240
14	Pier	A	A	A	A	A	A	85.4	0	-240
15	Pier	A	A	A	A	A	A	97.6	0	-240
16	Pier	A	A	A	A	A	A	116	0	-240
17	Pier	A	A	A	A	A	A	126.2	0	-240
18	Pier	A	A	A	A	A	A	130.4	0	-240
19	Pier	A	A	A	A	A	A	134.6	0	-240
20	Pier	A	A	A	A	A	A	138.8	0	-240
21	Pier	A	A	A	A	A	A	143	0	-240

Cross Sections

Segment Number 2



Dimensions

Width	44	(in)
Depth	30	(in)
Web thickness	0	(in)
Flange thickness	0	(in)
Orientation	0	
Void type	None	
Section void diameter	0	(in)
Section void width	0	(in)
Section void depth	0	(in)
Embedded h-pile width	0	(in)
Embedded h-pile depth	0	(in)
Embedded h-pile web thickness	0	(in)
Embedded h-pile flange thickness	0	(in)
Embedded h-pile orientation	0	
Shell thickness	0	(in)
Shear steel	tie	

Material Properties

Concrete compressive strength	5.5	(ksi)
Concrete elastic modulus	4428	(ksi)
Mild steel yield length	60	(ksi)
Mild steel elastic modulus	29000	(ksi)
Prestressing steel yield length	0	(ksi)
Prestressing steel elastic modulus	0	(ksi)
H-pile steel yield strength	0	(ksi)
H-pile steel elastic modulus	0	(ksi)
Steel casing yield strength	0	(ksi)
Steel casing elastic modulus	0	(ksi)

Gross Section Properties

Area	1320	(in^2)
Elastic modulus	4428	(ksi)
Inertia 2	213000	(in^4)
Inertia 3	99000	(in^4)
Shear modulus	1845	(ksi)
Torsional inertia	229400	(in^4)
Unit weight	8.6806E-05	(k/in^3)

Steel Groups

Bar Group #1

Bars	4

Layer diameter	0	(in)
Starting local "2" coordinate	11.06	(in)
Starting local "3" coordinate	18.06	(in)
Bar area	0.31	(in^2)
Prestress	0	(ksi)
Orientation	3	

Bar Group #2

Bars	4	
Layer diameter	0	(in)
Starting local "2" coordinate	-11.06	(in)
Starting local "3" coordinate	18.06	(in)
Bar area	0.31	(in^2)
Prestress	0	(ksi)
Orientation	3	

Stress-Strain Curves

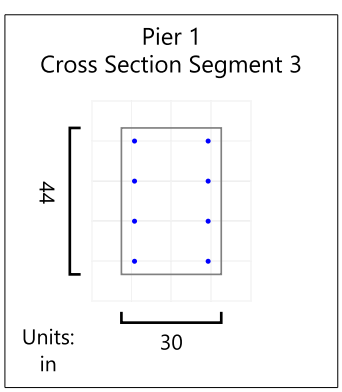
Concrete

Point	Stress (ksi)	Strain
1	-3.974	-0.0038
2	-4.675	-0.0021114
3	-4.58	-0.0018098
4	-4.293	-0.0015082
5	-3.816	-0.0012065
6	-3.148	-0.00090489
7	-2.29	-0.00060326
8	-1.336	-0.00030163
9	0	0
10	0.5827	0.00013158
11	0.2913	0.0003
12	0	0.002
13	0	0.005

Mild steel

Point	Stress (ksi)	Strain
1	-60	-0.09999
2	-60	-0.002069
3	60	0.002069
4	60	0.09999

Segment Number 3



Dimensions

Width	44	(in)
Depth	30	(in)

Web thickness	0	(in)
Flange thickness	0	(in)
Orientation	0	
Void type	None	
Section void diameter	0	(in)
Section void width	0	(in)
Section void depth	0	(in)
Embedded h-pile width	0	(in)
Embedded h-pile depth	0	(in)
Embedded h-pile web thickness	0	(in)
Embedded h-pile flange thickness	0	(in)
Embedded h-pile orientation	0	
Shell thickness	0	(in)
Shear steel	tie	

Material Properties

Concrete compressive strength	5.5	(ksi) •
Concrete elastic modulus	4428	(ksi) •
Mild steel yield length	60	(ksi) •
Mild steel elastic modulus	29000	(ksi) •
Prestressing steel yield length	0	(ksi)
Prestressing steel elastic modulus	0	(ksi)
H-pile steel yield strength	0	(ksi)
H-pile steel elastic modulus	0	(ksi)
Steel casing yield strength	0	(ksi)
Steel casing elastic modulus	0	(ksi)

Gross Section Properties

Area	1320	(in^2)
Elastic modulus	4428	(ksi)
Inertia 2	213000	(in^4)
Inertia 3	99000	(in^4)
Shear modulus	1845	(ksi)
Torsional inertia	229400	(in^4)
Unit weight	8.6806E-05	(k/in^3)

Steel Groups

Bar Group #1

Bars	4 •	
Layer diameter	0	(in)
Starting local “2” coordinate	11.06	(in)
Starting local “3” coordinate	18.06	(in)
Bar area	0.31	(in^2)
Prestress	0	(ksi)
Orientation	3	

Bar Group #2

Bars	4 •	
Layer diameter	0	(in)
Starting local “2” coordinate	-11.06	(in)
Starting local “3” coordinate	18.06	(in)
Bar area	0.31	(in^2)
Prestress	0	(ksi)
Orientation	3	

Stress-Strain Curves

Concrete

Point	Stress (ksi)	Strain
1	-3.974	-0.0038
2	-4.675	-0.0021114
3	-4.58	-0.0018098
4	-4.293	-0.0015082
5	-3.816	-0.0012065
6	-3.148	-0.00090489
7	-2.29	-0.00060326
8	-1.336	-0.00030163
9	0	0
10	0.5827	0.00013158
11	0.2913	0.0003
12	0	0.002
13	0	0.005

Mild steel

Point	Stress (ksi)	Strain
1	-60	-0.09999
2	-60	-0.002069
3	60	0.002069
4	60	0.09999

Pier Member Connectivity

Beam Element #

Load Case Data - Substructure 1

Load Values

Load Case	AASHTO Load Type	Node	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1	DC	1	0	0	0	0	0	0
17	WS1 •	2	0.92 •	0	0	0	0	0
17	WS1	26	0.2642	0	0	0	0	0
17	WS1	25	0.2642	0	0	0	0	0
17	WS1	24	0.2642	0	0	0	0	0
17	WS1	22	0.1321	0	0	0	0	0
17	WS1	52	0.1321	0	0	0	0	0
17	WS1	51	0.2642	0	0	0	0	0
17	WS1	50	0.2642	0	0	0	0	0
17	WS1	49	0.2642	0	0	0	0	0
17	WS1	47	0.1321	0	0	0	0	0
17	WS1	77	0.1321	0	0	0	0	0
17	WS1	76	0.2642	0	0	0	0	0
17	WS1	75	0.2642	0	0	0	0	0
17	WS1	74	0.2642	0	0	0	0	0
17	WS1	72	0.1321	0	0	0	0	0
17	WS1	23	0.2642	0	0	0	0	0
17	WS1	27	0.1321	0	0	0	0	0
17	WS1	48	0.2642	0	0	0	0	0
17	WS1 •	73	0.2642	0	0	0	0	0
18	WS2 •	2	2.18 •	0.0915	0	0	0	0
18	WS2	4	0	0.01575	0	0	0	0
18	WS2	5	0	0.0315	0	0	0	0
18	WS2	6	0	0.0315	0	0	0	0
18	WS2	7	0	0.0315	0	0	0	0
18	WS2	8	0	0.0315	0	0	0	0
18	WS2	1	0	0.1298	0	0	0	0
18	WS2	10	0	0.1148	0	0	0	0

18	WS2	11	0	0.0915	0	0	0	0
18	WS2	13	0	0.0915	0	0	0	0
18	WS2	14	0	0.0915	0	0	0	0
18	WS2	15	0	0.1147	0	0	0	0
18	WS2	3	0	0.1298	0	0	0	0
18	WS2	17	0	0.0315	0	0	0	0
18	WS2	18	0	0.0315	0	0	0	0
18	WS2	19	0	0.0315	0	0	0	0
18	WS2	20	0	0.0315	0	0	0	0
18	WS2	21	0	0.01575	0	0	0	0
18	WS2	12	0	0.0915	0	0	0	0
18	WS2	22	0.1566	0.09013	0	0	0	0
18	WS2	23	0.3133	0.1803	0	0	0	0
18	WS2	24	0.3133	0.1803	0	0	0	0
18	WS2	25	0.3133	0.1803	0	0	0	0
18	WS2	26	0.3133	0.1803	0	0	0	0
18	WS2	27	0.1566	0.09013	0	0	0	0
18	WS2	47	0.1566	0.09013	0	0	0	0
18	WS2	48	0.3133	0.1803	0	0	0	0
18	WS2	49	0.3133	0.1803	0	0	0	0
18	WS2	50	0.3133	0.1803	0	0	0	0
18	WS2	51	0.3133	0.1803	0	0	0	0
18	WS2	52	0.1566	0.09013	0	0	0	0
18	WS2	72	0.1566	0.09013	0	0	0	0
18	WS2	73	0.3133	0.1803	0	0	0	0
18	WS2	74	0.3133	0.1803	0	0	0	0
18	WS2	75	0.3133	0.1803	0	0	0	0
18	WS2	76	0.3133	0.1803	0	0	0	0
18	WS2	77	0.1566	0.09013	0	0	0	0
19	WS3	2	1.72	0.2237	0	0	0	0
19	WS3	4	0	0.0385	0	0	0	0
19	WS3	5	0	0.077	0	0	0	0
19	WS3	6	0	0.077	0	0	0	0
19	WS3	7	0	0.077	0	0	0	0
19	WS3	8	0	0.077	0	0	0	0
19	WS3	1	0	0.3172	0	0	0	0
19	WS3	10	0	0.2805	0	0	0	0
19	WS3	11	0	0.2237	0	0	0	0
19	WS3	13	0	0.2237	0	0	0	0
19	WS3	14	0	0.2237	0	0	0	0
19	WS3	15	0	0.2805	0	0	0	0
19	WS3	3	0	0.3172	0	0	0	0
19	WS3	17	0	0.077	0	0	0	0
19	WS3	18	0	0.077	0	0	0	0
19	WS3	19	0	0.077	0	0	0	0
19	WS3	20	0	0.077	0	0	0	0
19	WS3	21	0	0.0385	0	0	0	0
19	WS3	22	0.09013	0.1566	0	0	0	0
19	WS3	24	0.1803	0.3133	0	0	0	0
19	WS3	25	0.1803	0.3133	0	0	0	0
19	WS3	26	0.1803	0.3133	0	0	0	0
19	WS3	27	0.09013	0.1566	0	0	0	0
19	WS3	47	0.09013	0.1566	0	0	0	0
19	WS3	49	0.1803	0.3133	0	0	0	0
19	WS3	50	0.1803	0.3133	0	0	0	0
19	WS3	51	0.1803	0.3133	0	0	0	0
19	WS3	52	0.09013	0.1566	0	0	0	0
19	WS3	72	0.09013	0.1566	0	0	0	0
19	WS3	74	0.1803	0.3133	0	0	0	0
19	WS3	75	0.1803	0.3133	0	0	0	0
19	WS3	76	0.1803	0.3133	0	0	0	0
19	WS3	77	0.09013	0.1566	0	0	0	0

19	WS3	12	0	0.2237	0	0	0	0
19	WS3	23	0.1803	0.3133	0	0	0	0
19	WS3	48	0.1803	0.3133	0	0	0	0
19	WS3	73	0.1803	0.3133	0	0	0	0
20	WS4	2	0.18	0	0	0	0	0
20	WS4	22	0.02625	0	0	0	0	0
20	WS4	24	0.0525	0	0	0	0	0
20	WS4	25	0.0525	0	0	0	0	0
20	WS4	26	0.0525	0	0	0	0	0
20	WS4	27	0.02625	0	0	0	0	0
20	WS4	47	0.02625	0	0	0	0	0
20	WS4	49	0.0525	0	0	0	0	0
20	WS4	50	0.0525	0	0	0	0	0
20	WS4	51	0.0525	0	0	0	0	0
20	WS4	52	0.02625	0	0	0	0	0
20	WS4	72	0.02625	0	0	0	0	0
20	WS4	74	0.0525	0	0	0	0	0
20	WS4	75	0.0525	0	0	0	0	0
20	WS4	76	0.0525	0	0	0	0	0
20	WS4	77	0.02625	0	0	0	0	0
20	WS4	23	0.0525	0	0	0	0	0
20	WS4	48	0.0525	0	0	0	0	0
20	WS4	73	0.0525	0	0	0	0	0
21	WS5	2	0.43	0.02033	0	0	0	0
21	WS5	6	0	0.007	0	0	0	0
21	WS5	7	0	0.007	0	0	0	0
21	WS5	8	0	0.007	0	0	0	0
21	WS5	1	0	0.02883	0	0	0	0
21	WS5	10	0	0.0255	0	0	0	0
21	WS5	11	0	0.02033	0	0	0	0
21	WS5	13	0	0.02033	0	0	0	0
21	WS5	14	0	0.02033	0	0	0	0
21	WS5	15	0	0.0255	0	0	0	0
21	WS5	3	0	0.02883	0	0	0	0
21	WS5	17	0	0.007	0	0	0	0
21	WS5	18	0	0.007	0	0	0	0
21	WS5	19	0	0.007	0	0	0	0
21	WS5	20	0	0.007	0	0	0	0
21	WS5	22	0.03063	0.0175	0	0	0	0
21	WS5	24	0.06125	0.035	0	0	0	0
21	WS5	25	0.06125	0.035	0	0	0	0
21	WS5	26	0.06125	0.035	0	0	0	0
21	WS5	27	0.03063	0.0175	0	0	0	0
21	WS5	47	0.03063	0.0175	0	0	0	0
21	WS5	49	0.06125	0.035	0	0	0	0
21	WS5	50	0.06125	0.035	0	0	0	0
21	WS5	51	0.06125	0.035	0	0	0	0
21	WS5	52	0.03063	0.0175	0	0	0	0
21	WS5	72	0.03063	0.0175	0	0	0	0
21	WS5	74	0.06125	0.035	0	0	0	0
21	WS5	75	0.06125	0.035	0	0	0	0
21	WS5	76	0.06125	0.035	0	0	0	0
21	WS5	77	0.03063	0.0175	0	0	0	0
21	WS5	4	0	0.0035	0	0	0	0
21	WS5	5	0	0.007	0	0	0	0
21	WS5	12	0	0.02033	0	0	0	0
21	WS5	21	0	0.0035	0	0	0	0
21	WS5	23	0.06125	0.035	0	0	0	0
21	WS5	48	0.06125	0.035	0	0	0	0
21	WS5	73	0.06125	0.035	0	0	0	0
22	WS6	2	0.34	0.04067	0	0	0	0
22	WS6	4	0	0.007	0	0	0	0

22	WS6	5	0	0.014	0	0	0	0
22	WS6	6	0	0.014	0	0	0	0
22	WS6	7	0	0.014	0	0	0	0
22	WS6	8	0	0.014	0	0	0	0
22	WS6	1	0	0.05767	0	0	0	0
22	WS6	10	0	0.051	0	0	0	0
22	WS6	11	0	0.04067	0	0	0	0
22	WS6	13	0	0.04067	0	0	0	0
22	WS6	14	0	0.04067	0	0	0	0
22	WS6	15	0	0.051	0	0	0	0
22	WS6	3	0	0.05767	0	0	0	0
22	WS6	17	0	0.014	0	0	0	0
22	WS6	18	0	0.014	0	0	0	0
22	WS6	19	0	0.014	0	0	0	0
22	WS6	20	0	0.014	0	0	0	0
22	WS6	21	0	0.007	0	0	0	0
22	WS6	22	0.0175	0.03063	0	0	0	0
22	WS6	24	0.035	0.06125	0	0	0	0
22	WS6	25	0.035	0.06125	0	0	0	0
22	WS6	26	0.035	0.06125	0	0	0	0
22	WS6	27	0.0175	0.03063	0	0	0	0
22	WS6	47	0.0175	0.03063	0	0	0	0
22	WS6	49	0.035	0.06125	0	0	0	0
22	WS6	50	0.035	0.06125	0	0	0	0
22	WS6	51	0.035	0.06125	0	0	0	0
22	WS6	52	0.0175	0.03063	0	0	0	0
22	WS6	72	0.0175	0.03063	0	0	0	0
22	WS6	74	0.035	0.06125	0	0	0	0
22	WS6	75	0.035	0.06125	0	0	0	0
22	WS6	76	0.035	0.06125	0	0	0	0
22	WS6	77	0.0175	0.03063	0	0	0	0
22	WS6	12	0	0.04067	0	0	0	0
22	WS6	23	0.035	0.06125	0	0	0	0
22	WS6	48	0.035	0.06125	0	0	0	0
22	WS6	73	0.035	0.06125	0	0	0	0
25	TU	1	0	0	0	0	0	0

Load Case #1

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	DC	0	0	33.4	0	0	0
1R	DC	0	0	33.4	0	0	0
2L	DC	0	0	33.4	0	0	0
2R	DC	0	0	33.4	0	0	0
3L	DC	0	0	33.4	0	0	0
3R	DC	0	0	33.4	0	0	0

Load Case #2

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	LL1	0	0	6.98	0	0	0
1R	LL1	0	0	1.51	0	0	0
2L	LL1	0	0	5.53	0	0	0
2R	LL1	0	0	1.2	0	0	0
3L	LL1	0	0	3.49	0	0	0
3R	LL1	0	0	0.76	0	0	0

Load Case #3

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	LL2	0	0	3.49	0	0	0
1R	LL2	0	0	0.76	0	0	0
2L	LL2	0	0	5.53	0	0	0
2R	LL2	0	0	1.2	0	0	0
3L	LL2	0	0	6.98	0	0	0
3R	LL2	0	0	1.51	0	0	0

Load Case #4

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	LL3	0	0	9.54	0	0	0
1R	LL3	0	0	9.54	0	0	0
2L	LL3	0	0	9.54	0	0	0
2R	LL3	0	0	9.54	0	0	0
3L	LL3	0	0	9.54	0	0	0
3R	LL3	0	0	9.54	0	0	0

Load Case #5

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	LL4	0	0	9.54	0	0	0
1R	LL4	0	0	0	0	0	0
2L	LL4	0	0	9.54	0	0	0
2R	LL4	0	0	0	0	0	0
3L	LL4	0	0	9.54	0	0	0
3R	LL4	0	0	0	0	0	0

Load Case #6

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	LL5	0	0	0	0	0	0
1R	LL5	0	0	9.54	0	0	0
2L	LL5	0	0	0	0	0	0
2R	LL5	0	0	9.54	0	0	0
3L	LL5	0	0	0	0	0	0
3R	LL5	0	0	9.54	0	0	0

Load Case #7

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	IM1	0	0	0	0	0	0
1R	IM1	0	0	0	0	0	0
2L	IM1	0	0	0	0	0	0
2R	IM1	0	0	0	0	0	0
3L	IM1	0	0	0	0	0	0
3R	IM1	0	0	0	0	0	0

Load Case #8

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	IM2	0	0	0	0	0	0
1R	IM2	0	0	0	0	0	0
2L	IM2	0	0	0	0	0	0
2R	IM2	0	0	0	0	0	0
3L	IM2	0	0	0	0	0	0
3R	IM2	0	0	0	0	0	0

Load Case #9

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	IM3	0	0	0	0	0	0
1R	IM3	0	0	0	0	0	0
2L	IM3	0	0	0	0	0	0
2R	IM3	0	0	0	0	0	0
3L	IM3	0	0	0	0	0	0
3R	IM3	0	0	0	0	0	0

Load Case #10

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	IM4	0	0	0	0	0	0
1R	IM4	0	0	0	0	0	0
2L	IM4	0	0	0	0	0	0
2R	IM4	0	0	0	0	0	0
3L	IM4	0	0	0	0	0	0
3R	IM4	0	0	0	0	0	0

Load Case #11

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	IM5	0	0	0	0	0	0
1R	IM5	0	0	0	0	0	0
2L	IM5	0	0	0	0	0	0
2R	IM5	0	0	0	0	0	0
3L	IM5	0	0	0	0	0	0
3R	IM5	0	0	0	0	0	0

Load Case #12

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	BR1	0	0	0	0	0	0
1R	BR1	0	0	0	0	0	0
2L	BR1	0	0	0	0	0	0
2R	BR1	0	0	0	0	0	0
3L	BR1	0	0	0	0	0	0
3R	BR1	0	0	0	0	0	0

Load Case #13

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	BR2	0	0	0	0	0	0
1R	BR2	0	0	0	0	0	0
2L	BR2	0	0	0	0	0	0
2R	BR2	0	0	0	0	0	0
3L	BR2	0	0	0	0	0	0
3R	BR2	0	0	0	0	0	0

Load Case #14

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	BR3	0	0	0	0	0	0
1R	BR3	0	0	0	0	0	0
2L	BR3	0	0	0	0	0	0
2R	BR3	0	0	0	0	0	0
3L	BR3	0	0	0	0	0	0
3R	BR3	0	0	0	0	0	0

Load Case #15

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	BR4	0	0	0	0	0	0
1R	BR4	0	0	0	0	0	0
2L	BR4	0	0	0	0	0	0
2R	BR4	0	0	0	0	0	0
3L	BR4	0	0	0	0	0	0
3R	BR4	0	0	0	0	0	0

Load Case #16

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	BR5	0	0	0	0	0	0
1R	BR5	0	0	0	0	0	0
2L	BR5	0	0	0	0	0	0
2R	BR5	0	0	0	0	0	0
3L	BR5	0	0	0	0	0	0
3R	BR5	0	0	0	0	0	0

Load Case #17

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	WS1	3.55 •	0	-2.86 •	0	0	0
1R	WS1	3.55	0	-2.86 •	0	0	0
2L	WS1	3.55	0	0	0	0	0
2R	WS1	3.55	0	0	0	0	0
3L	WS1	3.55	0	2.86 •	0	0	0
3R	WS1	3.55 •	0	2.86 •	0	0	0

Load Case #18

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	WS2	2.91	0.85	-2.34	0	0	0
1R	WS2	2.91	0.85	-2.34	0	0	0
2L	WS2	2.91	0.85	0	0	0	0
2R	WS2	2.91	0.85	0	0	0	0
3L	WS2	2.91	0.85	2.34	0	0	0
3R	WS2	2.91	0.85	2.34	0	0	0

Load Case #19

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	WS3	1.21	1.35	-0.97	0	0	0
1R	WS3	1.21	1.35	-0.97	0	0	0
2L	WS3	1.21	1.35	0	0	0	0
2R	WS3	1.21	1.35	0	0	0	0
3L	WS3	1.21	1.35	0.97	0	0	0
3R	WS3	1.21	1.35	0.97	0	0	0

Load Case #20

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	WS4	0.71	0	-0.57	0	0	0
1R	WS4	0.71	0	-0.57	0	0	0
2L	WS4	0.71	0	0	0	0	0
2R	WS4	0.71	0	0	0	0	0
3L	WS4	0.71	0	0.57	0	0	0
3R	WS4	0.71	0	0.57	0	0	0

Load Case #21

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	WS5	0.58	0.17	-0.47	0	0	0
1R	WS5	0.58	0.17	-0.47	0	0	0
2L	WS5	0.58	0.17	0	0	0	0
2R	WS5	0.58	0.17	0	0	0	0
3L	WS5	0.58	0.17	0.47	0	0	0
3R	WS5	0.58	0.17	0.47	0	0	0

Load Case #22

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	WS6	0.24	0.27	-0.19	0	0	0
1R	WS6	0.24	0.27	-0.19	0	0	0
2L	WS6	0.24	0.27	0	0	0	0
2R	WS6	0.24	0.27	0	0	0	0
3L	WS6	0.24	0.27	0.19	0	0	0
3R	WS6	0.24	0.27	0.19	0	0	0

Load Case #23

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	VP	0	0	-6.27 •	0	0	0
1R	VP	0	0	-6.27 •	0	0	0
2L	VP	0	0	0	0	0	0
2R	VP	0	0	0	0	0	0
3L	VP	0	0	-0.93 •	0	0	0
3R	VP	0	0	-0.93 •	0	0	0

Load Case #24

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	VR	0	0	-0.93 •	0	0	0
1R	VR	0	0	-0.93 •	0	0	0
2L	VR	0	0	0	0	0	0
2R	VR	0	0	0	0	0	0
3L	VR	0	0	-6.27 •	0	0	0
3R	VR	0	0	-6.27 •	0	0	0

Output Summary - Substructure 1

Max/Min Pile Strains	Value		Load Case	Combination	Type	Segment	Pile
max strain in concrete	0.0015412	(in/in)	47	22	1	1	3
corresponding stress	0.08091	(ksi)	47	22	1	1	3
min strain in concrete	-0.0019725	(in/in)	47	22	1	1	3
corresponding stress	-5.013	(ksi)	47	22	1	1	3
max strain in reinforcement (prestressed)	0.0060377	(in/in)	47	22	1	1	3
corresponding stress	172.1	(ksi)	47	22	1	1	3
min strain in steel (prestressed)	0.0038198	(in/in)	47	22	1	1	3
corresponding stress	108.9	(ksi)	47	22	1	1	3

Max/Min Pier Column and Pier Cap Strains	Value		Load Case	Combination	Section	Element
max strain in concrete	0.00010547	(in/in)	50	25	3	6
corresponding stress	0.467	(ksi)	50	25	3	6
min strain in concrete	-0.00010763	(in/in)	50	25	3	6
corresponding stress	-0.4766	(ksi)	50	25	3	6
max strain in steel (mild)	8.0597E-05	(in/in)	50	25	3	6
corresponding stress	2.337	(ksi)	50	25	3	6
min strain in steel (mild)	-8.2762E-05	(in/in)	50	25	3	6
corresponding stress	-2.4	(ksi)	50	25	3	6
max strain in concrete	3.8913E-05	(in/in)	50	25	4	13
corresponding stress	0.1723	(ksi)	50	25	4	13
min strain in concrete	-3.8565E-05	(in/in)	50	25	4	13
corresponding stress	-0.1708	(ksi)	50	25	4	13
max strain in steel (mild)	2.9879E-05	(in/in)	50	25	4	13
corresponding stress	0.8665	(ksi)	50	25	4	13
min strain in steel (mild)	-2.9532E-05	(in/in)	50	25	4	13
corresponding stress	-0.8564	(ksi)	50	25	4	13

Max/Min Pile Internal Forces	Value		Load Case	Combination	Pile
max shear in 2 direction	10.22	(kip)	49	24	1
min shear in 2 direction	-10.22	(kip)	46	21	3

max shear in 3 direction	40.48	(kip)	57	32	1
min shear in 3 direction	-40.45	(kip)	54	29	3
max moment about 2 axis	158.3	(kip-ft)	54	29	1
min moment about 2 axis	-158.3	(kip-ft)	57	32	3
max moment about 3 axis	275.5	(kip-ft)	47	22	3
min moment about 3 axis	-275.6	(kip-ft)	50	25	1
max axial force	9.387	(kip)	55	30	3
min axial force	-175	(kip)	50	25	1
max torsional force	23.09	(kip-ft)	47	22	2
min torsional force	-23.09	(kip-ft)	47	22	2
max demand/capacity ratio	1.202		47	22	3

Max/Min Soil Reaction Forces	Value		Load Case	Combination	Pile
max axial soil force	155.9	(kip)	50	25	1
min axial soil force	-0.0069643	(kip)	55	30	3
max lateral in X direction	3.388	(kip)	49	24	1
min lateral in X direction	-3.388	(kip)	46	21	3
max lateral in Y direction	10.44	(kip)	54	29	2
min lateral in Y direction	-10.44	(kip)	57	32	2
max torsional soil force	0.3673	(kip-ft)	53	28	1

Max/Min Pile Head Displacements	Value		Load Case	Combination	Pile
max displacement in axial	0.5948	(in)	50	25	1
min displacement in axial	-0.00033338	(in)	55	30	3
max displacement in x	6.658	(in)	47	22	2
min displacement in x	-6.664	(in)	50	25	2
max displacement in y	55.26	(in)	57	32	1
min displacement in y	-55.21	(in)	54	29	3

Max/Min Pier Cap Internal Forces	Value		Load Case	Combination
max axial force	6.845	(kip)	50	25
min axial force	-8.742	(kip)	52	27
max shear in 2 direction	164.2	(kip)	50	25
min shear in 2 direction	-164.2	(kip)	47	22
max shear in 3 direction	8.612	(kip)	50	25
min shear in 3 direction	-2.781	(kip)	53	28
max torque	42.95	(kip-ft)	57	32
min torque	-42.95	(kip-ft)	57	32
max moment about 2 axis	23.21	(kip-ft)	50	25
min moment about 2 axis	-23.23	(kip-ft)	47	22
max moment about 3 axis	223.1	(kip-ft)	50	25
min moment about 3 axis	-277.9	(kip-ft)	50	25

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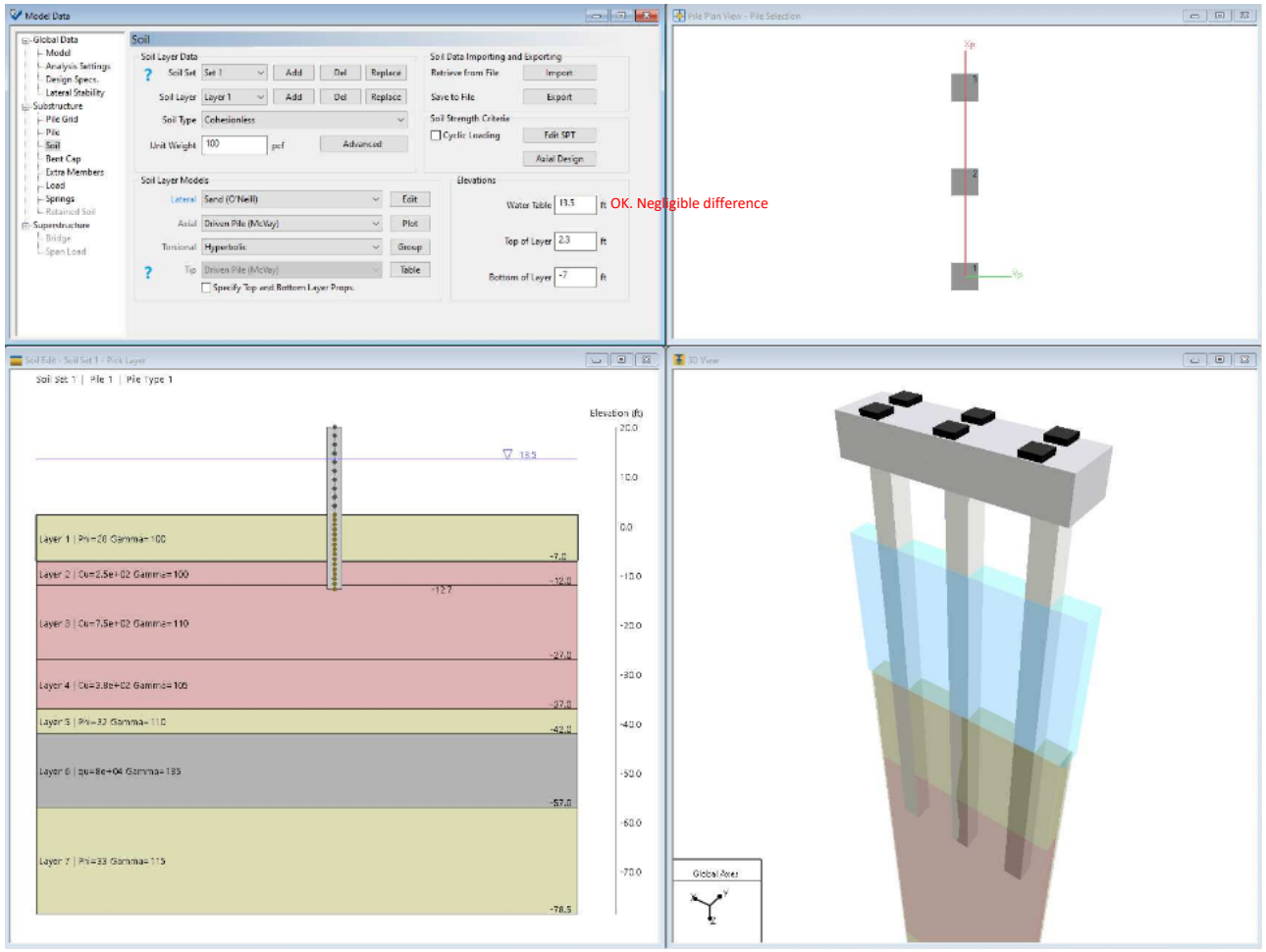
JOHN YARBROUGH LINEAR PARK

INTERMEDIATE BENT SOIL LAYERING – B-6



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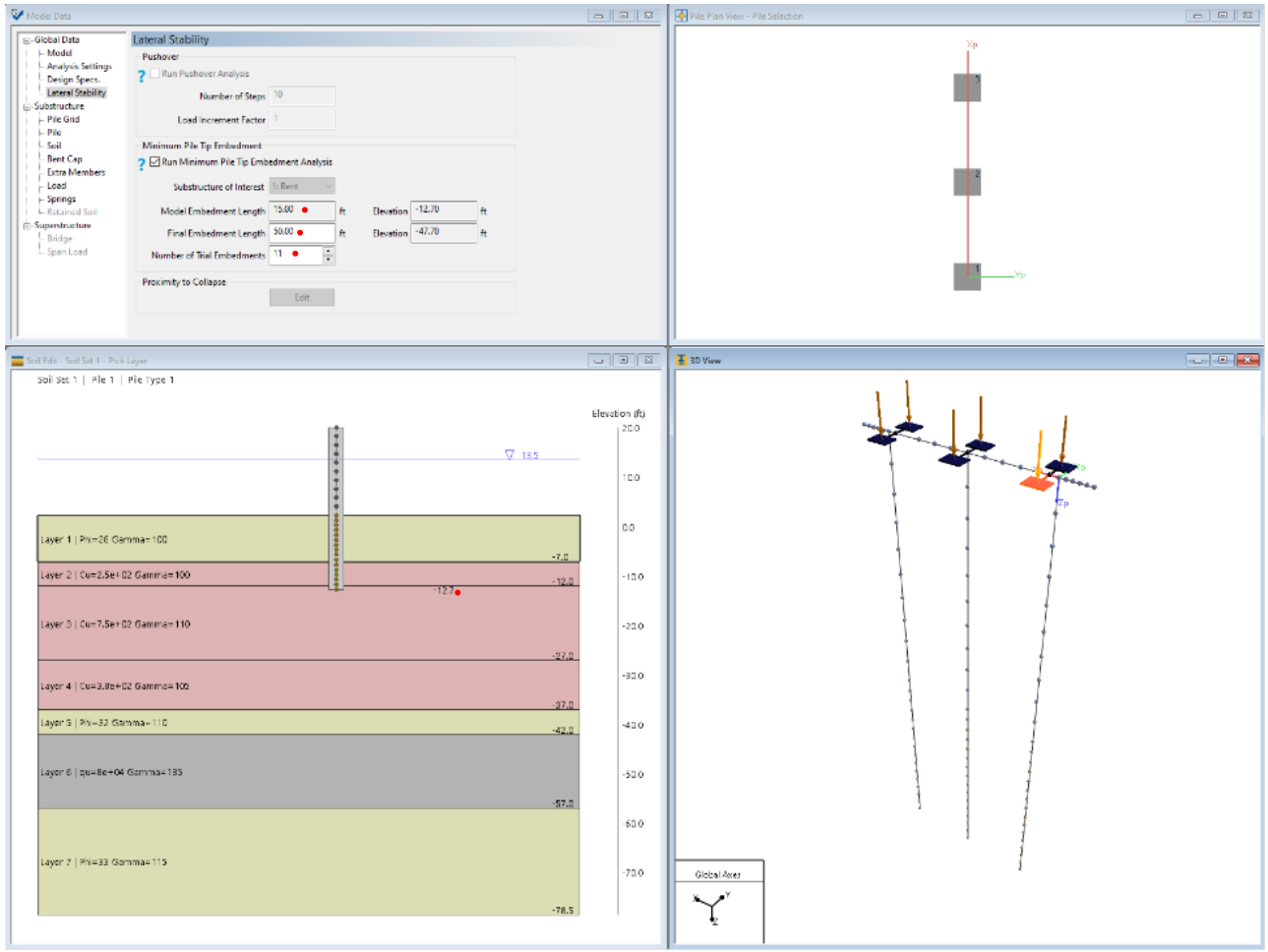
INTERMEDIATE BENT LATERAL STABILITY ANALYSIS – B-6



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INTERMEDIATE BENT MINIMUM TIP TABLE – B-6

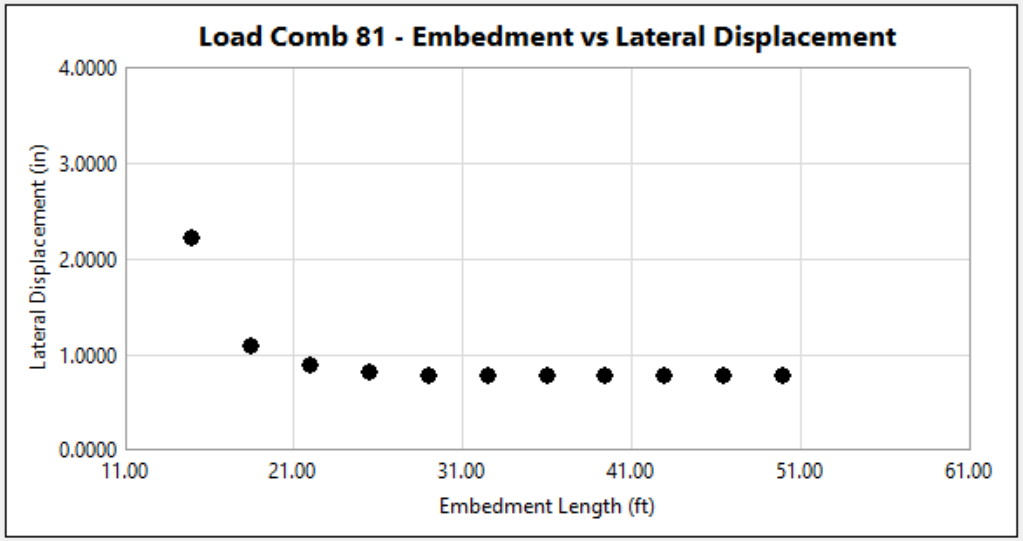
Minimum Pile Tip Embedment - Substructure 1

Individual Load Combination
Envelope (All Load Combs)

Load Comb 81

Plot Lateral Displacement

Max Envelope



Trial	Load Comb	Pile	Embedment (ft)	Displacement (in)
1	81	3	15.00	2.226250
2	81	3	18.50	1.086950
3	81	3	22.00	0.877922
4	81	3	25.50	0.805625
5	81	3	29.00	0.783409
6	81	3	32.50	0.779155
7	81	3	36.00	0.778330
8	81	3	39.50	0.777550

Notes

1. Results are presented for each load combination and each embedment.

2. The 'Lateral Displacement', 'Lateral X-Displacement', and 'Lateral Y-Displacement' plots display maximums of resultant Lateral Displacement, X-displacement, and Y-displacement across pile heads.

3. The 'Max Axial Force' plot displays maximum axial force for all pile elements. The 'Min Axial Force' plot displays minimum axial force for all pile elements.

Close

Excel

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FB-MultiPier

XML Report

XML File: MinTip_Soil2_new scour 8.4.22.XML

Project Information

Version	5.9.0
Client	Lee County
Project Name	Able Canal Pathway
Project Manager	
Description	Intermediate Bent - Minimum Tip - Soil 2 - Updated Scour
Computed By	JHJ
Project Date	08/2022

Boring B-6

Control Information

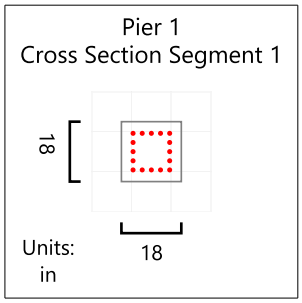
Control Settings

Units	English	
Maximum Iterations	100	
Tolerance	5	(kip)
Soil Behavior	0	
user supplied P-Y Multipliers must be given		
Tip Spring Option	0	
no linear tip springs on piles		
Pile Tip Stiffness	0	(kip/in)
Analysis	Static	
Design Code	AASHTO-LRFD	

NOTE:
Changes for this B-6 model vs previous B-5 model entail solely a change in soil parameters. Only soil parameters are checked in this output.

Pile Geometry - Substructure 1

Segment Number 1



Dimensions

Width	18	(in)
Depth	18	(in)
Web thickness	0	(in)
Flange thickness	0	(in)
Orientation	0	
Void type	None	
Section void diameter	0	(in)
Section void width	0	(in)
Section void depth	0	(in)
Embedded h-pile width	0	(in)
Embedded h-pile depth	0	(in)
Embedded h-pile web thickness	0	(in)
Embedded h-pile flange thickness	0	(in)

Embedded h-pile orientation	0	
Shell thickness	0	(in)
Shear steel	spiral	
Length	392.4	(in)

Material Properties

Concrete compressive strength	6	(ksi)
Concrete elastic modulus	4557	(ksi)
Mild steel yield length	0	(ksi)
Mild steel elastic modulus	0	(ksi)
Prestressing steel yield length	270	(ksi)
Prestressing steel elastic modulus	28500	(ksi)
H-pile steel yield strength	0	(ksi)
H-pile steel elastic modulus	0	(ksi)
Steel casing yield strength	0	(ksi)
Steel casing elastic modulus	0	(ksi)

Gross Section Properties

Area	324	(in^2)
Elastic modulus	4557	(ksi)
Inertia 2	8748	(in^4)
Inertia 3	8748	(in^4)
Shear modulus	1899	(ksi)
Torsional inertia	14780	(in^4)
Unit weight	8.6806E-05	(k/in^3)

Steel Groups

Bar Group #1

Bars	5	
Layer diameter	0	(in)
Starting local "2" coordinate	5.5	(in)
Starting local "3" coordinate	5.5	(in)
Bar area	0.153	(in^2)
Prestress	140	(ksi)
Orientation	2	

Bar Group #2

Bars	5	
Layer diameter	0	(in)
Starting local "2" coordinate	5.5	(in)
Starting local "3" coordinate	-5.5	(in)
Bar area	0.153	(in^2)
Prestress	140	(ksi)
Orientation	2	

Bar Group #3

Bars	3	
Layer diameter	0	(in)
Starting local "2" coordinate	5.5	(in)
Starting local "3" coordinate	2.75	(in)
Bar area	0.153	(in^2)
Prestress	140	(ksi)
Orientation	3	

Bar Group #4

Bars	3	
Layer diameter	0	(in)

Starting local "2" coordinate	-5.5	(in)
Starting local "3" coordinate	2.75	(in)
Bar area	0.153	(in^2)
Prestress	140	(ksi)
Orientation	3	

Stress-Strain Curves

Concrete

Point	Stress (ksi)	Strain
1	-4.335	-0.0038
2	-5.1	-0.0022383
3	-4.996	-0.0019186
4	-4.684	-0.0015988
5	-4.163	-0.001279
6	-3.435	-0.00095928
7	-2.498	-0.00063952
8	-1.457	-0.00031976
9	0	0
10	0.5996	0.00013158
11	0.2998	0.0003
12	0	0.002
13	0	0.005

Prestressing steel

Point	Stress (ksi)	Strain
1	-267.8	-0.03
2	-267.8	-0.0256
3	-266.7	-0.019
4	-262.6	-0.0124
5	-257.7	-0.0102
6	-245.1	-0.0086
7	0	0
8	245.1	0.0086
9	257.7	0.0102
10	262.6	0.0124
11	266.7	0.019
12	267.8	0.0256
13	267.8	0.03

Pile Group - Substructure 1

Bearing Capacity	0
Axial Efficiency (Uplift)	1
Axial Efficiency (Skin)	1
Axial Efficiency (Tip)	1

X Grid Spacings	(in)
Space 1	61
Space 2	61

Y Grid Spacings	()
Number of Piles	3

Pile to Cap Fixity

Pile #	Fixity
1	1

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2	1
3	1

Pile Sets

Set #1 Piles: 1 2 3

Soil Input Data

Lateral

Layer 1 Soil Set # 1 Model # 1

	Phi (deg)	Rk (lbs/in^3)	Gamma (pcf)
Top	28	6	100
Bottom	28	6	100

3. Say OK. Does not affect results.

Layer 2 Soil Set # 1 Model # 4

	Gamma (pcf)	Cu (psf)	E50
Top	100	250	0.05
Bottom	100	250	0.05

Layer 3 Soil Set # 1 Model # 5

	Rk (lbs/in^3)	Gamma (pcf)	Cu (psf)	E50	Cavg (psf)
Top	17	110	750	0.015	750
Bottom	17	110	750	0.015	750

Layer 4 Soil Set # 1 Model # 4

	Gamma (pcf)	Cu (psf)	E50
Top	105	375	0.02
Bottom	105	375	0.02

Layer 5 Soil Set # 1 Model # 1

	Phi (deg)	Rk (lbs/in^3)	Gamma (pcf)
Top	32	50	110
Bottom	32	50	110

Layer 6 Soil Set # 1 Model # 9

	Qu (psf)
Top	80000
Bottom	80000

Layer 7 Soil Set # 1 Model # 1

	Phi (deg)	Rk (lbs/in^3)	Gamma (pcf)
Top	33	69	115
Bottom	33	69	115

Layer 8 Soil Set # 1 Model # 9

	Qu (psf)
Top	80000
Bottom	80000

Axial

Layer 1 Soil Set # 1 Model # 1

	G (ksi)	Poisson's ratio	Tau max (psf)	Elevation (ft)	Piezometric elevation (ft)
Top	1	0.25	76	2.3	13.5

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Bottom	1	0.25	76	-7	13.5

Layer 2 Soil Set # 1 Model # 1

	G (ksi)	Poisson's ratio	Tau max (psf)	Elevation (ft)	Piezometric elevation (ft)
Top	1	0.45	216	-7	13.5
Bottom	1	0.45	216	-12	13.5

Layer 3 Soil Set # 1 Model # 1

	G (ksi)	Poisson's ratio	Tau max (psf)	Elevation (ft)	Piezometric elevation (ft)
Top	3	0.45	545	-12	13.5
Bottom	3	0.45	545	-27	13.5

Layer 4 Soil Set # 1 Model # 1

	G (ksi)	Poisson's ratio	Tau max (psf)	Elevation (ft)	Piezometric elevation (ft)
Top	1.5	0.45	320	-27	13.5
Bottom	1.5	0.45	320	-37	13.5

Layer 5 Soil Set # 1 Model # 1

	G (ksi)	Poisson's ratio	Tau max (psf)	Elevation (ft)	Piezometric elevation (ft)
Top	9	0.3	684	-37	13.5
Bottom	9	0.3	684	-42	13.5

Layer 6 Soil Set # 1 Model # 1

	G (ksi)	Poisson's ratio	Tau max (psf)	Elevation (ft)	Piezometric elevation (ft)
Top	50	0.5	1200	-42	13.5
Bottom	50	0.5	1200	-57	13.5

Layer 7 Soil Set # 1 Model # 1

	G (ksi)	Poisson's ratio	Tau max (psf)	Elevation (ft)	Piezometric elevation (ft)
Top	12.5	0.3	950	-57	13.5
Bottom	12.5	0.3	950	-78.5	13.5

Layer 8 Soil Set # 1 Model # 1

	G (ksi)	Poisson's ratio	Tau max (psf)	Elevation (ft)	Piezometric elevation (ft)
Top	50	0.5	1200	-78.5	13.5
Bottom	50	0.5	1200	-78.51	13.5

Torsion

Layer 1 Soil Set # 1 Model # 1

	G (ksi)	Tau max (psf)
Top	1	76
Bottom	1	76

G = 0.5 and Tau.max = 38.
Say OK. Does not affect results.

Layer 2 Soil Set # 1 Model # 1

	G (ksi)	Tau max (psf)
Top	1	216
Bottom	1	216

Layer 3 Soil Set # 1 Model # 1

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	G (ksi)	Tau max (psf)
Top	3	545
Bottom	3	545

Layer 4 Soil Set # 1 Model # 1

	G (ksi)	Tau max (psf)
Top	1.5	320
Bottom	1.5	320

Layer 5 Soil Set # 1 Model # 1

	G (ksi)	Tau max (psf)
Top	9	684
Bottom	9	684

Layer 6 Soil Set # 1 Model # 1

	G (ksi)	Tau max (psf)
Top	50	1200
Bottom	50	1200

Layer 7 Soil Set # 1 Model # 1

	G (ksi)	Tau max (psf)
Top	12.5	950
Bottom	12.5	950

Layer 8 Soil Set # 1 Model # 1

	G (ksi)	Tau max (psf)
Top	50	1200
Bottom	50	1200

Tip

Say OK. Not a factor to determine minimum tip at the high end bearing value.

	Model #	G (ksi)	Poisson's ratio	Ultimate bearing force (kips)
Soil Set #1	1	9	0.3	1000000

Pile Element Segments

Pile #1

Element #	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
Segment #	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1

Pile #2

Element #	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
Segment #	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1

Pile #3

Element #	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
Segment #	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1	1

Pile Coordinates

Pile #1

Point	X (in)	Y (in)	Z (in)
1	0	0	-240
2	0	0	-218.8
3	0	0	-197.5
4	0	0	-176.3

5	0	0	-155
6	0	0	-133.8
7	0	0	-112.6
8	0	0	-91.32
9	0	0	-70.08
10	0	0	-48.84
11	0	0	-27.6
12	0	0	-15.6
13	0	0	-3.6
14	0	0	8.4
15	0	0	20.4
16	0	0	32.4
17	0	0	44.4
18	0	0	56.4
19	0	0	68.4
20	0	0	80.4
21	0	0	92.4
22	0	0	104.4
23	0	0	116.4
24	0	0	128.4
25	0	0	140.4
26	0	0	152.4

Pile #2

Point	X (in)	Y (in)	Z (in)
1	61	0	-240
2	61	0	-218.8
3	61	0	-197.5
4	61	0	-176.3
5	61	0	-155
6	61	0	-133.8
7	61	0	-112.6
8	61	0	-91.32
9	61	0	-70.08
10	61	0	-48.84
11	61	0	-27.6
12	61	0	-15.6
13	61	0	-3.6
14	61	0	8.4
15	61	0	20.4
16	61	0	32.4
17	61	0	44.4
18	61	0	56.4
19	61	0	68.4
20	61	0	80.4
21	61	0	92.4
22	61	0	104.4
23	61	0	116.4
24	61	0	128.4
25	61	0	140.4
26	61	0	152.4

Pile #3

Point	X (in)	Y (in)	Z (in)
1	122	0	-240
2	122	0	-218.8
3	122	0	-197.5
4	122	0	-176.3
5	122	0	-155
6	122	0	-133.8
7	122	0	-112.6
8	122	0	-91.32

9	122	0	-70.08
10	122	0	-48.84
11	122	0	-27.6
12	122	0	-15.6
13	122	0	-3.6
14	122	0	8.4
15	122	0	20.4
16	122	0	32.4
17	122	0	44.4
18	122	0	56.4
19	122	0	68.4
20	122	0	80.4
21	122	0	92.4
22	122	0	104.4
23	122	0	116.4
24	122	0	128.4
25	122	0	140.4
26	122	0	152.4

Pier Geometry - Substructure 1

Modeling

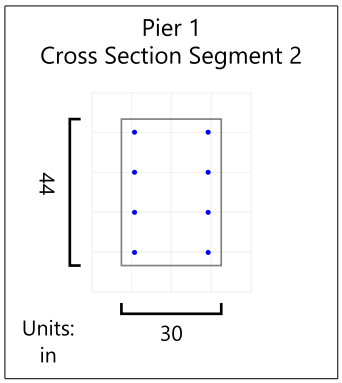
Structure type	Pile bent	
Number of columns	3	
Number of elements per span	5	
Cantilever length	1.75	(ft)
Number of elements per cantilever	5	
Bearing pads		
Pad # 1	Located @ Node 9	
Pad # 2	Located @ Node 2	
Pad # 3	Located @ Node 16	
Column taper	no	
Cantilever taper	no	
Beam taper	no	

Nodal Coordinates

		Boundary Conditions						Coordinates		
Node #	Location	X	Y	Z	Xx	Yy	Zz	X (in)	Y (in)	Z (in)
4	Pier	A	A	A	A	A	A	-21	0	-240
5	Pier	A	A	A	A	A	A	-16.8	0	-240
6	Pier	A	A	A	A	A	A	-12.6	0	-240
7	Pier	A	A	A	A	A	A	-8.4	0	-240
8	Pier	A	A	A	A	A	A	-4.2	0	-240
9	Pier	A	A	A	A	A	A	6	0	-240
10	Pier	A	A	A	A	A	A	24.4	0	-240
11	Pier	A	A	A	A	A	A	36.6	0	-240
12	Pier	A	A	A	A	A	A	48.8	0	-240
13	Pier	A	A	A	A	A	A	73.2	0	-240
14	Pier	A	A	A	A	A	A	85.4	0	-240
15	Pier	A	A	A	A	A	A	97.6	0	-240
16	Pier	A	A	A	A	A	A	116	0	-240
17	Pier	A	A	A	A	A	A	126.2	0	-240
18	Pier	A	A	A	A	A	A	130.4	0	-240
19	Pier	A	A	A	A	A	A	134.6	0	-240
20	Pier	A	A	A	A	A	A	138.8	0	-240
21	Pier	A	A	A	A	A	A	143	0	-240

Cross Sections

Segment Number 2



Dimensions

Width	44	(in)
Depth	30	(in)
Web thickness	0	(in)
Flange thickness	0	(in)
Orientation	0	
Void type	None	
Section void diameter	0	(in)
Section void width	0	(in)
Section void depth	0	(in)
Embedded h-pile width	0	(in)
Embedded h-pile depth	0	(in)
Embedded h-pile web thickness	0	(in)
Embedded h-pile flange thickness	0	(in)
Embedded h-pile orientation	0	
Shell thickness	0	(in)
Shear steel	tie	

Material Properties

Concrete compressive strength	5.5	(ksi)
Concrete elastic modulus	4428	(ksi)
Mild steel yield length	60	(ksi)
Mild steel elastic modulus	29000	(ksi)
Prestressing steel yield length	0	(ksi)
Prestressing steel elastic modulus	0	(ksi)
H-pile steel yield strength	0	(ksi)
H-pile steel elastic modulus	0	(ksi)
Steel casing yield strength	0	(ksi)
Steel casing elastic modulus	0	(ksi)

Gross Section Properties

Area	1320	(in^2)
Elastic modulus	4428	(ksi)
Inertia 2	213000	(in^4)
Inertia 3	99000	(in^4)
Shear modulus	1845	(ksi)
Torsional inertia	229400	(in^4)
Unit weight	8.6806E-05	(k/in^3)

Steel Groups

Bar Group #1

Bars	4	
Layer diameter	0	(in)
Starting local "2" coordinate	11.06	(in)
Starting local "3" coordinate	18.06	(in)

Bar area	0.31	(in^2)
Prestress	0	(ksi)
Orientation	3	

Bar Group #2

Bars	4	
Layer diameter	0	(in)
Starting local "2" coordinate	-11.06	(in)
Starting local "3" coordinate	18.06	(in)
Bar area	0.31	(in^2)
Prestress	0	(ksi)
Orientation	3	

Stress-Strain Curves

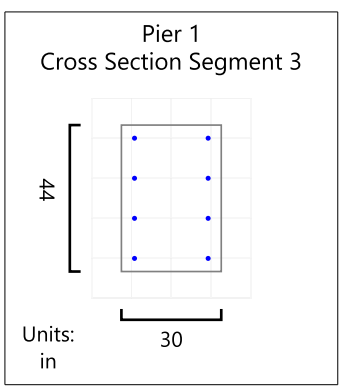
Concrete

Point	Stress (ksi)	Strain
1	-3.974	-0.0038
2	-4.675	-0.0021114
3	-4.58	-0.0018098
4	-4.293	-0.0015082
5	-3.816	-0.0012065
6	-3.148	-0.00090489
7	-2.29	-0.00060326
8	-1.336	-0.00030163
9	0	0
10	0.5827	0.00013158
11	0.2913	0.0003
12	0	0.002
13	0	0.005

Mild steel

Point	Stress (ksi)	Strain
1	-60	-0.09999
2	-60	-0.002069
3	60	0.002069
4	60	0.09999

Segment Number 3



Dimensions

Width	44	(in)
Depth	30	(in)
Web thickness	0	(in)
Flange thickness	0	(in)
Orientation	0	

Void type	None	
Section void diameter	0	(in)
Section void width	0	(in)
Section void depth	0	(in)
Embedded h-pile width	0	(in)
Embedded h-pile depth	0	(in)
Embedded h-pile web thickness	0	(in)
Embedded h-pile flange thickness	0	(in)
Embedded h-pile orientation	0	
Shell thickness	0	(in)
Shear steel	tie	

Material Properties

Concrete compressive strength	5.5	(ksi)
Concrete elastic modulus	4428	(ksi)
Mild steel yield length	60	(ksi)
Mild steel elastic modulus	29000	(ksi)
Prestressing steel yield length	0	(ksi)
Prestressing steel elastic modulus	0	(ksi)
H-pile steel yield strength	0	(ksi)
H-pile steel elastic modulus	0	(ksi)
Steel casing yield strength	0	(ksi)
Steel casing elastic modulus	0	(ksi)

Gross Section Properties

Area	1320	(in^2)
Elastic modulus	4428	(ksi)
Inertia 2	213000	(in^4)
Inertia 3	99000	(in^4)
Shear modulus	1845	(ksi)
Torsional inertia	229400	(in^4)
Unit weight	8.6806E-05	(k/in^3)

Steel Groups

Bar Group #1

Bars	4	
Layer diameter	0	(in)
Starting local "2" coordinate	11.06	(in)
Starting local "3" coordinate	18.06	(in)
Bar area	0.31	(in^2)
Prestress	0	(ksi)
Orientation	3	

Bar Group #2

Bars	4	
Layer diameter	0	(in)
Starting local "2" coordinate	-11.06	(in)
Starting local "3" coordinate	18.06	(in)
Bar area	0.31	(in^2)
Prestress	0	(ksi)
Orientation	3	

Stress-Strain Curves

Concrete

Point	Stress (ksi)	Strain
1	-3.974	-0.0038
2	-4.675	-0.0021114

3	-4.58	-0.0018098
4	-4.293	-0.0015082
5	-3.816	-0.0012065
6	-3.148	-0.00090489
7	-2.29	-0.00060326
8	-1.336	-0.00030163
9	0	0
10	0.5827	0.00013158
11	0.2913	0.0003
12	0	0.002
13	0	0.005

Mild steel

Point	Stress (ksi)	Strain
1	-60	-0.09999
2	-60	-0.002069
3	60	0.002069
4	60	0.09999

Pier Member Connectivity

Beam Element #

Load Case Data - Substructure 1

Load Values

Load Case	AASHTO Load Type	Node	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1	DC	1	0	0	0	0	0	0
17	WS1	2	0.92	0	0	0	0	0
17	WS1	26	0.2642	0	0	0	0	0
17	WS1	25	0.2642	0	0	0	0	0
17	WS1	24	0.2642	0	0	0	0	0
17	WS1	22	0.1321	0	0	0	0	0
17	WS1	52	0.1321	0	0	0	0	0
17	WS1	51	0.2642	0	0	0	0	0
17	WS1	50	0.2642	0	0	0	0	0
17	WS1	49	0.2642	0	0	0	0	0
17	WS1	47	0.1321	0	0	0	0	0
17	WS1	77	0.1321	0	0	0	0	0
17	WS1	76	0.2642	0	0	0	0	0
17	WS1	75	0.2642	0	0	0	0	0
17	WS1	74	0.2642	0	0	0	0	0
17	WS1	72	0.1321	0	0	0	0	0
17	WS1	23	0.2642	0	0	0	0	0
17	WS1	27	0.1321	0	0	0	0	0
17	WS1	48	0.2642	0	0	0	0	0
17	WS1	73	0.2642	0	0	0	0	0
18	WS2	2	2.18	0.0915	0	0	0	0
18	WS2	4	0	0.01575	0	0	0	0
18	WS2	5	0	0.0315	0	0	0	0
18	WS2	6	0	0.0315	0	0	0	0
18	WS2	7	0	0.0315	0	0	0	0
18	WS2	8	0	0.0315	0	0	0	0
18	WS2	1	0	0.1298	0	0	0	0
18	WS2	10	0	0.1148	0	0	0	0
18	WS2	11	0	0.0915	0	0	0	0
18	WS2	13	0	0.0915	0	0	0	0
18	WS2	14	0	0.0915	0	0	0	0

18	WS2	15	0	0.1147	0	0	0	0
18	WS2	3	0	0.1298	0	0	0	0
18	WS2	17	0	0.0315	0	0	0	0
18	WS2	18	0	0.0315	0	0	0	0
18	WS2	19	0	0.0315	0	0	0	0
18	WS2	20	0	0.0315	0	0	0	0
18	WS2	21	0	0.01575	0	0	0	0
18	WS2	12	0	0.0915	0	0	0	0
18	WS2	22	0.1566	0.09013	0	0	0	0
18	WS2	23	0.3133	0.1803	0	0	0	0
18	WS2	24	0.3133	0.1803	0	0	0	0
18	WS2	25	0.3133	0.1803	0	0	0	0
18	WS2	26	0.3133	0.1803	0	0	0	0
18	WS2	27	0.1566	0.09013	0	0	0	0
18	WS2	47	0.1566	0.09013	0	0	0	0
18	WS2	48	0.3133	0.1803	0	0	0	0
18	WS2	49	0.3133	0.1803	0	0	0	0
18	WS2	50	0.3133	0.1803	0	0	0	0
18	WS2	51	0.3133	0.1803	0	0	0	0
18	WS2	52	0.1566	0.09013	0	0	0	0
18	WS2	72	0.1566	0.09013	0	0	0	0
18	WS2	73	0.3133	0.1803	0	0	0	0
18	WS2	74	0.3133	0.1803	0	0	0	0
18	WS2	75	0.3133	0.1803	0	0	0	0
18	WS2	76	0.3133	0.1803	0	0	0	0
18	WS2	77	0.1566	0.09013	0	0	0	0
19	WS3	2	1.72	0.2237	0	0	0	0
19	WS3	4	0	0.0385	0	0	0	0
19	WS3	5	0	0.077	0	0	0	0
19	WS3	6	0	0.077	0	0	0	0
19	WS3	7	0	0.077	0	0	0	0
19	WS3	8	0	0.077	0	0	0	0
19	WS3	1	0	0.3172	0	0	0	0
19	WS3	10	0	0.2805	0	0	0	0
19	WS3	11	0	0.2237	0	0	0	0
19	WS3	13	0	0.2237	0	0	0	0
19	WS3	14	0	0.2237	0	0	0	0
19	WS3	15	0	0.2805	0	0	0	0
19	WS3	3	0	0.3172	0	0	0	0
19	WS3	17	0	0.077	0	0	0	0
19	WS3	18	0	0.077	0	0	0	0
19	WS3	19	0	0.077	0	0	0	0
19	WS3	20	0	0.077	0	0	0	0
19	WS3	21	0	0.0385	0	0	0	0
19	WS3	22	0.09013	0.1566	0	0	0	0
19	WS3	24	0.1803	0.3133	0	0	0	0
19	WS3	25	0.1803	0.3133	0	0	0	0
19	WS3	26	0.1803	0.3133	0	0	0	0
19	WS3	27	0.09013	0.1566	0	0	0	0
19	WS3	47	0.09013	0.1566	0	0	0	0
19	WS3	49	0.1803	0.3133	0	0	0	0
19	WS3	50	0.1803	0.3133	0	0	0	0
19	WS3	51	0.1803	0.3133	0	0	0	0
19	WS3	52	0.09013	0.1566	0	0	0	0
19	WS3	72	0.09013	0.1566	0	0	0	0
19	WS3	74	0.1803	0.3133	0	0	0	0
19	WS3	75	0.1803	0.3133	0	0	0	0
19	WS3	76	0.1803	0.3133	0	0	0	0
19	WS3	77	0.09013	0.1566	0	0	0	0
19	WS3	12	0	0.2237	0	0	0	0
19	WS3	23	0.1803	0.3133	0	0	0	0
19	WS3	48	0.1803	0.3133	0	0	0	0

19	WS3	73	0.1803	0.3133	0	0	0	0
20	WS4	2	0.18	0	0	0	0	0
20	WS4	22	0.02625	0	0	0	0	0
20	WS4	24	0.0525	0	0	0	0	0
20	WS4	25	0.0525	0	0	0	0	0
20	WS4	26	0.0525	0	0	0	0	0
20	WS4	27	0.02625	0	0	0	0	0
20	WS4	47	0.02625	0	0	0	0	0
20	WS4	49	0.0525	0	0	0	0	0
20	WS4	50	0.0525	0	0	0	0	0
20	WS4	51	0.0525	0	0	0	0	0
20	WS4	52	0.02625	0	0	0	0	0
20	WS4	72	0.02625	0	0	0	0	0
20	WS4	74	0.0525	0	0	0	0	0
20	WS4	75	0.0525	0	0	0	0	0
20	WS4	76	0.0525	0	0	0	0	0
20	WS4	77	0.02625	0	0	0	0	0
20	WS4	23	0.0525	0	0	0	0	0
20	WS4	48	0.0525	0	0	0	0	0
20	WS4	73	0.0525	0	0	0	0	0
21	WS5	2	0.43	0.02033	0	0	0	0
21	WS5	6	0	0.007	0	0	0	0
21	WS5	7	0	0.007	0	0	0	0
21	WS5	8	0	0.007	0	0	0	0
21	WS5	1	0	0.02883	0	0	0	0
21	WS5	10	0	0.0255	0	0	0	0
21	WS5	11	0	0.02033	0	0	0	0
21	WS5	13	0	0.02033	0	0	0	0
21	WS5	14	0	0.02033	0	0	0	0
21	WS5	15	0	0.0255	0	0	0	0
21	WS5	3	0	0.02883	0	0	0	0
21	WS5	17	0	0.007	0	0	0	0
21	WS5	18	0	0.007	0	0	0	0
21	WS5	19	0	0.007	0	0	0	0
21	WS5	20	0	0.007	0	0	0	0
21	WS5	22	0.03063	0.0175	0	0	0	0
21	WS5	24	0.06125	0.035	0	0	0	0
21	WS5	25	0.06125	0.035	0	0	0	0
21	WS5	26	0.06125	0.035	0	0	0	0
21	WS5	27	0.03063	0.0175	0	0	0	0
21	WS5	47	0.03063	0.0175	0	0	0	0
21	WS5	49	0.06125	0.035	0	0	0	0
21	WS5	50	0.06125	0.035	0	0	0	0
21	WS5	51	0.06125	0.035	0	0	0	0
21	WS5	52	0.03063	0.0175	0	0	0	0
21	WS5	72	0.03063	0.0175	0	0	0	0
21	WS5	74	0.06125	0.035	0	0	0	0
21	WS5	75	0.06125	0.035	0	0	0	0
21	WS5	76	0.06125	0.035	0	0	0	0
21	WS5	77	0.03063	0.0175	0	0	0	0
21	WS5	4	0	0.0035	0	0	0	0
21	WS5	5	0	0.007	0	0	0	0
21	WS5	12	0	0.02033	0	0	0	0
21	WS5	21	0	0.0035	0	0	0	0
21	WS5	23	0.06125	0.035	0	0	0	0
21	WS5	48	0.06125	0.035	0	0	0	0
21	WS5	73	0.06125	0.035	0	0	0	0
22	WS6	2	0.34	0.04067	0	0	0	0
22	WS6	4	0	0.007	0	0	0	0
22	WS6	5	0	0.014	0	0	0	0
22	WS6	6	0	0.014	0	0	0	0
22	WS6	7	0	0.014	0	0	0	0

22	WS6	8	0	0.014	0	0	0	0
22	WS6	1	0	0.05767	0	0	0	0
22	WS6	10	0	0.051	0	0	0	0
22	WS6	11	0	0.04067	0	0	0	0
22	WS6	13	0	0.04067	0	0	0	0
22	WS6	14	0	0.04067	0	0	0	0
22	WS6	15	0	0.051	0	0	0	0
22	WS6	3	0	0.05767	0	0	0	0
22	WS6	17	0	0.014	0	0	0	0
22	WS6	18	0	0.014	0	0	0	0
22	WS6	19	0	0.014	0	0	0	0
22	WS6	20	0	0.014	0	0	0	0
22	WS6	21	0	0.007	0	0	0	0
22	WS6	22	0.0175	0.03063	0	0	0	0
22	WS6	24	0.035	0.06125	0	0	0	0
22	WS6	25	0.035	0.06125	0	0	0	0
22	WS6	26	0.035	0.06125	0	0	0	0
22	WS6	27	0.0175	0.03063	0	0	0	0
22	WS6	47	0.0175	0.03063	0	0	0	0
22	WS6	49	0.035	0.06125	0	0	0	0
22	WS6	50	0.035	0.06125	0	0	0	0
22	WS6	51	0.035	0.06125	0	0	0	0
22	WS6	52	0.0175	0.03063	0	0	0	0
22	WS6	72	0.0175	0.03063	0	0	0	0
22	WS6	74	0.035	0.06125	0	0	0	0
22	WS6	75	0.035	0.06125	0	0	0	0
22	WS6	76	0.035	0.06125	0	0	0	0
22	WS6	77	0.0175	0.03063	0	0	0	0
22	WS6	12	0	0.04067	0	0	0	0
22	WS6	23	0.035	0.06125	0	0	0	0
22	WS6	48	0.035	0.06125	0	0	0	0
22	WS6	73	0.035	0.06125	0	0	0	0
25	TU	1	0	0	0	0	0	0

Load Case #1

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	DC	0	0	33.4	0	0	0
1R	DC	0	0	33.4	0	0	0
2L	DC	0	0	33.4	0	0	0
2R	DC	0	0	33.4	0	0	0
3L	DC	0	0	33.4	0	0	0
3R	DC	0	0	33.4	0	0	0

Load Case #2

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	LL1	0	0	6.98	0	0	0
1R	LL1	0	0	1.51	0	0	0
2L	LL1	0	0	5.53	0	0	0
2R	LL1	0	0	1.2	0	0	0
3L	LL1	0	0	3.49	0	0	0
3R	LL1	0	0	0.76	0	0	0

Load Case #3

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	LL2	0	0	3.49	0	0	0
1R	LL2	0	0	0.76	0	0	0
2L	LL2	0	0	5.53	0	0	0
2R	LL2	0	0	1.2	0	0	0
3L	LL2	0	0	6.98	0	0	0
3R	LL2	0	0	1.51	0	0	0

Load Case #4

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	LL3	0	0	9.54	0	0	0
1R	LL3	0	0	9.54	0	0	0
2L	LL3	0	0	9.54	0	0	0
2R	LL3	0	0	9.54	0	0	0
3L	LL3	0	0	9.54	0	0	0
3R	LL3	0	0	9.54	0	0	0

Load Case #5

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	LL4	0	0	9.54	0	0	0
1R	LL4	0	0	0	0	0	0
2L	LL4	0	0	9.54	0	0	0
2R	LL4	0	0	0	0	0	0
3L	LL4	0	0	9.54	0	0	0
3R	LL4	0	0	0	0	0	0

Load Case #6

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	LL5	0	0	0	0	0	0
1R	LL5	0	0	9.54	0	0	0
2L	LL5	0	0	0	0	0	0
2R	LL5	0	0	9.54	0	0	0
3L	LL5	0	0	0	0	0	0
3R	LL5	0	0	9.54	0	0	0

Load Case #7

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	IM1	0	0	0	0	0	0
1R	IM1	0	0	0	0	0	0
2L	IM1	0	0	0	0	0	0
2R	IM1	0	0	0	0	0	0
3L	IM1	0	0	0	0	0	0
3R	IM1	0	0	0	0	0	0

Load Case #8

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	IM2	0	0	0	0	0	0
1R	IM2	0	0	0	0	0	0
2L	IM2	0	0	0	0	0	0
2R	IM2	0	0	0	0	0	0
3L	IM2	0	0	0	0	0	0
3R	IM2	0	0	0	0	0	0

Load Case #9

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	IM3	0	0	0	0	0	0
1R	IM3	0	0	0	0	0	0
2L	IM3	0	0	0	0	0	0
2R	IM3	0	0	0	0	0	0
3L	IM3	0	0	0	0	0	0
3R	IM3	0	0	0	0	0	0

Load Case #10

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	IM4	0	0	0	0	0	0
1R	IM4	0	0	0	0	0	0
2L	IM4	0	0	0	0	0	0
2R	IM4	0	0	0	0	0	0
3L	IM4	0	0	0	0	0	0
3R	IM4	0	0	0	0	0	0

Load Case #11

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	IM5	0	0	0	0	0	0
1R	IM5	0	0	0	0	0	0
2L	IM5	0	0	0	0	0	0
2R	IM5	0	0	0	0	0	0
3L	IM5	0	0	0	0	0	0
3R	IM5	0	0	0	0	0	0

Load Case #12

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	BR1	0	0	0	0	0	0
1R	BR1	0	0	0	0	0	0
2L	BR1	0	0	0	0	0	0
2R	BR1	0	0	0	0	0	0
3L	BR1	0	0	0	0	0	0
3R	BR1	0	0	0	0	0	0

Load Case #13

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	BR2	0	0	0	0	0	0
1R	BR2	0	0	0	0	0	0
2L	BR2	0	0	0	0	0	0
2R	BR2	0	0	0	0	0	0
3L	BR2	0	0	0	0	0	0
3R	BR2	0	0	0	0	0	0

Load Case #14

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	BR3	0	0	0	0	0	0
1R	BR3	0	0	0	0	0	0
2L	BR3	0	0	0	0	0	0
2R	BR3	0	0	0	0	0	0
3L	BR3	0	0	0	0	0	0
3R	BR3	0	0	0	0	0	0

Load Case #15

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	BR4	0	0	0	0	0	0
1R	BR4	0	0	0	0	0	0
2L	BR4	0	0	0	0	0	0
2R	BR4	0	0	0	0	0	0
3L	BR4	0	0	0	0	0	0
3R	BR4	0	0	0	0	0	0

Load Case #16

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	BR5	0	0	0	0	0	0
1R	BR5	0	0	0	0	0	0
2L	BR5	0	0	0	0	0	0
2R	BR5	0	0	0	0	0	0
3L	BR5	0	0	0	0	0	0
3R	BR5	0	0	0	0	0	0

Load Case #17

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	WS1	3.55	0	-2.86	0	0	0
1R	WS1	3.55	0	-2.86	0	0	0
2L	WS1	3.55	0	0	0	0	0
2R	WS1	3.55	0	0	0	0	0
3L	WS1	3.55	0	2.86	0	0	0
3R	WS1	3.55	0	2.86	0	0	0

Load Case #18

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	WS2	2.91	0.85	-2.34	0	0	0
1R	WS2	2.91	0.85	-2.34	0	0	0
2L	WS2	2.91	0.85	0	0	0	0
2R	WS2	2.91	0.85	0	0	0	0
3L	WS2	2.91	0.85	2.34	0	0	0
3R	WS2	2.91	0.85	2.34	0	0	0

Load Case #19

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	WS3	1.21	1.35	-0.97	0	0	0
1R	WS3	1.21	1.35	-0.97	0	0	0
2L	WS3	1.21	1.35	0	0	0	0
2R	WS3	1.21	1.35	0	0	0	0
3L	WS3	1.21	1.35	0.97	0	0	0
3R	WS3	1.21	1.35	0.97	0	0	0

Load Case #20

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	WS4	0.71	0	-0.57	0	0	0
1R	WS4	0.71	0	-0.57	0	0	0
2L	WS4	0.71	0	0	0	0	0
2R	WS4	0.57	0	0	0	0	0
3L	WS4	0.71	0	0.57	0	0	0
3R	WS4	0.71	0	0.57	0	0	0

Load Case #21

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	WS5	0.58	0.17	-0.47	0	0	0
1R	WS5	0.58	0.17	-0.47	0	0	0
2L	WS5	0.58	0.17	0	0	0	0
2R	WS5	0.58	0.17	0	0	0	0
3L	WS5	0.58	0.17	0.47	0	0	0
3R	WS5	0.58	0.17	0.47	0	0	0

Load Case #22

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	WS6	0.24	0.27	-0.19	0	0	0
1R	WS6	0.24	0.27	-0.19	0	0	0
2L	WS6	0.24	0.27	0	0	0	0
2R	WS6	0.24	0.27	0	0	0	0
3L	WS6	0.24	0.27	0.19	0	0	0
3R	WS6	0.24	0.27	0.19	0	0	0

Load Case #23

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	VP	0	0	-6.27	0	0	0
1R	VP	0	0	-6.27	0	0	0
2L	VP	0	0	0	0	0	0
2R	VP	0	0	0	0	0	0
3L	VP	0	0	-0.93	0	0	0
3R	VP	0	0	-0.93	0	0	0

Load Case #24

Bearing Values

Node	AASHTO Load Type	Fx (kip)	Fy (kip)	Fz (kip)	Mx (kip-ft)	My (kip-ft)	Mz (kip-ft)
1L	VR	0	0	-0.93	0	0	0
1R	VR	0	0	-0.93	0	0	0
2L	VR	0	0	0	0	0	0
2R	VR	0	0	0	0	0	0
3L	VR	0	0	-6.27	0	0	0
3R	VR	0	0	-6.27	0	0	0

Output Summary - Substructure 1

Max/Min Pile Strains	Value		Load Case	Combination	Type	Segment	Pile
max strain in concrete	0.0010167	(in/in)	53	28	1	1	3
corresponding stress	0.1734	(ksi)	53	28	1	1	3
min strain in concrete	-0.0015877	(in/in)	53	28	1	1	3
corresponding stress	-4.666	(ksi)	53	28	1	1	3
max strain in reinforcement (prestressed)	0.0056809	(in/in)	53	28	1	1	3
corresponding stress	161.9	(ksi)	53	28	1	1	3
min strain in steel (prestressed)	0.004037	(in/in)	53	28	1	1	3
corresponding stress	115.1	(ksi)	53	28	1	1	3

Max/Min Pier Column and Pier Cap Strains	Value		Load Case	Combination	Section	Element
max strain in concrete	9.4369E-05	(in/in)	47	22	3	15
corresponding stress	0.4179	(ksi)	47	22	3	15
min strain in concrete	-9.6593E-05	(in/in)	47	22	3	15
corresponding stress	-0.4277	(ksi)	47	22	3	15
max strain in steel (mild)	7.1916E-05	(in/in)	47	22	3	15
corresponding stress	2.086	(ksi)	47	22	3	15
min strain in steel (mild)	-7.414E-05	(in/in)	47	22	3	15
corresponding stress	-2.15	(ksi)	47	22	3	15
max strain in concrete	3.7174E-05	(in/in)	50	25	4	13
corresponding stress	0.1646	(ksi)	50	25	4	13
min strain in concrete	-3.6927E-05	(in/in)	50	25	4	13
corresponding stress	-0.1635	(ksi)	50	25	4	13
max strain in steel (mild)	2.8474E-05	(in/in)	50	25	4	13
corresponding stress	0.8258	(ksi)	50	25	4	13
min strain in steel (mild)	-2.8227E-05	(in/in)	50	25	4	13
corresponding stress	-0.8186	(ksi)	50	25	4	13

Max/Min Pile Internal Forces	Value		Load Case	Combination	Pile
max shear in 2 direction	9.954	(kip)	52	27	3
min shear in 2 direction	-9.954	(kip)	55	30	1
max shear in 3 direction	25.22	(kip)	51	26	1
min shear in 3 direction	-25.19	(kip)	48	23	3

max moment about 2 axis	87.91 (kip-ft)	53	28	1
min moment about 2 axis	-87.75 (kip-ft)	56	31	3
max moment about 3 axis	252.3 (kip-ft)	47	22	3
min moment about 3 axis	-252.2 (kip-ft)	50	25	1
max axial force	6.868 (kip)	55	30	3
min axial force	-170.4 (kip)	47	22	3
max torsional force	16.88 (kip-ft)	53	28	3
min torsional force	-16.88 (kip-ft)	53	28	3
max demand/capacity ratio	1.096	47	22	3

Max/Min Soil Reaction Forces	Value	Load Case	Combination	Pile
max axial soil force	165.9 (kip)	47	22	3
min axial soil force	0.01449 (kip)	55	30	3
max lateral in X direction	3.396 (kip)	49	24	1
min lateral in X direction	-3.396 (kip)	46	21	3
max lateral in Y direction	7.271 (kip)	54	29	3
min lateral in Y direction	-7.275 (kip)	57	32	1
max torsional soil force	0.01051 (kip-ft)	86	61	1

Max/Min Pile Head Displacements	Value	Load Case	Combination	Pile
max displacement in axial	0.3307 (in)	47	22	3
min displacement in axial	0.0011722 (in)	55	30	3
max displacement in x	4.603 (in)	47	22	2
min displacement in x	-4.6 (in)	50	25	2
max displacement in y	33.38 (in)	57	32	1
min displacement in y	-33.34 (in)	54	29	3

Max/Min Pier Cap Internal Forces	Value	Load Case	Combination
max axial force	6.777 (kip)	49	24
min axial force	-8.614 (kip)	52	27
max shear in 2 direction	159.2 (kip)	50	25
min shear in 2 direction	-159.2 (kip)	47	22
max shear in 3 direction	6.885 (kip)	53	28
min shear in 3 direction	-0.6319 (kip)	108	83
max torque	45.11 (kip-ft)	53	28
min torque	-45.11 (kip-ft)	53	28
max moment about 2 axis	17.01 (kip-ft)	56	31
min moment about 2 axis	-17.02 (kip-ft)	53	28
max moment about 3 axis	207.4 (kip-ft)	47	22
min moment about 3 axis	-255.5 (kip-ft)	47	22

4.2. Factored Design Load

KISINGER CAMPO & ASSOCIATES

LEE COUNTY
ABLE CANAL PATHWAY

FOUNDATION FACTORED DESIGN LOAD

FACTORED DESIGN LOAD – INTERMEDIATE BENT

CALCULATION QUALITY CONTROL		
RESPONSIBLE PROFESSIONAL / ORIGINATOR	INITIALS: JHJ	DATE: 08/22
QUALITY REVIEWER	INITIALS: LM	DATE: 08/2022

SUMMARY OF SECTION

Factored design load for the intermediate bent is established. The analysis employs FB-MultiPier v5.9.0 The FB-MultiPier analyses are organized as follows:

- *Pile capacity curves*
- *Factored Design Load Model with B-5 parameters*
- *Factored Design Load Model with B-6 parameters*

This analysis includes only Control Information and Output Summary. For full model input information see the Minimum Tip section.

See included Geotechnical Parameters provided by the Geotechnical Engineer.

Boring	*Tip Elevation (ft.)*	*Factored Design Load (kips)*
B-5	-37.0	133 (205)
B-6	-38.0	138 (214)

*Nominal Bearing Resistance shown in parenthesis

Geotechnical Parameters for FB-MultiPier Input

B-5 (18 INCH PSC PILE)
Able Canal Pathway Over Charlie Diversion Canal
Lehigh Acres, Lee County, Florida

Ardaman Project No. 21-33-4537

Reference Boring	B-5
Ground Surface Elevation (ft)	20.00
Ground Water Table Elevation (ft)	13.50

Foundation Type	Driven Concrete Pile
Size (inch)	18
Base Area (ft^2)	2.25

	Layer No.	1	2	3	4	5
	Soil Description	Sand	Clay	Limestone	Sand	Limestone
	Soil Type	Cohesionless	Cohesive	Rock	Cohesionless	Rock
	Top Boundary Elevation (ft)	2.30	-7.50	-42.50	-47.00	-79.50
	Bottom boundary Elevation (ft)	-7.50	-42.50	-47.00	-79.50	-79.70
	Average SPT N-Value (Blows/ft)	3	7	57	24	100
LATERAL	Soil Model	Sand (O'Neill)	Clay (Soft<Water)	Limestone (McVay)	Sand (O'Neill)	Limestone (McVay)
	Internal Friction Angle, ϕ	29	-	-	33	-
	Total Unit Weight (pcf), γ_t	100	110	-	115	-
	Subgrade Modulus (pci), k	8	-	-	67	-
	Undrained Shear Strength (psf), c_u	-	875	-	-	-
	Major Principal Strain @ ε_{50}	-	0.015	-	-	-
	Major Principal Strain @ ε_{100}	-	-	-	-	-
	Average Undrained Shear Strength (psf)	-	-	-	-	-
	Unconfined Compressive Strength (psf)	-	-	57060	-	80000
AXIAL	Soil Model	Driven Pile	Driven Pile	Driven Pile	Driven Pile	Driven Pile
	Total Unit Weight (pcf), γ_t	100	110	135	115	135
	Shear Modulus (ksi), G	1.50	3.50	28.50	12.00	50.00
	Poisson's ratio, ν	0.25	0.45	0.50	0.30	0.50
	Vertical Failure Shear Stress (psf)	114	720	1140	912	1200
	Undrained Shear Strength (psf), c_u	-	875	-	-	-
	Ultimate Unit Skin Friction (psf)	-	-	-	-	-
	Mass Modulus (ksi)	-	-	-	-	-
	Modulus Ratio	-	-	-	-	-
	Surface (Rough/Smooth)	-	-	-	-	-
	Unconfined Compressive Strength (psf)	-	-	-	-	-
	Split Tensile Strength (psf)	-	-	-	-	-
	Concrete Unit Weight (pcf)	-	-	-	-	-
	Slump (in)	-	-	-	-	-
TORSIONAL	Soil Model	Hyperbolic	Hyperbolic	Hyperbolic	Hyperbolic	Hyperbolic
	Total Unit Weight (pcf), γ_t	100	110	135	115	135
	Internal Friction Angle, ϕ	29	-	35	33	35
	Undrained Shear Strength (psf), c_u	-	875	-	-	-
	Shear Modulus (ksi), G	1.5	3.5	28.5	12.0	50.0
	Torsional Shear Stress (psf)	114	720	1140	912	1200
TIP	Soil Model	Driven Pile	Driven Pile	Driven Pile	Driven Pile	Driven Pile
	Shear Modulus (ksi), G	1.50	3.50	28.50	12.00	50.00
	Poisson's ratio, ν	0.25	0.45	0.50	0.30	0.50
	Axial Bearing Failure, kips	43	22	923	346	972
	Uncorrected SPT-N Value (blows/ft)	-	-	-	-	-
	Undrained Shear Strength (psf), c_u	-	-	-	-	-
	IGM Mass Modulus (ksi), E_m	-	-	-	-	-

Ardaman & Associates, Inc.

Geotechnical Parameters for FB-MultiPier Input

B-6 (18 INCH PSC PILE)
Able Canal Pathway Over Charlie Diversion Canal
Lehigh Acres, Lee County, Florida

Ardaman Project No. 21-33-4537

Reference Boring	B-6
Ground Surface Elevation (ft)	20.00
Ground Water Table Elevation (ft)	13.50

Foundation Type	Driven Concrete Pile
Size (inch)	18
Base Area (ft^2)	2.25

	Layer No.	1	2	3	4	5	6	7	8
	Soil Description	Sand	Clay	Sand-Clay	Clay	Sand	Limestone	Sand	Limestone
	Soil Type	Cohesionless	Cohesive	Cohesive	Cohesive	Cohesionless	Rock	Cohesionless	Rock
	Top Boundary Elevation (ft)	2.30	-7.00	-12.00	-27.00	-37.00	-42.00	-57.00	-78.50
	Bottom boundary Elevation (ft)	-7.00	-12.00	-27.00	-37.00	-42.00	-57.00	-78.50	-78.51
	Average SPT N-Value (Blows/ft)	1	2	6	3	18	100	25	100
LATERAL	Soil Model	Sand (O'Neill)	Clay (Soft<Water)	Sand (O'Neill)	Clay (Soft<Water)	Sand (O'Neill)	Limestone (McVay)	Sand (O'Neill)	Limestone (McVay)
	Internal Friction Angle, ϕ	28	-	-	-	32	-	33	-
	Total Unit Weight (pcf), γ_t	100	100	110	105	110	-	115	-
	Subgrade Modulus (pci), k	3	-	17	-	50	-	69	-
	Undrained Shear Strength (psf), c_u	-	250	750	375	-	-	-	-
	Major Principal Strain @ ε_{50}	-	0.050	0.015	0.020	-	-	-	-
	Major Principal Strain @ ε_{100}	-	-	-	-	-	-	-	-
	Average Undrained Shear Strength (psf)	-	-	750	-	-	-	-	-
	Unconfined Compressive Strength (psf)	-	-	-	-	-	80000	-	80000
AXIAL	Soil Model	Driven Pile	Driven Pile	Driven Pile	Driven Pile	Driven Pile	Driven Pile	Driven Pile	Driven Pile
	Total Unit Weight (pcf), γ_t	100	100	110	105	110	135	115	135
	Shear Modulus (ksi), G	0.50	1.00	3.00	1.50	9.00	50.00	12.50	50.00
	Poisson's ratio, ν	0.25	0.45	0.45	0.45	0.30	0.50	0.30	0.50
	Vertical Failure Shear Stress (psf)	38	216	545	320	684	1200	950	1200
	Undrained Shear Strength (psf), c_u	-	250	750	375	-	-	-	-
	Ultimate Unit Skin Friction (psf)	-	-	-	-	-	-	-	-
	Mass Modulus (ksi)	-	-	-	-	-	-	-	-
	Modulus Ratio	-	-	-	-	-	-	-	-
	Surface (Rough/Smooth)	-	-	-	-	-	-	-	-
	Unconfined Compressive Strength (psf)	-	-	-	-	-	-	-	-
	Split Tensile Strength (psf)	-	-	-	-	-	-	-	-
	Concrete Unit Weight (pcf)	-	-	-	-	-	-	-	-
	Slump (in)	-	-	-	-	-	-	-	-
TORSIONAL	Soil Model	Hyperbolic	Hyperbolic	Hyperbolic	Hyperbolic	Hyperbolic	Hyperbolic	Hyperbolic	Hyperbolic
	Total Unit Weight (pcf), γ_t	100	100	110	105	110	135	115	135
	Internal Friction Angle, ϕ	28	-	-	-	32	35	33	35
	Undrained Shear Strength (psf), c_u	-	250	750	375	-	-	-	-
	Shear Modulus (ksi), G	0.5	1.0	3.0	1.5	9.0	50.0	12.5	50.0
	Torsional Shear Stress (psf)	38	216	545	320	684	1200	950	1200
TIP	Soil Model	Driven Pile	Driven Pile	Driven Pile	Driven Pile	Driven Pile	Driven Pile	Driven Pile	Driven Pile
	Shear Modulus (ksi), G	0.50	1.00	3.00	1.50	9.00	50.00	12.50	50.00
	Poisson's ratio, ν	0.25	0.45	0.45	0.45	0.30	0.50	0.30	0.50
	Axial Bearing Failure, kips	14	6	43	9	259	972	360	972
	Uncorrected SPT-N Value (blows/ft)	-	-	-	-	-	-	-	-
	Undrained Shear Strength (psf), c_u	-	-	-	-	-	-	-	-
	IGM Mass Modulus (ksi), E_m	-	-	-	-	-	-	-	-

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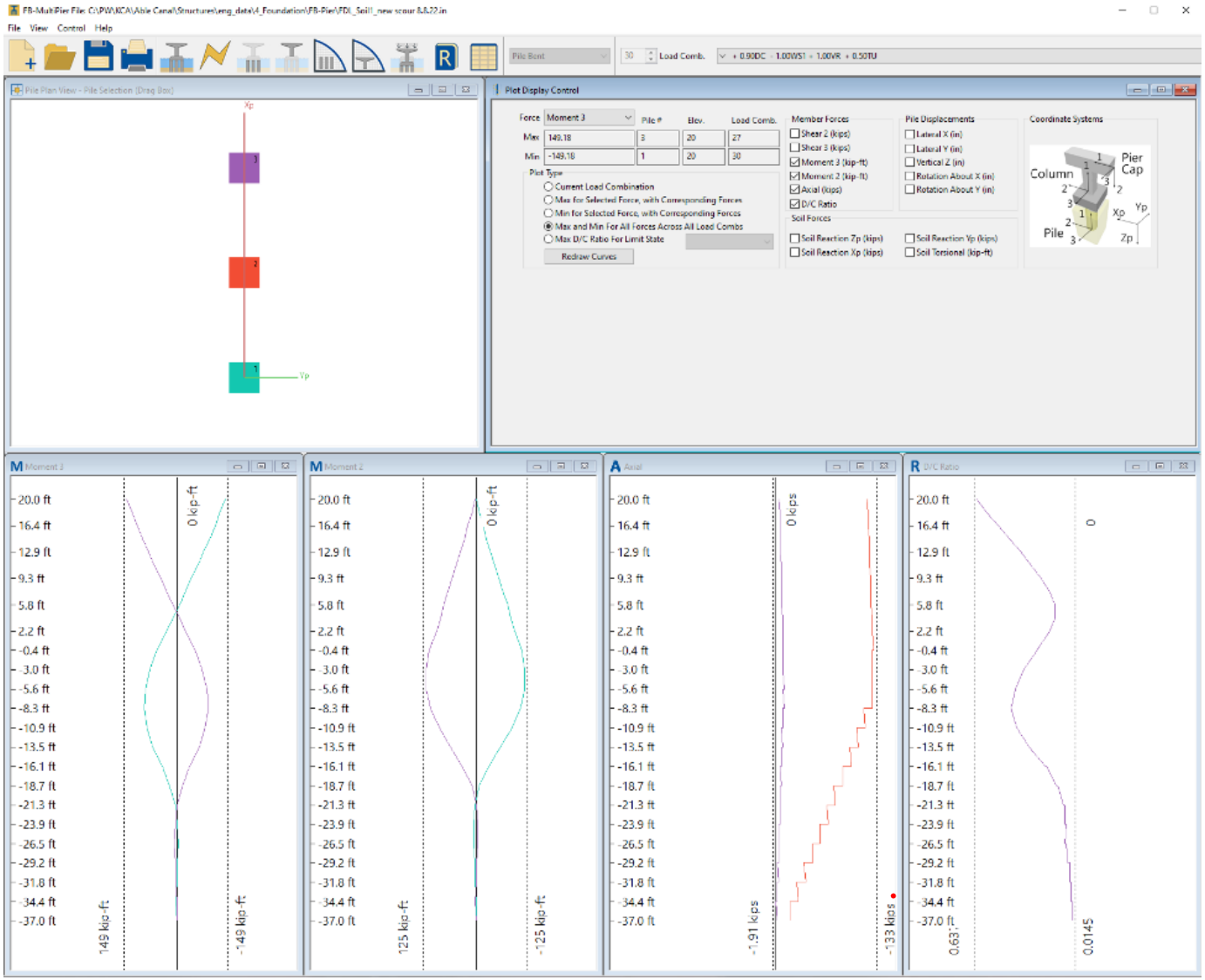
CITY OF FORT MYERS
ABLE CANAL PATHWAY

INTERMEDIATE BENT FACTORED DESIGN LOAD – B-5



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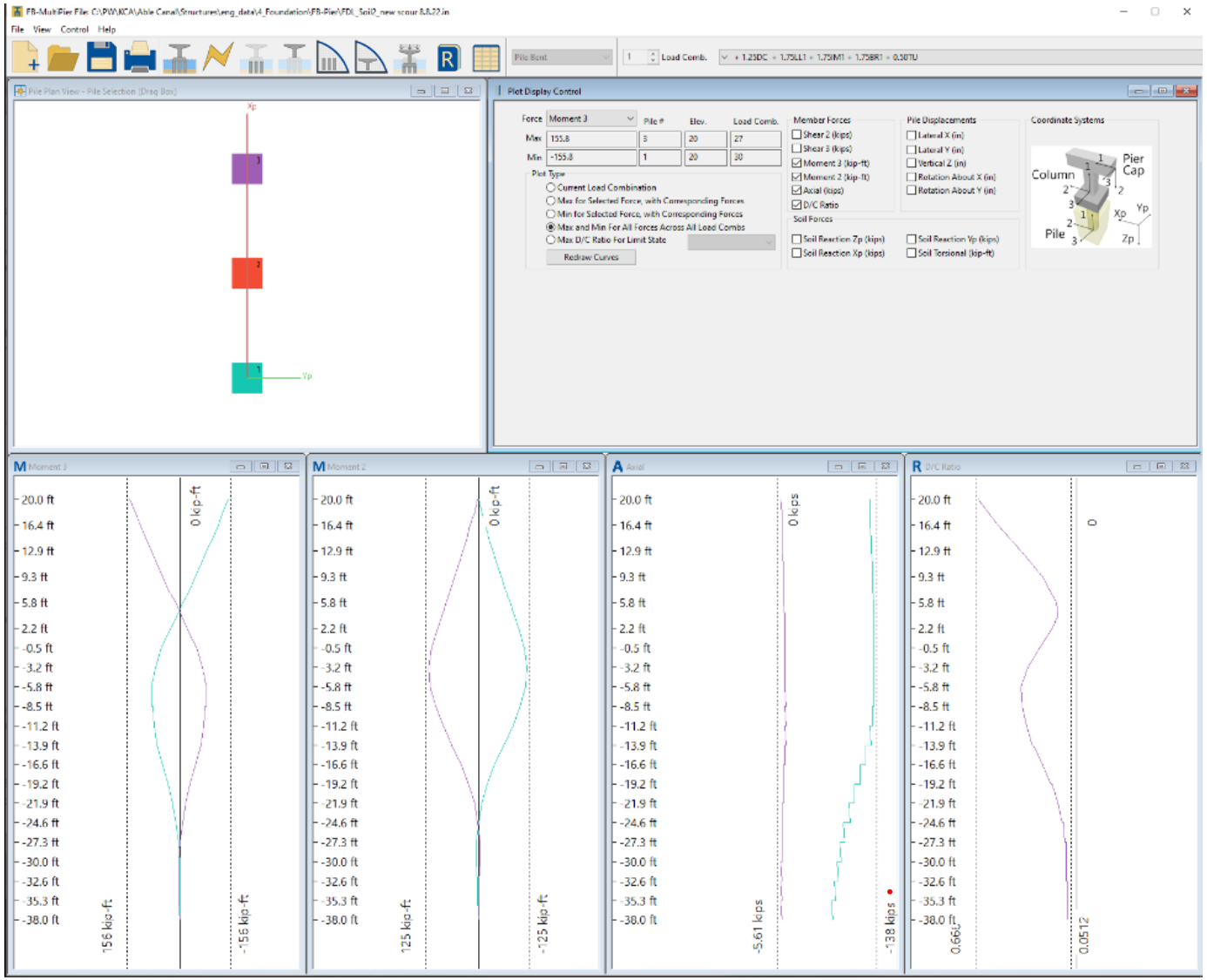
ABLE CANAL PATHWAY

INTERMEDIATE BENT FACTORED DESIGN LOAD – B-6



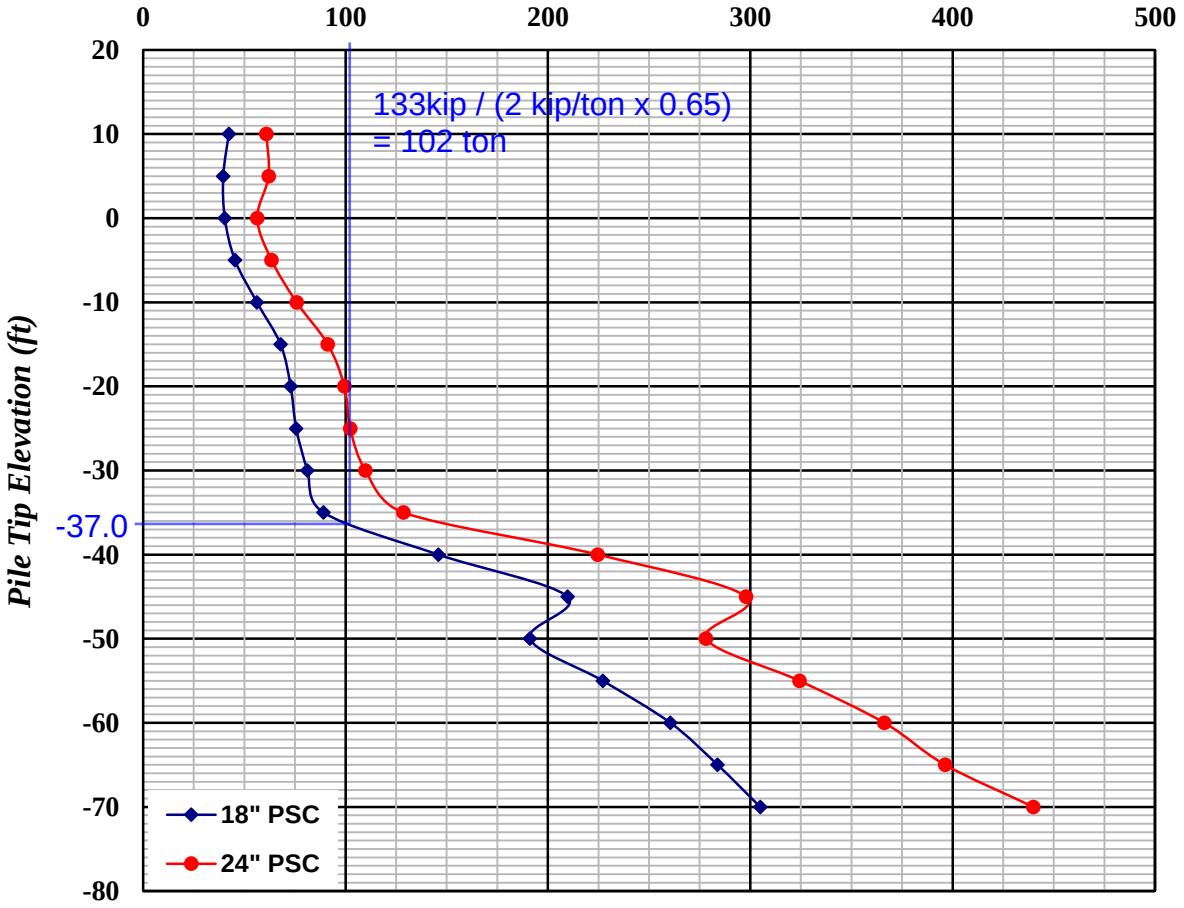
Able Canal Pathway Over Charlie Diversion Canal

Lee County, Florida

SPT Boring B-5

--- Precast Prestressed Concrete Piles ---

Estimated Nominal Bearing Resistance (tons)



LRFD Estimated Factored Axial Pile Capacities

Able Canal Pathway Over Charlie Diversion Canal

Lee County, Florida

DRAWN	SCALE	PROJ. NO.
ML	NOTED	21-33-4537
CHECKED	DATE	
IS	9/3/2021	PLATE 1

Ardaman & Associates, Inc.

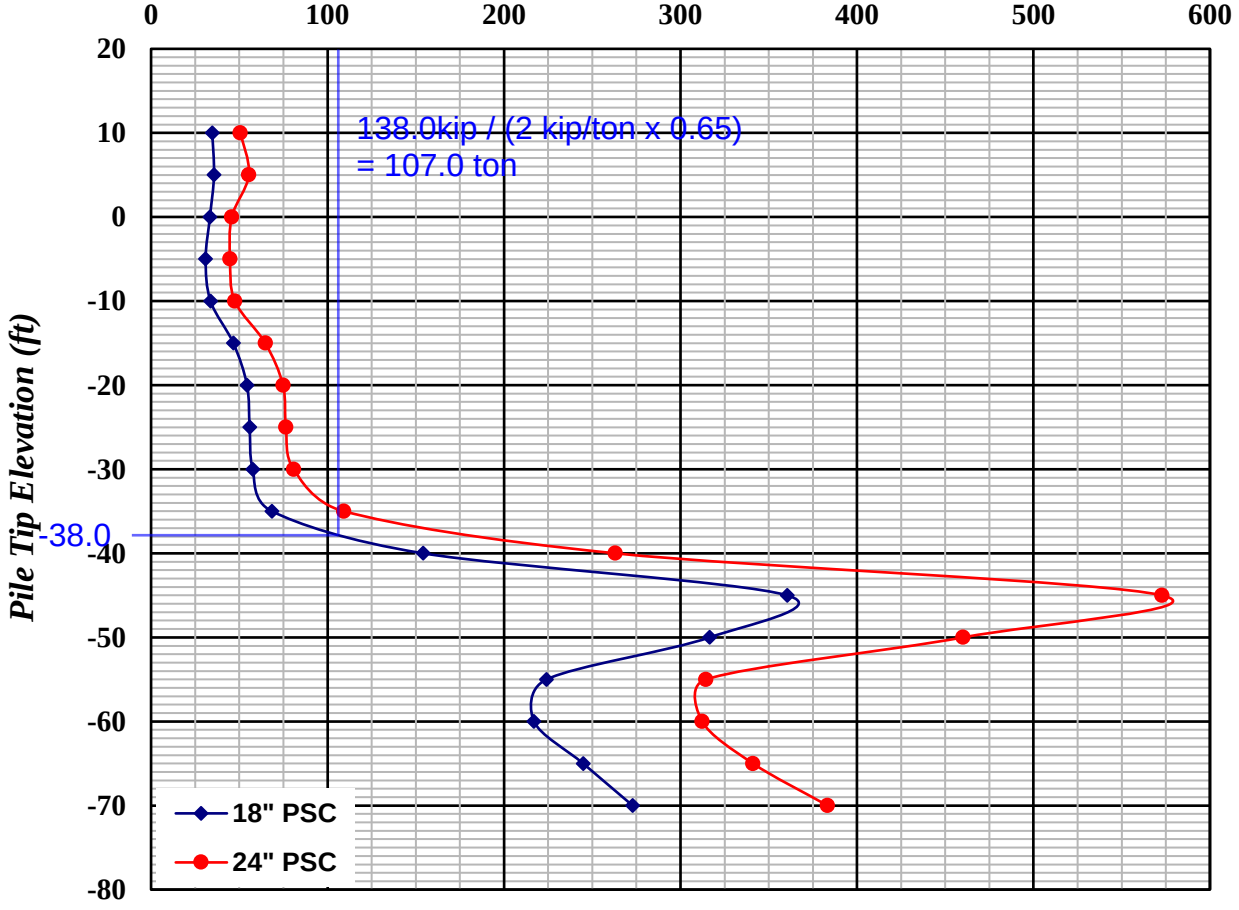
Able Canal Pathway Over Charlie Diversion Canal

Lee County, Florida

SPT Boring B-6

--- Precast Prestressed Concrete Piles ---

Estimated Nominal Bearing Resistance (tons)



LRFD Estimated Factored Axial Pile Capacities

Able Canal Pathway Over Charlie Diversion Canal

Lee County, Florida

DRAWN	SCALE	PROJ. NO.
ML	NOTED	21-33-4537
CHECKED	DATE	
IS	9/3/2021	PLATE 2

Ardaman & Associates, Inc.

LM 08/2022

FB-MultiPier

XML Report

XML File: FDL_Soil1_new scour 8.8.22.XML

Project Information

Version	5.9.0
Client	Lee County
Project Name	Able Canal Pathway
Project Manager	
Description	Intermediate Bent - FDL - Soil 1 - Updated Scour
Computed By	JHJ
Project Date	08/08/22

Control Information

Control Settings

Units	English	
Maximum Iterations	100	
Tolerance	1	(kip)
Soil Behavior	0	
user supplied P-Y Multipliers must be given		
Tip Spring Option	0	
no linear tip springs on piles		
Pile Tip Stiffness	0	(kip/in)
Analysis	Static	
Design Code	AASHTO-LRFD	

Output Summary - Substructure 1

Max/Min Pile Strains	Value		Load Case	Combination	Type	Segment	Pile
max strain in concrete	0.00016814	(in/in)	50	25	1	1	1
corresponding stress	0.5345	(ksi)	50	25	1	1	1
min strain in concrete	-0.00085596	(in/in)	50	25	1	1	1
corresponding stress	-3.132	(ksi)	50	25	1	1	1
max strain in reinforcement (prestressed)	0.0051247	(in/in)	57	32	1	1	3
corresponding stress	146.1	(ksi)	57	32	1	1	3
min strain in steel (prestressed)	0.0044773	(in/in)	50	25	1	1	1
corresponding stress	127.6	(ksi)	50	25	1	1	1

Max/Min Pier Column and Pier Cap Strains	Value		Load Case	Combination	Section	Element
max strain in concrete	5.2586E-05	(in/in)	55	30	3	6
corresponding stress	0.2329	(ksi)	55	30	3	6
min strain in concrete	-5.5514E-05	(in/in)	55	30	3	6
corresponding stress	-0.2458	(ksi)	55	30	3	6
max strain in steel (mild)	3.9711E-05	(in/in)	55	30	3	6
corresponding stress	1.152	(ksi)	55	30	3	6
min strain in steel (mild)	-4.2638E-05	(in/in)	55	30	3	6
corresponding stress	-1.237	(ksi)	55	30	3	6
max strain in concrete	2.3097E-05	(in/in)	28	3	4	13
corresponding stress	0.1023	(ksi)	28	3	4	13
min strain in concrete	-2.3156E-05	(in/in)	28	3	4	13
corresponding stress	-0.1025	(ksi)	28	3	4	13
max strain in steel (mild)	1.7588E-05	(in/in)	28	3	4	13

corresponding stress	0.51	(ksi)	28	3	4	13
min strain in steel (mild)	-1.7647E-05	(in/in)	28	3	4	13
corresponding stress	-0.5118	(ksi)	28	3	4	13

Max/Min Pile Internal Forces	Value		Load Case	Combination	Pile
max shear in 2 direction	11.98	(kip)	49	24	1
min shear in 2 direction	-11.98	(kip)	46	21	3
max shear in 3 direction	12.31	(kip)	48	23	3
min shear in 3 direction	-12.3	(kip)	51	26	1
max moment about 2 axis	124.7	(kip-ft)	51	26	3
min moment about 2 axis	-124.7	(kip-ft)	48	23	1
max moment about 3 axis	149.2	(kip-ft)	52	27	3
min moment about 3 axis	-149.2	(kip-ft)	55	30	1
max axial force	-1.908	(kip)	55	30	3
min axial force	-132.7	(kip)	28	3	2
max torsional force	0.375	(kip-ft)	56	31	3
min torsional force	-0.375	(kip-ft)	56	31	3
max demand/capacity ratio	0.637		52	27	3

Max/Min Soil Reaction Forces	Value		Load Case	Combination	Pile
max axial soil force	20.11	(kip)	28	3	2
min axial soil force	0.2444	(kip)	55	30	3
max lateral in X direction	4.926	(kip)	46	21	3
min lateral in X direction	-4.926	(kip)	49	24	1
max lateral in Y direction	4.683	(kip)	48	23	3
min lateral in Y direction	-4.682	(kip)	51	26	1
max torsional soil force	0.07262	(kip-ft)	56	31	3

Max/Min Pile Head Displacements	Value		Load Case	Combination	Pile
max displacement in axial	0.5548	(in)	28	3	2
min displacement in axial	0.0080645	(in)	55	30	3
max displacement in x	2.117	(in)	46	21	2
min displacement in x	-2.117	(in)	49	24	2
max displacement in y	2.814	(in)	48	23	3
min displacement in y	-2.814	(in)	51	26	1

Max/Min Pier Cap Internal Forces	Value		Load Case	Combination
max axial force	6.597	(kip)	55	30
min axial force	-8.879	(kip)	46	21
max shear in 2 direction	121.7	(kip)	28	3
min shear in 2 direction	-121.7	(kip)	28	3
max shear in 3 direction	2.97	(kip)	48	23
min shear in 3 direction	-3.145	(kip)	48	23
max torque	18.77	(kip-ft)	39	14
min torque	-18.77	(kip-ft)	39	14
max moment about 2 axis	1.852	(kip-ft)	48	23
min moment about 2 axis	-1.851	(kip-ft)	51	26
max moment about 3 axis	122.9	(kip-ft)	52	27
min moment about 3 axis	-151.1	(kip-ft)	52	27

LM 08/2022

FB-MultiPier

XML Report

XML File: FDL_Soil2_new scour 8.8.22.XML

Project Information

Version	5.9.0
Client	Lee County
Project Name	Able Canal Pathway
Project Manager	
Description	Intermediate Bent - FDL - Soil 2 - Updated Scour
Computed By	JHJ
Project Date	08/08/22

Control Information

Control Settings

Units	English	
Maximum Iterations	100	•
Tolerance	5	(kip) •
Soil Behavior	0	
user supplied P-Y Multipliers must be given		
Tip Spring Option	0	
no linear tip springs on piles		
Pile Tip Stiffness	0	(kip/in)
Analysis	Static	
Design Code	AASHTO-LRFD	

Output Summary - Substructure 1

Max/Min Pile Strains	Value		Load Case	Combination	Type	Segment	Pile
max strain in concrete	0.00014409	(in/in)	51	26	1	1	1
corresponding stress	0.5773	(ksi)	51	26	1	1	1
min strain in concrete	-0.00081984	(in/in)	51	26	1	1	1
corresponding stress	-3.026	(ksi)	51	26	1	1	1
max strain in reinforcement (prestressed)	0.0051142	(in/in)	54	29	1	1	1
corresponding stress	145.8	(ksi)	54	29	1	1	1
min strain in steel (prestressed)	0.0045007	(in/in)	49	24	1	1	1
corresponding stress	128.3	(ksi)	49	24	1	1	1

Max/Min Pier Column and Pier Cap Strains	Value		Load Case	Combination	Section	Element
max strain in concrete	5.5025E-05	(in/in)	55	30	3	6
corresponding stress	0.2437	(ksi)	55	30	3	6
min strain in concrete	-5.8015E-05	(in/in)	55	30	3	6
corresponding stress	-0.2569	(ksi)	55	30	3	6
max strain in steel (mild)	4.1561E-05	(in/in)	55	30	3	6
corresponding stress	1.205	(ksi)	55	30	3	6
min strain in steel (mild)	-4.4551E-05	(in/in)	55	30	3	6
corresponding stress	-1.292	(ksi)	55	30	3	6
max strain in concrete	2.4368E-05	(in/in)	50	25	4	13
corresponding stress	0.1079	(ksi)	50	25	4	13
min strain in concrete	-2.4457E-05	(in/in)	50	25	4	13
corresponding stress	-0.1083	(ksi)	50	25	4	13
max strain in steel (mild)	1.8574E-05	(in/in)	50	25	4	13

corresponding stress	0.5387	(ksi)	50	25	4	13
min strain in steel (mild)	-1.8663E-05	(in/in)	50	25	4	13
corresponding stress	-0.5412	(ksi)	50	25	4	13

Max/Min Pile Internal Forces	Value		Load Case	Combination	Pile
max shear in 2 direction	10.4	(kip)	46	21	3
min shear in 2 direction	-10.4	(kip)	49	24	1
max shear in 3 direction	8.574	(kip)	48	23	3
min shear in 3 direction	-8.575	(kip)	51	26	1
max moment about 2 axis	125.2	(kip-ft)	51	26	3
min moment about 2 axis	-125.2	(kip-ft)	48	23	1
max moment about 3 axis	155.8	(kip-ft)	52	27	3
min moment about 3 axis	-155.8	(kip-ft)	55	30	1
max axial force	-5.61	(kip) •	55	30	3
min axial force	-138.3	(kip) •	49	24	1
max torsional force	0.4529	(kip-ft)	53	28	1
min torsional force	-0.4529	(kip-ft)	53	28	1
max demand/capacity ratio	0.6656		52	27	3

Max/Min Soil Reaction Forces	Value		Load Case	Combination	Pile
max axial soil force	75.15	(kip)	49	24	1
min axial soil force	0.2944	(kip)	55	30	3
max lateral in X direction	4.707	(kip)	46	21	3
min lateral in X direction	-4.707	(kip)	49	24	1
max lateral in Y direction	4.121	(kip)	48	23	1
min lateral in Y direction	-4.121	(kip)	51	26	3
max torsional soil force	0.08	(kip-ft)	53	28	1

Max/Min Pile Head Displacements	Value		Load Case	Combination	Pile
max displacement in axial	0.2884	(in)	49	24	1
min displacement in axial	0.008094	(in)	55	30	3
max displacement in x	2.085	(in)	46	21	2
min displacement in x	-2.085	(in)	49	24	2
max displacement in y	2.987	(in)	48	23	3
min displacement in y	-2.987	(in)	51	26	1

Max/Min Pier Cap Internal Forces	Value		Load Case	Combination
max axial force	6.464	(kip)	55	30
min axial force	-8.945	(kip)	46	21
max shear in 2 direction	127.5	(kip)	49	24
min shear in 2 direction	-127.5	(kip)	46	21
max shear in 3 direction	2.941	(kip)	51	26
min shear in 3 direction	-3.16	(kip)	51	26
max torque	18.78	(kip-ft)	40	15
min torque	-18.78	(kip-ft)	40	15
max moment about 2 axis	1.867	(kip-ft)	48	23
min moment about 2 axis	-1.867	(kip-ft)	51	26
max moment about 3 axis	132.1	(kip-ft)	49	24
min moment about 3 axis	-157.7	(kip-ft)	46	21

5. Reinforcing Bar Lists

5.1. Approach Slabs

LM 03/2022

REINFORCING STEEL QUANTITIES

DATE RAN: MON MAR 07 08:19:32 2022

NAME OF UNIT	QUANTITY/UNIT	NO. UNITS	TOTAL-QUANTITY	COST/LB	TOTAL-COST
APPROACH SLABS 1 & 2	989 LBS X	2 =	1978 LBS AT	0.000 = $	0.00
		GRAND TOTAL =	1978 LBS	$	0.00

LOCATION APPROACH SLABS 1 & 2 • NO. REQUIRED = 2 • LBS/MARK

5	A1 •	9- 8	15 • 1 •	9- 8 •	151.22
8	A2 •	9- 8	19 • 1 •	9- 8 •	490.36
5	B1 •	13- 4	25 • 1 •	13- 4 •	347.66

5.2. Superstructure

LM 11/2023

KISINGER CAMPO & ASSOCIATES

LEE COUNTY
ABLE CANAL PATHWAY

SUPERSTRUCTURE REBAR LIST

REINFORCING STEEL QUANTITIES

DATE RAN: THU NOV 30 14:17:58 2023

NAME OF UNIT	QUANTITY/UNIT	NO. UNITS	TOTAL-QUANTITY	COST/LB	TOTAL-COST
SPANS 1 THROUGH 5	4418 LBS X	5 =	22090 LBS AT	0.000 =	$ 0.00
	GRAND TOTAL =		22090 LBS		$ 0.00

					LOCATION	SPANS 1 THROUGH 5	NO. REQUIRED = 5	LBS/MARK
5	S01	52- 6	20	1	52- 6			1095.15
5	S02	13- 3	108	1	13- 3			1492.53
5	S03	4- 7	206	4 4 4	1- 0	0- 9 1/2		984.48
5	S04	52- 6	8	1	52- 6			438.06
4	S05	2- 9	36	10	1- 6	1- 3		66.13
4	S06	2- 5	212	10	1- 6	0-11		342.14

5.3. Substructure

LM 03/2022

REINFORCING STEEL QUANTITIES

DATE RAN: MON MAR 07 08:48:39 2022

NAME OF UNIT	QUANTITY/UNIT	NO. UNITS	TOTAL-QUANTITY	COST/LB	TOTAL-COST
END BENT 1 AND 6	849 LBS X	2 =	1698 LBS AT	0.000 = $	0.00
INT. BENTS 2 THROUGH 5	350 LBS X	4 =	1400 LBS AT	0.000 = $	0.00
	GRAND TOTAL =		3098 LBS	$	0.00

LOCATION END BENT 1 AND 6 NO. REQUIRED = 2

								LBS/MARK
6	D01	3- 3	5 23	1- 4	0- 2 1/4	1- 4		24.43
4	E02	1- 7	28 1	1- 7				29.61
4	E03	15- 3	2 1	15- 3				20.37
6	E04	15- 3	16 1	15- 3				366.49
6	E05	10- 0	2 18 3 3	8- 0				30.04
4	E10	10- 7	12 4 4 4	1-11	3- 0			84.83
4	E11	7-10	4 5	1-11	3- 0	0- 6	0- 6	20.93
4	E31	2-11	10 17 3	2- 3				19.49
4	E32	2-11	12 17 3	2- 3				23.38
4	E33	6- 6	32 1	6- 6				138.94
4	E34	3- 9	36 1	3- 9				90.18

LOCATION INT. BENTS 2 THROUGH 5 NO. REQUIRED = 4

								LBS/MARK
5	E01	13- 2	14 1	13- 2				192.25
4	E02	4- 5	6 18 3 3	3- 1				17.70
4	E03	11- 1	12 4 4 4	2- 0	3- 2			88.83
4	E04	8- 4	4 5	2- 0	3- 2	0- 7	0- 7	22.26
4	E05	4- 4	10 18 3 3	3- 0				28.94

6. Gravity Wall

7.0 WALLS

CALCULATION QUALITY CONTROL		
RESPONSIBLE PROFESSIONAL / ORIGINATOR	INITIALS: JHJ	DATE: 08/22
QUALITY REVIEWER	INITIALS: LM	DATE: 08/22

Calculations included in this section are as follows:

7.1 Gravity Wall GW-1 Elevations

SUMMARY OF WALL DESIGN

Wall geometry is set using a 6H:1V backfill slope and the existing groundline elevation. Geopak was utilized to obtain the existing groundline elevation. Excel was utilized to calculate elevations and quantities.

Wall quantities are tabulated as Scheme 2 to account for maintenance vehicle loading.

KISINGER CAMPO & ASSOCIATES

ABLE CANAL PATHWAY
LEE COUNTY
FPID 435351-2-58-01

DESIGNED BY: JHJ 08/22
CHECKED BY: LM 08/2022

Gravity Wall GW-1

Vertical Alignment Along Baseline Able (BL_ABLE_11)

	Station	Elevation	Grade 1 =	-0.199%
PC	445+66.00	19.132	Grade 2 =	3.250%
PI	445+96.00	19.072	G2-G1=	3.449%
PT	446+26.00	20.047	LVC =	60.000

Vertical Alignment Along Baseline Able (BL_ABLE_11)

	Station	Elevation	Grade 1 =	3.250%
PC	446+75.00	21.639	Grade 2 =	0.200%
PI	447+05.00	22.614	G2-G1=	-3.050%
PT	447+35.00	22.674	LVC =	60.000

Gravity Wall GW-1 Elevations

Station Along PGL	Offset to FFRW from PGL (ft.)	PGL Elev. (ft.)	Length Along Wall (ft.)	Back-Slope (ft/ft)	Cross-Slope (ft/ft)	Proposed Ground Elev. (ft.)	Existing Ground Elev. (ft.)	Top of Wall Elev. (ft.)	Retained Height (ft.)	Bottom of Wall Elev. (ft.)	Total Wall Height (ft.)
445+68.00	7.83	19.129	0.000	-16.667%	-0.020%	18.268	18.276	18.268	0.000	17.276	0.992
445+80.00	7.98	19.160	12.001	-16.667%	-0.020%	18.273	18.013	18.273	0.260	17.013	1.260
446+00.00	8.24	19.396	32.003	-16.667%	-0.020%	18.467	17.787	18.467	0.680	16.787	1.680
446+20.00	8.49	19.862	52.004	-16.667%	-0.020%	18.891	17.518	18.891	1.373	16.518	2.373
446+40.00	8.75	20.502	72.006	-16.667%	-0.020%	19.488	17.295	19.488	2.193	16.295	3.193
446+60.00	9.00	21.152	92.007	-16.667%	-0.020%	20.096	17.223	20.096	2.873	16.223	3.873
446+80.00	9.26	21.796	112.009	-16.667%	-0.020%	20.697	17.056	20.697	3.641	16.056	4.641
447+00.00	10.25	22.293	133.156	-16.667%	-0.020%	21.029	19.854	21.029	1.175	18.854	2.175
447+09.00	11.99	22.451	143.303	-16.667%	-0.020%	20.897	20.958	20.897	0.000	19.958	0.939

Value obtained from Geopak.

Note: Vertical alignment data obtained from 1.2_Vertical Profile.pdf.

KISINGER CAMPO & ASSOCIATES

ABLE CANAL PATHWAY
LEE COUNTY
FPID 435351-2-58-01

DESIGNED BY: JHJ 08/22
CHECKED BY: LM 08/2022

Gravity Wall GW-1

Quantities - Scheme 2			
Volume/ft	Volume	Steel/ft	Steel
CY/ft	CY	LB/ft	LB
0.032	0.000	2.000	0.000
0.053	0.510	2.260	25.564
0.085	1.378	2.680	49.411
0.144	2.288	3.373	60.537
0.223	3.668	4.193	75.666
0.305	5.280	4.873	90.668
0.416	7.210	5.641	105.149
0.126	5.730	3.175	93.215
0.032	0.802	2.000	26.255

26.9 **CY** 527 **LB**

7. Quantities

KISINGER CAMPO & ASSOCIATES

LEE COUNTY — DESIGNED BY: JHJ 08/22

ABLE CANAL PATHWAY — CHECKED BY: LM 11/2023

Foundation Quantities

Able Canal Pathway over Charlie Diversion Canal

0455 34 3 PRESTRESSED CONCRETE PILING, 18" SQ

Location	No. Piles *(ea.)*	Avg. Cutoff Elev. *(ft.)*	Pile Tip Elev. *(ft.)*	Length *(ft.)*	Pile Length *(ft.)*
End Bent 1	2	19.4	-40 Cons	59	119
Bent 2	2	19.5	-38	57	115
Bent 3	3	19.6	-38	58	173
Bent 4	2	19.5	-38	58	115
Bent 5	3	19.5	-38	57	172
End Bent 6	1	19.4	-40 Cons	59	59

PAY ITEM TOTAL (LF) 753

Note: End bent pile tip determind by Avg. Cutoff Elevation - (Test Pile Length - 15ft.)

Applicable Equation: Pile Length = No. Piles x (Cutoff Elev. - Pile Length)

0455143 3 TEST PILES-PRESTRESSED CONCRETE,18" SQ

Location	No. Piles *(ea.)*	Test Pile Length *(ft.)*
End Bent 1	0	0
Bent 2	1	75
Bent 3	0	0
Bent 4	1	75
Bent 5	0	0
End Bent 6	1	74

PAY ITEM TOTAL (LF) 224

Applicable Equation: Pile Length = No. Piles x Test Pile Length

KISINGER CAMPO & ASSOCIATES

LEE COUNTY — DESIGNED BY: JHJ 08/22

ABLE CANAL PATHWAY — CHECKED BY: LM 11/2023

Substructure Quantities

Able Canal Pathway over Charlie Diversion Canal

0400 4 5 • CLASS IV CONCRETE, SUBSTRUCTURE •

End Bents					
Location	Length *(ft.)*	Width *(ft.)*	Height *(ft.)*	Quantity	Volume *(CY)*
Cap	13.75 •	3.50 •	2.50 •	2 •	8.91 •
Backwall	13.75 •	1.00 •	0.50 •	2 •	0.51 •
LT Wingwall	7.00 •	1.00 •	4.42 OK	2 •	2.29 •
RT Wingwall	7.00 •	1.00 •	4.42 OK	2 •	2.29 •

Pile Voids					
Location	Length *(ft.)*	Width *(ft.)*	Height *(ft.)*	Quantity	Volume *(CY)*
Cap	1.50 •	1.50 •	1.00 •	4 •	0.33 •

TOTAL (CY) 13.7 •

Intermediate Bents					
Location	Length *(ft.)*	Width *(ft.)*	Height *(ft.)*	Quantity	Volume *(CY)*
Cap	13.67 •	3.67 •	2.50 •	4 •	18.56 •

Pile Voids					
Location	Length *(ft.)*	Width *(ft.)*	Height *(ft.)*	Quantity	Volume *(CY)*
Cap	1.50 •	1.50 •	1.00 •	12 •	1.00 •

TOTAL (CY) 17.6 •

Summary	
Location	Volume *(CY)*
End Bent 1	6.9 •
Bent 2	4.4 •
Bent 3	4.4 •
Bent 4	4.4 •
Bent 5	4.4 •
End Bent 6	6.9 •

PAY ITEM TOTAL (CY) 31.4 •

Applicable Equation: Volume = Quantity x (Length x Width x Height) / (27 ft^3/CY)

KISINGER CAMPO & ASSOCIATES

LEE COUNTY | DESIGNED BY: JHJ 08/22
ABLE CANAL PATHWAY | CHECKED BY: LM 11/2023

Substructure Quantities

Able Canal Pathway over Charlie Diversion Canal

0415 1 5 • REINFORCING STEEL - BRIDGE SUBSTRUCTURE •

Location	Weight *(lb.)*
End Bent 1 •	849 •
Int. Bent 2 •	350 •
Int. Bent 3 •	350 •
Int. Bent 4 •	350 •
Int. Bent 5 •	350 •
End Bent 6 •	849 •

PAY ITEM TOTAL (LB) 3098 •

KISINGER CAMPO & ASSOCIATES

LEE COUNTY — DESIGNED BY: JHJ 08/22

ABLE CANAL PATHWAY — CHECKED BY: LM 11/2023

Superstructure Quantities

Able Canal Pathway over Charlie Diversion Canal

0400 2 47 • CONCRETE CLASS II, CAST-IN-PLACE TOPPING WITH SHRINKAGE REDUCING ADMIXTURE •

Deck Topping - Span 1 and 5								
Location	Span *(ft.)*	Bridge Width *(ft.)*	Dim. B & D *(ft.)*	Dim. C *(ft.)*	Gap Width *(in.)*	FSB Depth *(in.)*	No. of FSB *(ea.)*	Volume *(CY)*
Above FSBs	53.08	13.58	0.59	0.50	1.00	15	3	13.92 •
Interior Pockets	53.08	13.58	0.59	0.50	1.00	15	3	3.83 •
Exterior Pockets	53.08	13.58	0.59	0.50	1.00	15	3	1.62 •
Top Fillets	53.08	13.58	0.59	0.50	1.00	15	3	0.16 •
End Diaphragms	53.08	13.58	0.59	0.50	1.00	15	3	0.93 •

TOTAL PER SPAN (CY)	20.5 •
NO. OF SPANS	2 •
SUB TOTAL (CY)	41.0 •

Deck Topping - Span 2 through 4								
Location	Span *(ft.)*	Bridge Width *(ft.)*	Dim. B & D *(ft.)*	Dim. C *(ft.)*	Gap Width *(in.)*	FSB Depth *(in.)*	No. of FSB *(ea.)*	Volume *(CY)*
Above FSBs	53.00	13.58	0.59	0.50	1.00	15	3	13.90 •
Interior Pockets	53.00	13.58	0.59	0.50	1.00	15	3	3.83 •
Exterior Pockets	53.00	13.58	0.59	0.50	1.00	15	3	1.62 •
Top Fillets	53.00	13.58	0.59	0.50	1.00	15	3	0.16 •
End Diaphragms	53.00	13.58	0.59	0.50	1.00	15	3	0.93 •

TOTAL PER SPAN (CY)	20.4 •
NO. OF SPANS	3 •
SUB TOTAL (CY)	61.2 •

Location	No. Railings	No. Spans	Span *(ft.)*	Width *(ft.)*	Height *(ft.)*	Volume *(CY)*
Railing Curb	2 •	2 •	53.08 •	0.67 •	0.50 •	2.60
	2 •	3 •	53.00 •	0.67 •	0.50 •	3.90

SUB TOTAL (CY)	6.5 •

KISINGER CAMPO & ASSOCIATES

LEE COUNTY — DESIGNED BY: JHJ 08/22

ABLE CANAL PATHWAY — CHECKED BY: LM 11/2023

Superstructure Quantities

Able Canal Pathway over Charlie Diversion Canal

0400 2 47 •CONCRETE CLASS II, CAST-IN-PLACE TOPPING WITH SHRINKAGE REDUCING ADMIXTURE •

Summary	
Location	Volume *(CY)*
Span 1	21.8 •
Span 2	21.7 •
Span 3	21.7 •
Span 4	21.7 •
Span 5	21.8 •

Applicable Equations:

Deck Topping [Reference SPI 450-450]

PAY ITEM TOTAL (CY) 108.7 •

0400 148 • PLAIN NEOPRENE BEARING PADS •

Location	Length *(in.)*	Width *(in.)*	Thickness *(in.)*	No. *(ea.)*	Volume *(CF)*
Span 1	42	8	1	6	1.2 •
Span 2	42	8	1	6	1.2 •
Span 3	42	8	1	6	1.2 •
Span 4	42	8	1	6	1.2 •
Span 5	42	8	1	6	1.2 •

PAY ITEM TOTAL (CF) 6.0 •

0415 1 4 • REINFORCING STEEL - BRIDGE SUPERSTRUCTURE •

Location	Weight *(lb.)*
Span 1	4418 •
Span 2	4418 •
Span 3	4418 •
Span 4	4418 •
Span 5	4418 •

PAY ITEM TOTAL (LB) 22090 •

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ABLE CANAL PATHWAY — CHECKED BY: LM 11/2023

Superstructure Quantities

Able Canal Pathway over Charlie Diversion Canal

0450 8 22 • PRESTRESSED BEAM: FLORIDA SLAB BEAM, BEAM DEPTH 15" CARBON STEEL, WIDTH 52-54" •

Span	Length *(ft.)*	No. Beams *(ea.)*	Length *(ft.)*
1	51.83	3	156 •
2	51.83	3	156 •
3	51.83	3	156 •
4	51.83	3	156 •
5	51.83	3	156 •

PAY ITEM TOTAL (LF) 780 •

0458 1 11 • BRIDGE DECK EXPANSION JOINT, NEW CONSTRUCTION, F&I POURED JOINT WITH BACKER ROD •

Location	Width *(ft.)*	Bridge Skew *(deg.)*	Length *(ft.)*
End Bent 1	13.58	0.00	14 •
Bent 2	13.58	0.00	14 •
Bent 3	13.58	0.00	14 •
Bent 4	13.58	0.00	14 •
Bent 5	13.58	0.00	14 •
End Bent 6	13.58	0.00	14 •

PAY ITEM TOTAL (LF) 84 •

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LEE COUNTY
ABLE CANAL PATHWAY

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Approach Slab Quantities

Able Canal Pathway over Charlie Diversion Canal

0400 2 10 CLASS II CONCRETE, APPROACH SLABS

Location	Length *(ft.)*	Width *(ft.)*	Depth - Slab *(ft.)*	Depth - Topping *(ft.)*	Depth - To Cap *(ft.)*	Volume *(CY)*
Approach Slab 1	10.00	13.58	1.00	0.00	0.42	5.3
Approach Slab 2	10.00	13.58	1.00	0.00	0.42	5.3

SUB TOTAL (CY) 10.6

Location	No. Railings	No. Spans	Span *(ft.)*	Width *(ft.)*	Height *(ft.)*	Volume *(CY)*
Railing Curb	1	1	10.00	0.67	0.50	0.10
	1	1	10.00	0.67	0.50	0.10

SUB TOTAL (CY) 0.2

PAY ITEM TOTAL (CY) 10.8

Applicable Equation: AS Volume = (Length x Width x Depth Slab + 2-ft x Width x Depth Topping + Width x Depth To Cap x (1-ft + 0.5 x Depth To Cap)) / (27 ft^3/CY)

0415 1 9 REINFORCING STEEL - APPROACH SLABS

Location	Weight *(lb.)*
Approach Slab 1	989
Approach Slab 2	989

PAY ITEM TOTAL (LB) 1978

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ABLE CANAL PATHWAY

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Barrier Quantities

Able Canal Pathway over Charlie Diversion Canal

0515 2211 PEDESTRIAN / BICYCLE RAILING, STEEL, 42" TYPE 1

Location	Length *(ft.)*	No. Railings	Length *(ft.)*
Approach Slab 1	10.00	1	10.00
Bridge	265.17	2	531.00
Approach Slab 2	10.00	1	10.00

PAY ITEM TOTAL (LF) 551

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RipRap Quantities

Able Canal Pathway over Charlie Diversion Canal

LC01 FLOATING BOOM BARRIER

Location	Total Length *(LF)*
Canal	300.0

PAY ITEM TOTAL (LF) 300

0530 3 3 RIPRAP - RUBBLE, BANK AND SHORE

Location	Berm Area *(ft^2)*	Toe Area *(ft^2)*	Slope Area *(ft^2)*	Slope Factor *	Total Area *(ft^2)*	Thickness *(ft)*	Volume *(ft^3)*
End Bent 1	243	245	833	1.12	1420.69	2.50	3551.7
E.B. 1 over Sand Cement	23	0	0	0.00	23.00	0.50	11.5
End Bent 6	110	347	818	1.12	1373.26	2.50	3433.1
E.B. 6 over Sand Cement	23	0	0	0.00	23.00	0.50	11.5

*Based on 1:2 slope = (1^2 +2^2)^0.5 / 2 = 1.12

S.G.	2.3	
W.w	62.4	*(lb/ft^3)*
Vf	0.9	

Bent #	Weight *(TON)*
End Bent 1	230.1
End Bent 6	222.5

PAY ITEM TOTAL (TON) 452.6

Applicable Equation: Weight = (Volume x S.G. x Ww + Vf)/2000lb/ton

0530 74 BEDDING STONE

Location	Berm Area *(ft^2)*	Toe Area *(ft^2)*	Slope Area *(ft^2)*	Slope Factor *	Total Area *(ft^2)*	Thickness *(ft)*	Volume *(ft^3)*
End Bent 1	243	245	833	1.12	1420.69	1.00	1420.7
End Bent 6	110	347	818	1.12	1373.26	1.00	1373.3

*Based on 1:2 slope = (1^2 +2^2)^0.5 / 2 = 1.12

Unit Weight 115 *(lb/ft^3)*

Bent #	Weight *(TON)*
End Bent 1	81.7
End Bent 6	79.0

PAY ITEM TOTAL (TON) 160.7

Applicable Equation: Weight = (Volume x Unit Weight)/2000

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RipRap Quantities

Able Canal Pathway over Charlie Diversion Canal

0530 1100 RIPRAP, SAND-CEMENT BAGS

Location	Length *(ft)*	Height *(ft)*	Width *(ft)*	Volume *(CY)*
End Bent 1	23.0	4.00	1.00	3.5
End Bent 6	23.0	4.00	1.00	3.5

PAY ITEM TOTAL (CY) 7.0

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Wall Quantities

Able Canal Pathway over Charlie Diversion Canal

0400 0 11 • CONCRETE CLASS NS, GRAVITY WALL •

Location	Volume *(CY)*
GW-1 •	26.90 •

PAY ITEM TOTAL (CY) 26.9 •

8. Estimate

LM 11/2023

COST ESTIMATE

for

Able Canal Pathway over Charlie Diversion Canal

Lee County

PROJECT ID. 435351-2

PHASE: FINAL STRUCTURES

Item Number	Item Description	Quantity	Unit	Unit Price	Total Amount Per Item
Able Canal Pathway over Charlie Diversion Canal					
	Lump Sum Items				
0108 1	MONITOR EXISTING STRUCTURES- INSPECTION AND SETTLEMENT MONITORING	1	EA	$20,000.00	$20,000.00
0108 2	MONITOR EXISTING STRUCTURES- VIBRATION MONITORING	1	EA	$15,000.00	$15,000.00
				SUBTOTAL	$35,000.00
	Substructure				
0400 4 5	CONCRETE CLASS IV, BRIDGE SUBSTRUCTURE	31.4	CY	$2,500.00	$78,500.00
0415 1 5	REINFORCING STEEL- BRIDGE SUBSTRUCTURE	3098	LB	$2.00	$6,196.00
0455 34 3	PRESTRESSED CONCRETE PILING, 18" SQ	753	LF	$180.00	$135,540.00
0455143 3	TEST PILES-PRESTRESSED CONCRETE,18" SQ	224	LF	$460.00	$103,040.00
				SUBTOTAL	$323,276.00
	Slope Protection				
0530 1100	RIPRAP, SAND-CEMENT BAGS	7.0	CY	$750.00	$5,250.00
0530 3 3	RIPRAP- RUBBLE, BANK AND SHORE	452.6	TN	$200.00	$90,520.00
0530 74	BEDDING STONE	160.7	TN	$250.00	$40,175.00
LC01	FLOATING BOOM BARRIER	300.0	LF	$335.00	$100,500.00
				SUBTOTAL	$236,445.00
	Superstructure				
0400 2 10	CONCRETE CLASS II, APPROACH SLABS	10.8	CY	$1,000.00	$10,800.00
0400 2 47	CONCRETE CLASS II, CAST-IN-PLACE TOPPING WITH SHRINKAGE REDUCING ADMIXTURE	108.7	CY	$2,050.00	$222,835.00
0400148	PLAIN NEOPRENE BEARING PADS	6.0	CF	$1,980.00	$11,880.00
0415 1 4	REINFORCING STEEL - BRIDGE SUPERSTRUCTURE	22090	LB	$2.00	$44,180.00
0415 1 9	REINFORCING STEEL- APPROACH SLABS	1978	LB	$2.00	$3,956.00
0450 8 22	PRESTRESSED BEAM: FLORIDA SLAB BEAM, BEAM DEPTH 15" CARBON STEEL, WIDTH 52-54"	780	LF	$600.00	$468,000.00
0458 1 11	BRIDGE DECK EXPANSION JOINT, NEW CONSTRUCTION, F&I POURED JOINT WITH BACKER ROD	84	LF	$80.00	$6,720.00
0515 2211	PEDESTRIAN / BICYCLE RAILING, STEEL, 42" TYPE 1	551	LF	$150.00	$82,650.00
				SUBTOTAL	$851,021.00
	Retaining Walls				
0400 0 11	CONCRETE CLASS NS, GRAVITY WALL INDEX 400-011	26.9	CY	$2,000.00	$53,800.00
				SUBTOTAL	$53,800.00
	Conditional Variables				
	CONSTRUCTION OVER WATER (PERCENTAGE OF SUPERSTRUCTURE AND SUBSTRUCTURE)	$1,174,297.00		8.0%	$93,944.00
				SUBTOTAL	$93,944.00
	TOTAL				**$1,593,486.00**