EMH&T
Engineers, Surveyors, Planners, Scientists

MEMORANDUM

Date: June 15th, 2020

To: Tim Stillion, PE – ODOT District 11

From: EMH&T

Subject: BEL-7-1187C (SFN 0700312) – Steel Pier Cap Load Rating

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INTRODUCTION

The existing BEL-7-1187C (SFN 0700312) bridge pair facilitates a seven span continuous steel structure with rolled steel beams (fy=36 ksi) and a non-composite reinforced concrete deck. The bridge was constructed in 1983 and designed to accommodate HS20-44 and Alternate Military vehicular live load. Due to the high-skew railroad crossing, the substructure units were constructed with a variety of geometries and support configurations, including: straddle-bent steel boxes, cantilever tapered steel box caps, and reinforced concrete T-type pier caps.

Pier locations 1 and 7 are reinforced concrete hammerhead supports while the remaining Piers 2 through 6 are comprised of welded steel box girder sections. The following figure depicts the pier nomenclature used in the analysis:

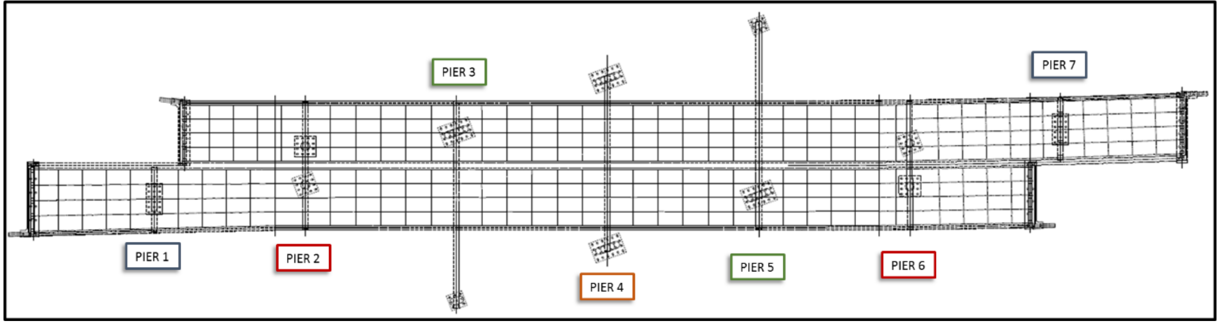


Figure A: Pier Nomenclature

As the current BEL-7-1152 (PID 110324) project scope proposes a new 3" asphalt overlay to the existing traveled way, EMH&T has been contracted to evaluate the structural adequacy through standard load rating procedures for both the superstructure and intermediate substructure (pier) spanning elements. Due to the presence of a rigid overlay and proposed flexible overlay, in conjunction with the understanding that the deck will be replaced in a reasonable timeline, an additional 60 psf future wearing surface has not currently been included in the rating of this structure.

The load ratings were performed per the 2020 ODOT Bridge Design Manual (BDM) Section 900 for current HL-93 vehicular and Ohio Legal loads. Since the steel box girder pier caps are classified as fracture critical components, additional fatigue criteria was considered in the rating of these members.

APPROACH

The load ratings of the superstructure and substructure were completed with two software packages as to better meet ODOT standards and structural accuracy. Record plan and inspection information provided by ODOT District 11 was used to model the existing bridge pair within Midas Civil and AASHTOWare BrR. The Midas finite element analysis software was used to model the entire superstructure and substructure system, including individual cap properties and material specifications. This methodology was able to more accurately model the design loading and distribution based on individual pier cap stiffness interactions along the length of the bridges. Once the applied loading was generated from the modeling software, it was analyzed in a separate spreadsheet to determine member capacities and subsequent rating factors. Following the rating of the substructure components, an AASHTOWare BrR model was created to verify the loading as well as generate superstructure rating factors for the main longitudinal members as this is the preferred software for rating bridges in Ohio.

SUBSTRUCTURE ANALYSIS

Design loading used for the substructure rating has been generated from Midas to analyze each pier cap. Load and geometry inputs have been derived from record plan information and material specifications. As stated in the 2019 inspection report and from the April 2020 site visit performed by the design team, all steel members appear to be in overall good condition with no visible signs of section loss. Because of this, the original dimensions shown in the record plans have been utilized for analysis and capacity calculations. The existing rolled WF member geometries have also been derived from the archived 1966 AISC manual for ASTM A36 steel members applicable at the time.

Applied dead loads have been calculated based on record plan geometry and material densities specified in BDM Section 909. The existing deck and 1.5" Latex Modified Concrete (LMC) overlay have been analyzed as DC1 loads and distributed based on tributary width to each beam. Member selfweight includes a 10% allowance for splices, cover plates, stiffeners, etc. Barriers have been analyzed as a DC2 load acting on the non-composite steel section and distributed uniformly to each beam per AASHTO 4.6.2.2. The proposed 3" asphalt wearing surface is applied as a DW load with a 1.5 load factor. As previously stated, additional 60 psf future wearing surfaces have been omitted for this analysis considering the current overlay thicknesses at the site and likelihood for deck replacement prior to additional overlays being performed.

Within Midas, each bridge assumes 3 possible lane locations with potential vehicular float in each lane to maximize the live load effects. The existing toe/toe of barrier is 34.25' which implies a maximum number of two 12' lanes per AASHTO guidelines. However, considering the near 36' width a theoretical third lane has been included to conservatively place traffic within the outside shoulder to maximize the negative moments at the pier cap cantilevered ends. The model also conservatively assumes bidirectional lane orientation/traffic flow along the bridge to capture all potential axle positions.

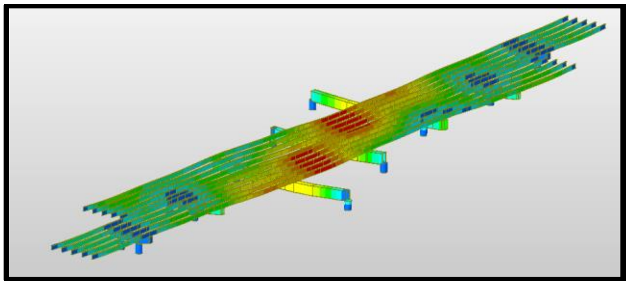


Figure B: Midas Deformed Shape

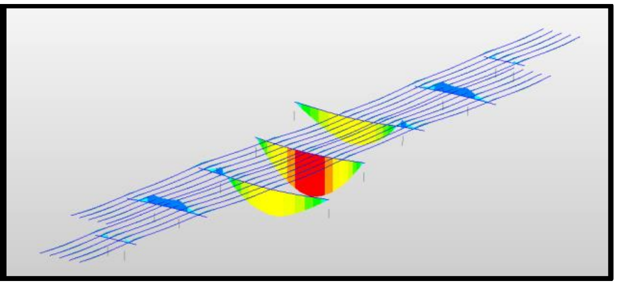


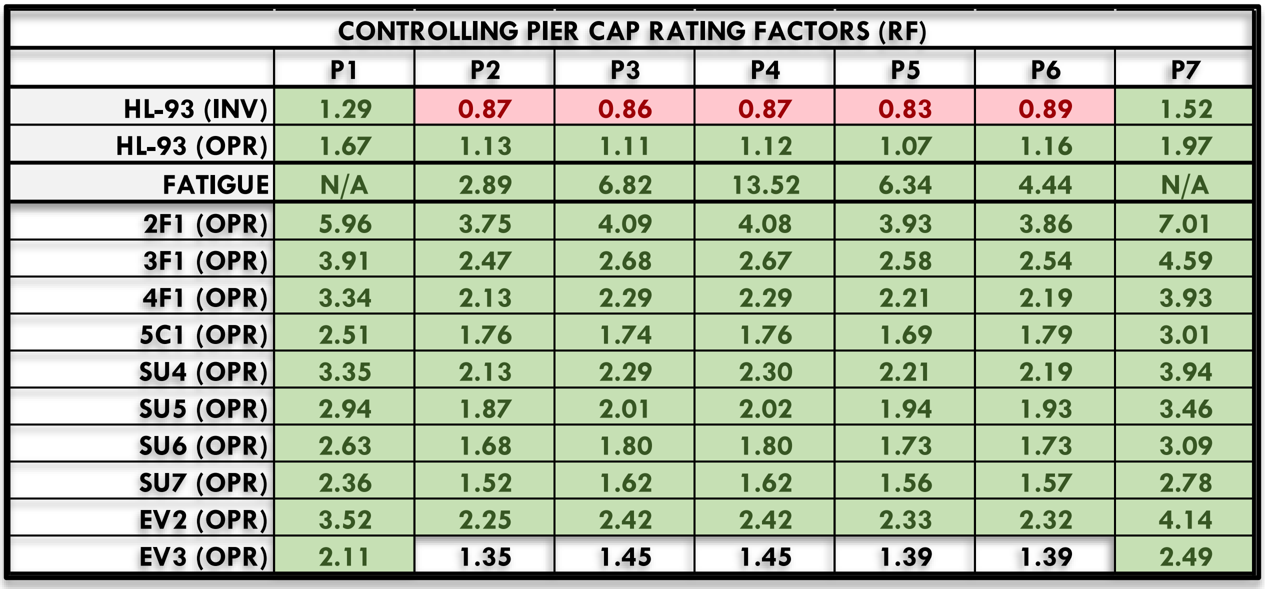
Figure C: Midas Moment Diagram

SUBSTRUCTURE SUMMARY

Capacity calculations are based on the current AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017. These capacities in addition to the provisions of BDM Section 900 have been used for the determination of the rating factors.

All steel pier caps have been treated as non-redundant features and include a 1.05 load modifier per current AASHTO guidelines. The limit states and load factors depicted in BDM Tables 925-1 and 925-2 have also been included in the analysis. It should be noted that a 1.10 load factor has been applied to the EV3 vehicle for STR I as the bridge is not located on an interstate route.

The table below depicts the rating factors determined through the structural analysis of all pier types. Each analysis event considers the unit capacity along the member's entire length and compares it against the loading within a user specified load combination (see **Figure E** for illustration). It was determined in most cases that the bearing stiffeners or flexural capacity of the pier cap controls the rating factors.



CONTROLLING PIER CAP RATING FACTORS (RF)							
	P1	**P2**	**P3**	**P4**	**P5**	**P6**	**P7**
HL-93 (INV)	1.29	0.87	0.86	0.87	0.83	0.89	1.52
HL-93 (OPR)	1.67	1.13	1.11	1.12	1.07	1.16	1.97
FATIGUE	N/A	2.89	6.82	13.52	6.34	4.44	N/A
2F1 (OPR)	5.96	3.75	4.09	4.08	3.93	3.86	7.01
3F1 (OPR)	3.91	2.47	2.68	2.67	2.58	2.54	4.59
4F1 (OPR)	3.34	2.13	2.29	2.29	2.21	2.19	3.93
5C1 (OPR)	2.51	1.76	1.74	1.76	1.69	1.79	3.01
SU4 (OPR)	3.35	2.13	2.29	2.30	2.21	2.19	3.94
SU5 (OPR)	2.94	1.87	2.01	2.02	1.94	1.93	3.46
SU6 (OPR)	2.63	1.68	1.80	1.80	1.73	1.73	3.09
SU7 (OPR)	2.36	1.52	1.62	1.62	1.56	1.57	2.78
EV2 (OPR)	3.52	2.25	2.42	2.42	2.33	2.32	4.14
EV3 (OPR)	2.11	1.35	1.45	1.45	1.39	1.39	2.49

Figure D: Pier Cap Ratings

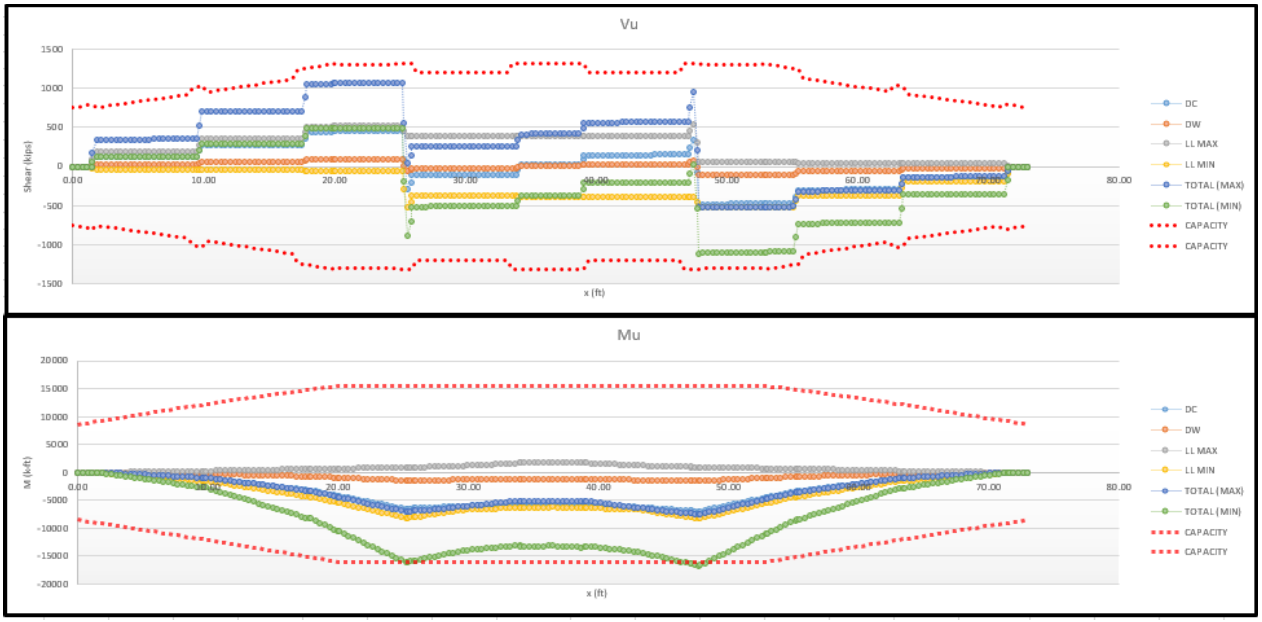


Figure E: Sample P2 Load/Capacity Summary for HL-93 TRK (INV)

Based on the table above, all pier types achieve an acceptable rating for the legal loads applied concurrently with the 3" proposed asphalt wearing surface. Further a majority of the pier types meet the preferred 150% rating factor specified by the BDM. The current HL-93 design vehicle fails the operating/inventory rating for select pier types; however, as the original bridge was not designed for the heavier HL-93 loads it was not anticipated that it would meet the live load demands of the more current inventory/operating loads.

All fracture critical members were analyzed for fatigue along the top and bottom welded plate connections and at member stiffener locations / areas of stress concentration. Components comprising the steel pier caps were assigned to various detail categories per AASHTO LRFD Table 6.6.1.2.3-1. The stress range was calculated per the design loading and compared against the infinite life stress threshold to assign a rating factor. A Fatigue load factor of 0.75 has been utilized per BDM guidelines. Since the fatigue rating factor exceeds 1.0 additional pier cap retrofits are not required and is further supported by BDM 404.1.4.

It should be noted that existing rocker and fixed bearings have not been analyzed for load carrying capacities as they were not included within the project scope.

SUPERSTRUCTURE SUMMARY

Standard rating procedures were performed to determine live load rating factors for the existing superstructure with the addition of the proposed 3" asphalt overlay. AASHTOWare BrR was used for analysis and results, as well as for comparison against the Midas model for confirmation and load verification. The results in **Figure F** depict the current rating factors of the main longitudinal beams.

STRUCTURE RATING SUMMARY

OHIO LEGAL VEHICLES			
Loading Type	GVW (Tons)	Operating Rating RF	Legal Weight (Tons)
2F1	15	2.324	15.00
3F1	23	1.576	23.00
4F1	27	1.391	27.00
5C1	40	1.529	40.00
SPECIALIZED HAULING VEHICLES (SHV)			
SU4	27	1.382	27.00
SU5	31	1.251	31.00
SU6	34.75	1.131	34.75
SU7	38.75	1.048	38.75
EMERGENCY VEHICLES (EV) *Check box if this is an NBI bridge* ☑			
EV2	28.75	1.341	28.75
EV3	43	0.889	38.23

DESIGN VEHICLE		
Loading Type	Rating by RF	
	Operating	Inventory
HL93 Loading	0.976	0.756

Overall Legal Posting Rating	90%
Posting Recommendation	EV Posting Recommended
Sign Posting Recommendation:	EMERGENCY VEHICLE WEIGHT LIMIT 2 AXLE 29 T 3 AXLE 38 T

Figure F: Superstructure Load Rating BR100 Form

All legal loads and special hauling vehicles achieve ratings above 1.0. However, the emergency vehicle (EV3) rates below 1.0 and would require structure posting. The original structure was not designed for HL-93 loading and thus similar to the pier cap results, the current inventory and operating factors are below 1.0.

It should be noted that the EV3 rating is controlled by the service limit state and includes a SER II load factor of 1.30. Although not specifically stated within the footnotes of BDM table 925-2, since the STR I load factor of 1.10 is utilized for non-interstate bridges, it is conservative to apply a larger load factor for the service limit state. If a reduced load factor was also considered for the service limit state of the EV3 load combination, a rating factor above 1.0 would be achieved and load posting would not be necessary.

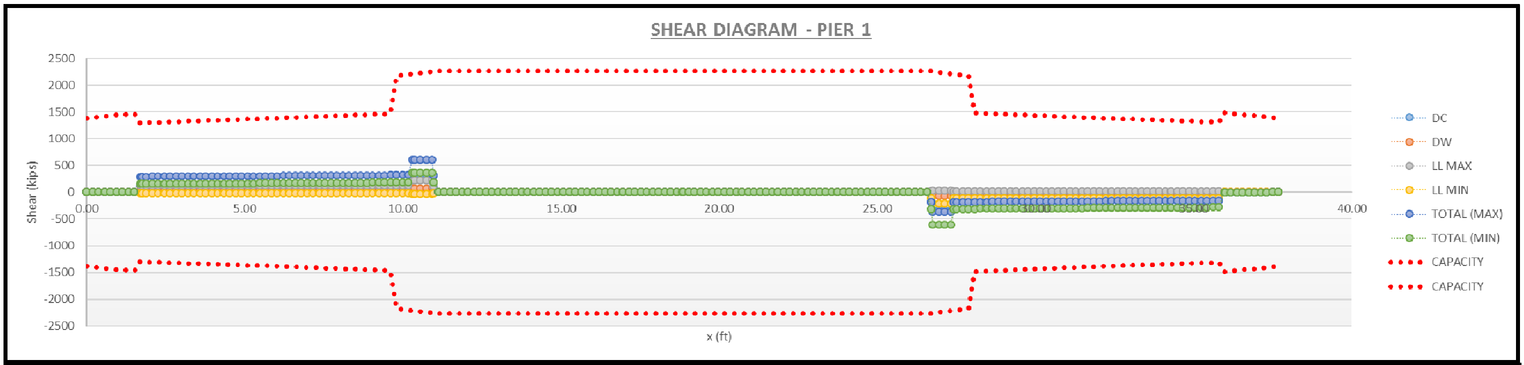
Within the superstructure rating a fatigue analysis was performed at the existing cover plate locations. The current condition does not meet the fatigue requirements at the moment plate end weld terminations, but does achieve an acceptable rating along the length of the plate. As the structure is located on a functional classification 02, the fatigue retrofits described within BDM 404.1.2.4.a would be required. These efforts are not currently included in the BEL-7-1152 (PID 110324) project scope but should be considered in the future rehabilitations.

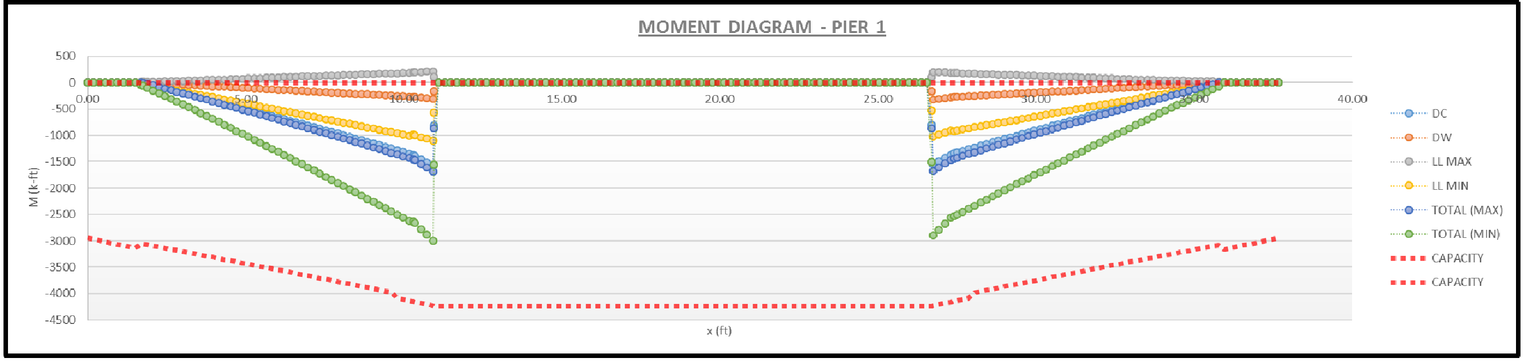
As all legal vehicles but the EV3 have a rating above 1.0 (a common rating result for bridges across the state), EMH&T recommends the installation of the proposed 3" asphalt wearing surface as scoped by the BEL-7-1152 (PID 110324) project.

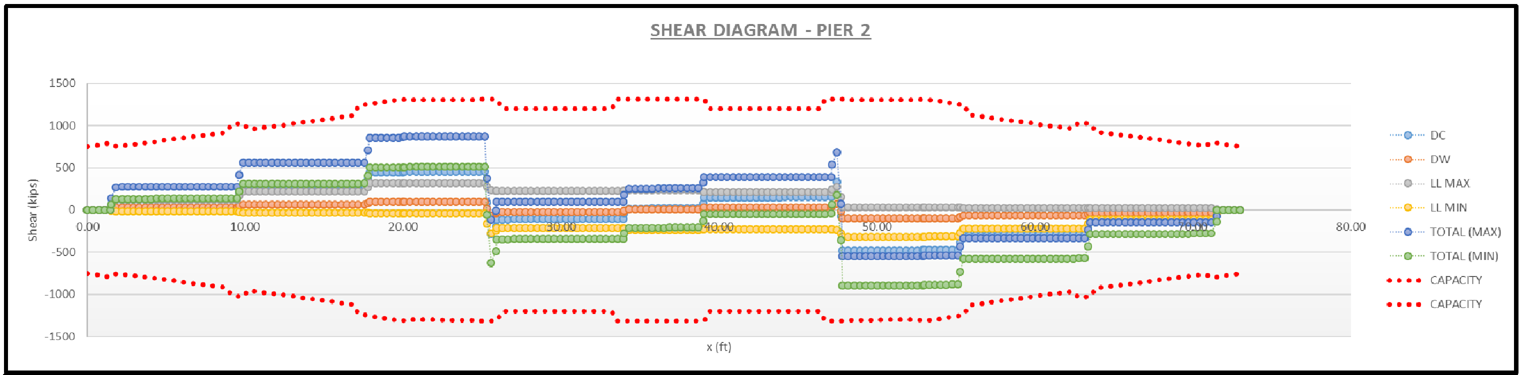
APPENDIX A – SHEAR AND MOMENT DIAGRAMS

Subject:	APPENDIX A - SHEAR AND MOMENT DIAGRAMS		
SFN:	0700312		
Date:	6/15/2020	Job No. :	20170025
Computed:	RMW	Checked:	CAS

MOMENT AND SHEAR DIAGRAMS FOR CONTROLLING LEGAL VEHICLE (EV3)AT EACH PIER


SHEAR DIAGRAM - PIER 1


MOMENT DIAGRAM - PIER 1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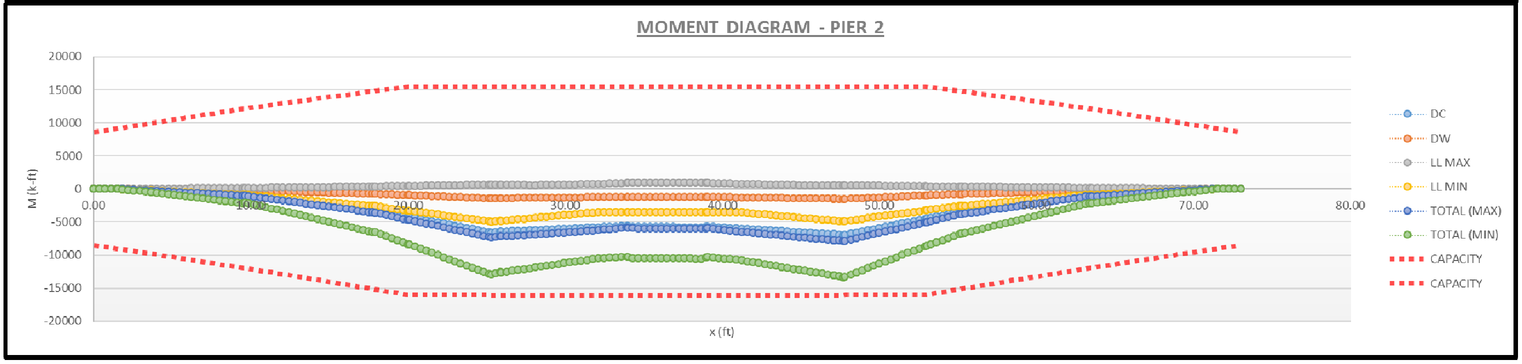
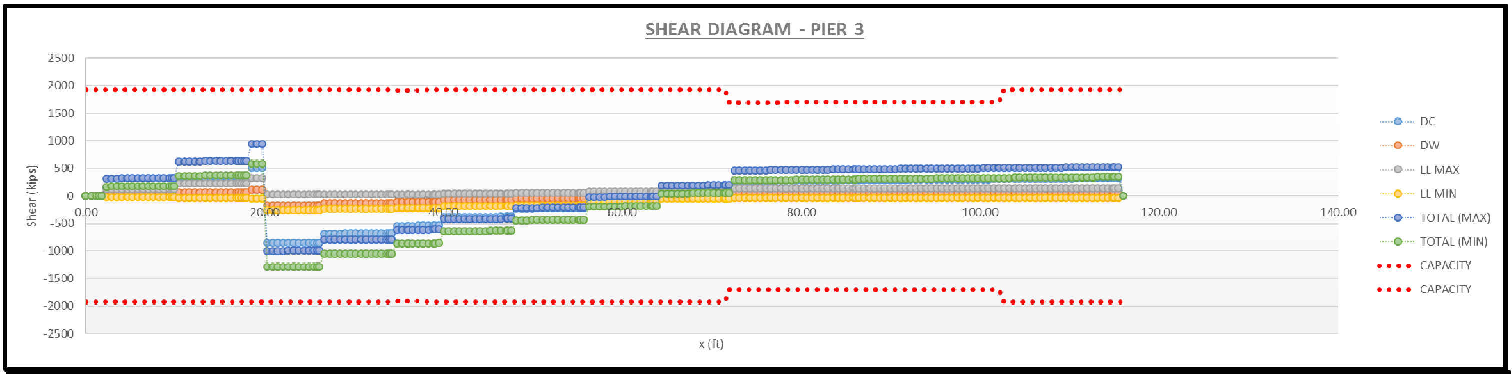

MOMENT DIAGRAM - PIER 2

EMH&T

Subject:	APPENDIX A - SHEAR AND MOMENT DIAGRAMS		
SFN:	0700312		
Date:	6/15/2020	Job No. :	20170025
Computed:	RMW	Checked:	CAS

MOMENT AND SHEAR DIAGRAMS FOR CONTROLLING LEGAL VEHICLE (EV3)AT EACH PIER CONTD.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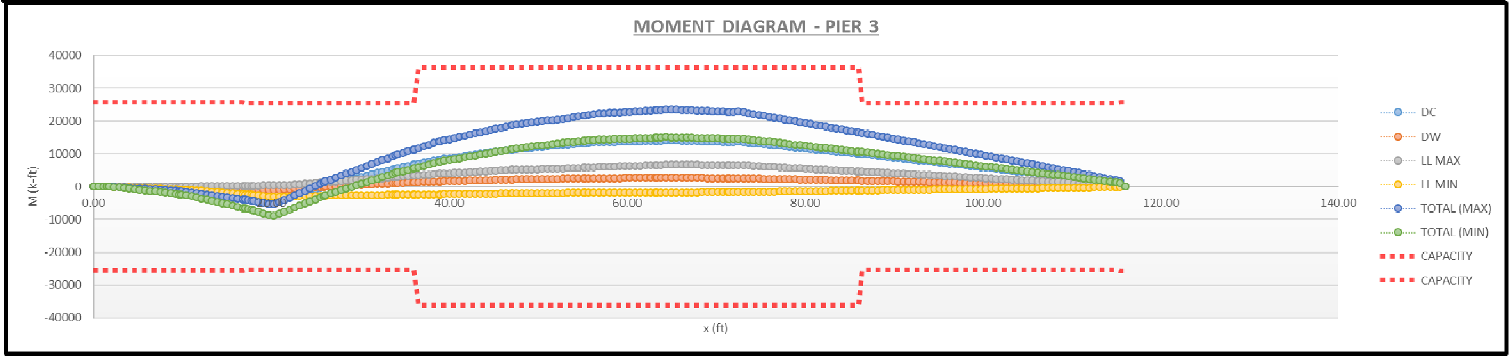
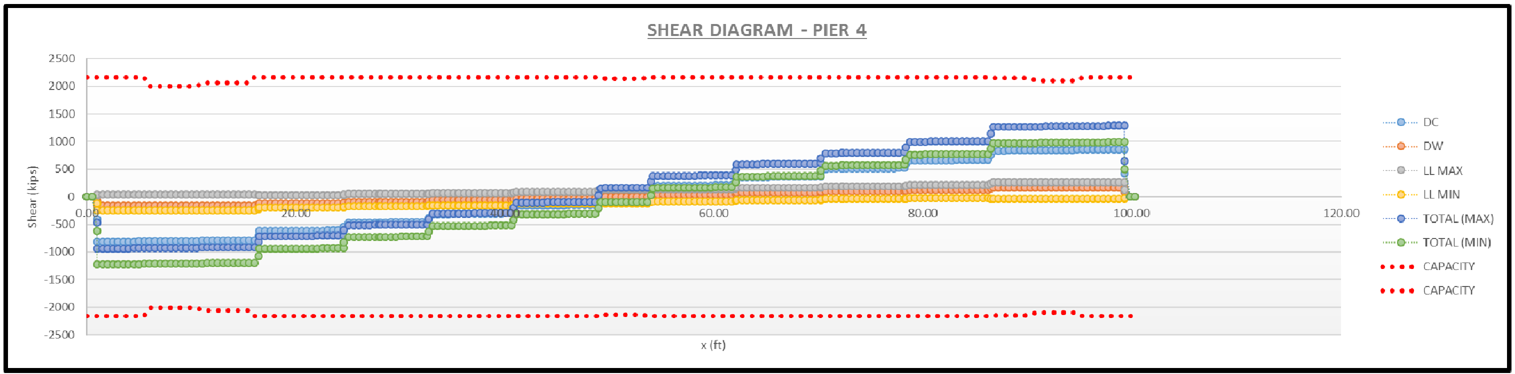

MOMENT DIAGRAM - PIER 3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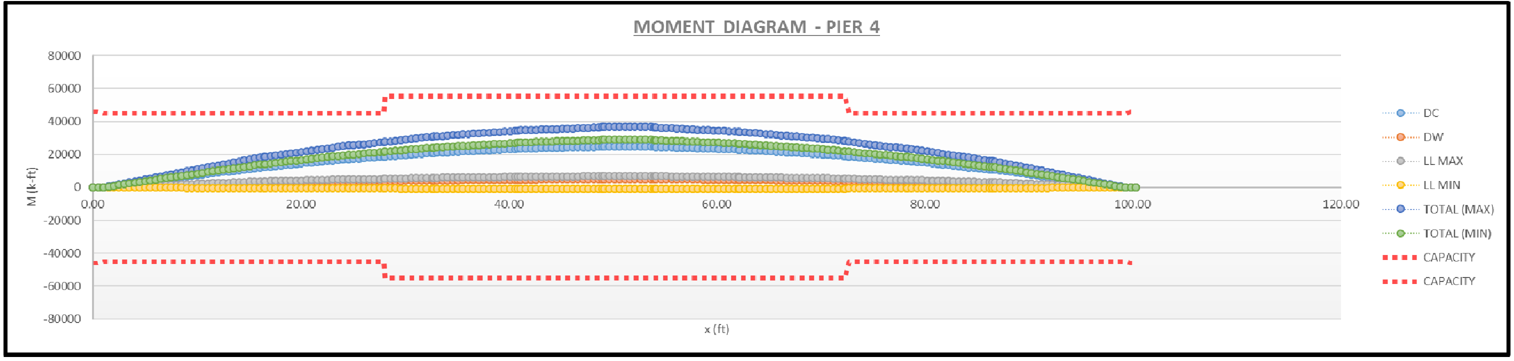
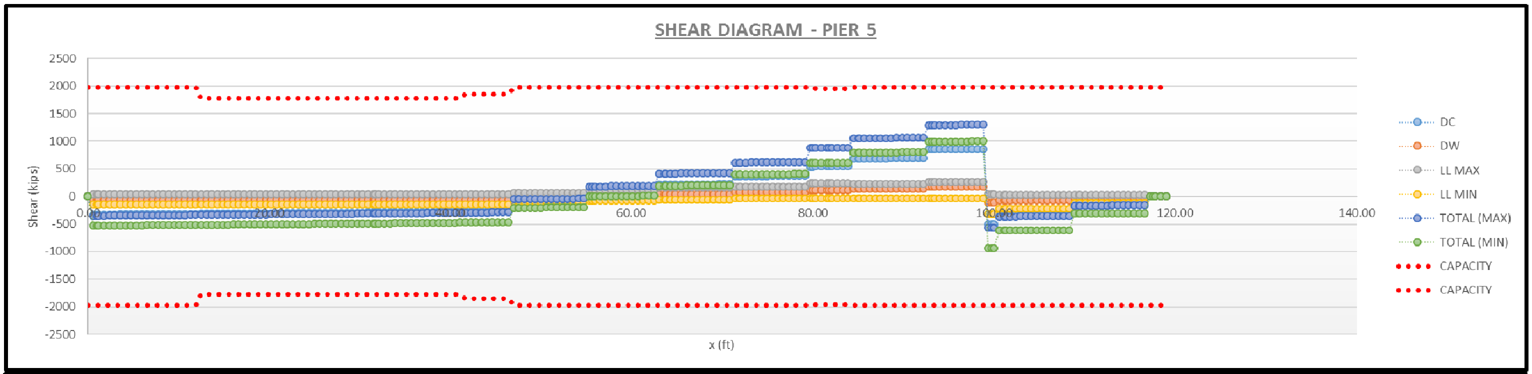
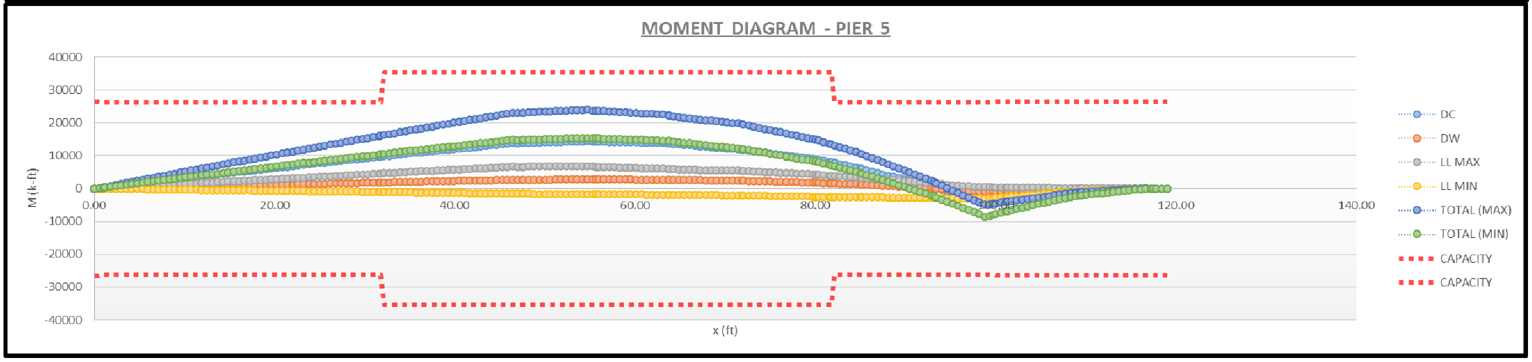
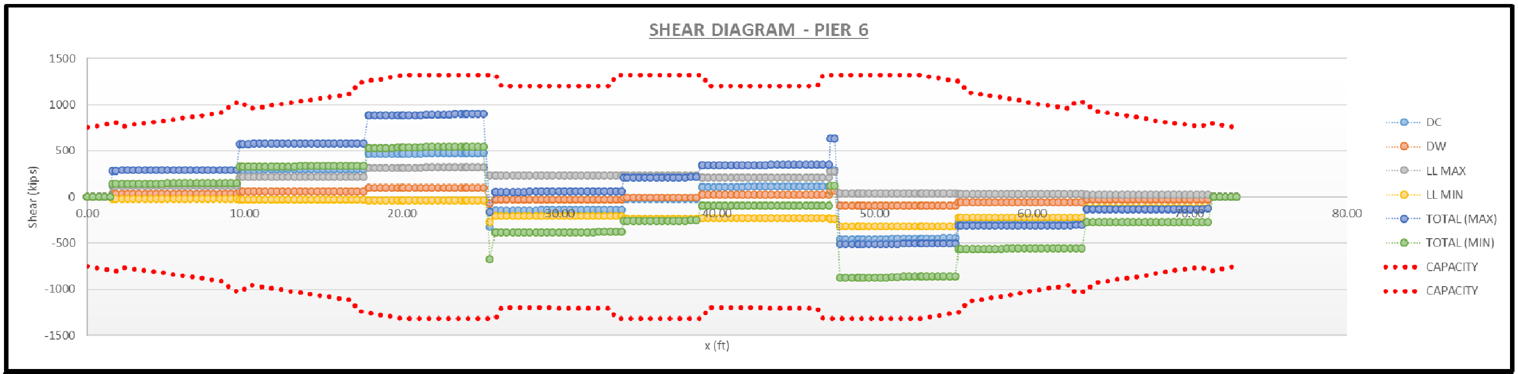
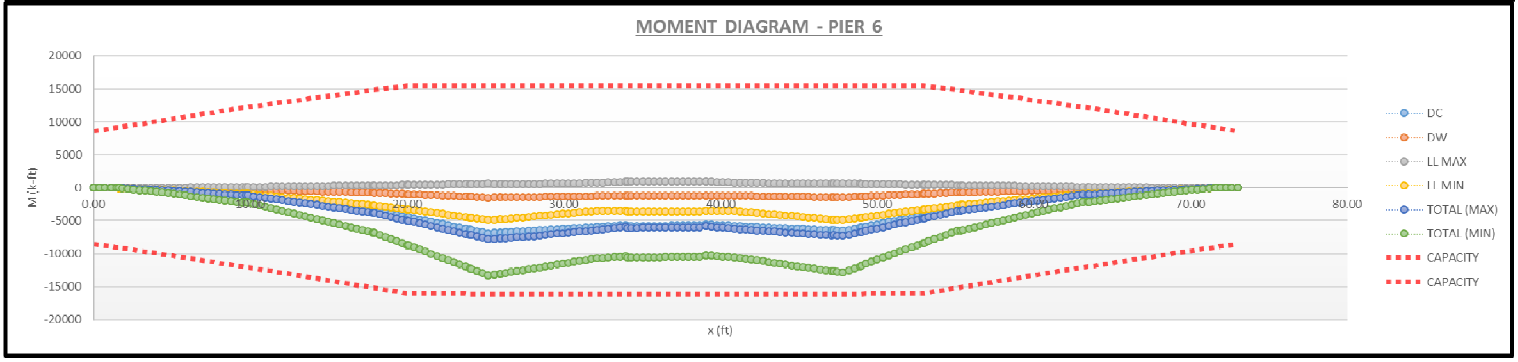

MOMENT DIAGRAM - PIER 4

MOMENT AND SHEAR DIAGRAMS FOR CONTROLLING LEGAL VEHICLE (EV3)AT EACH PIER CONTD.

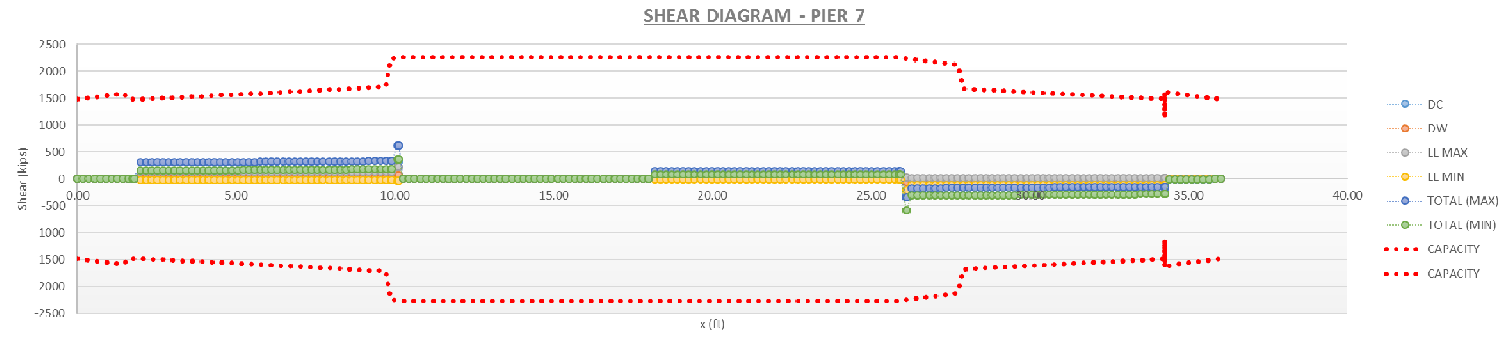


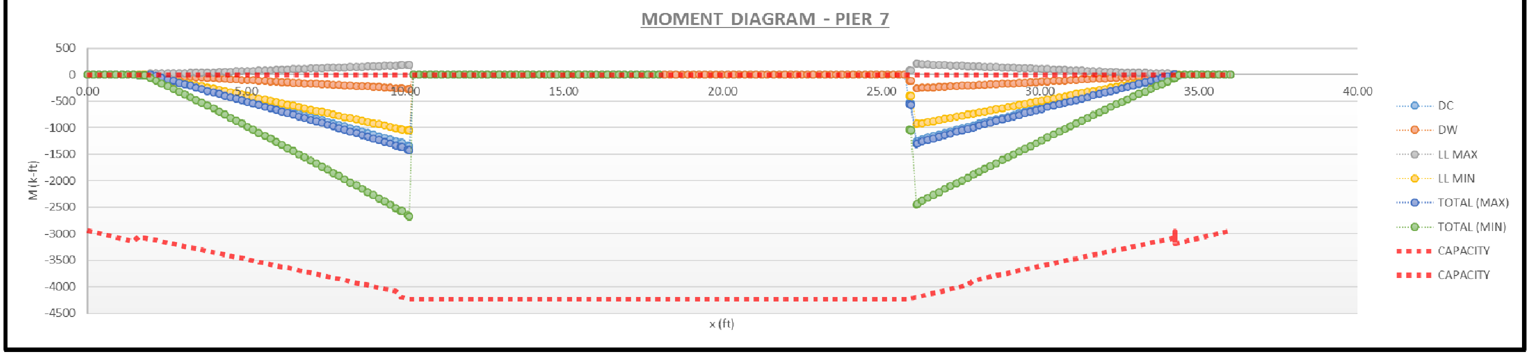




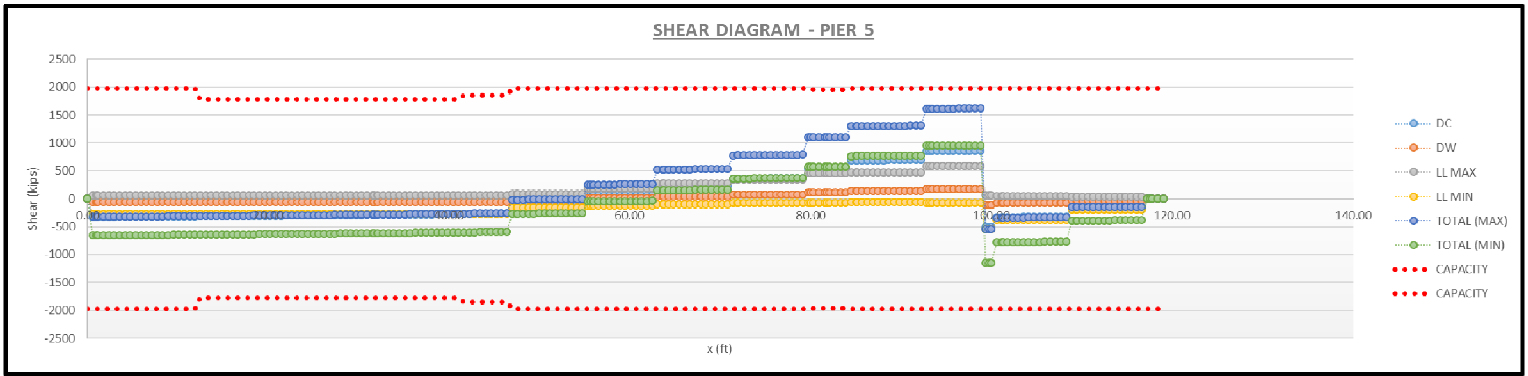


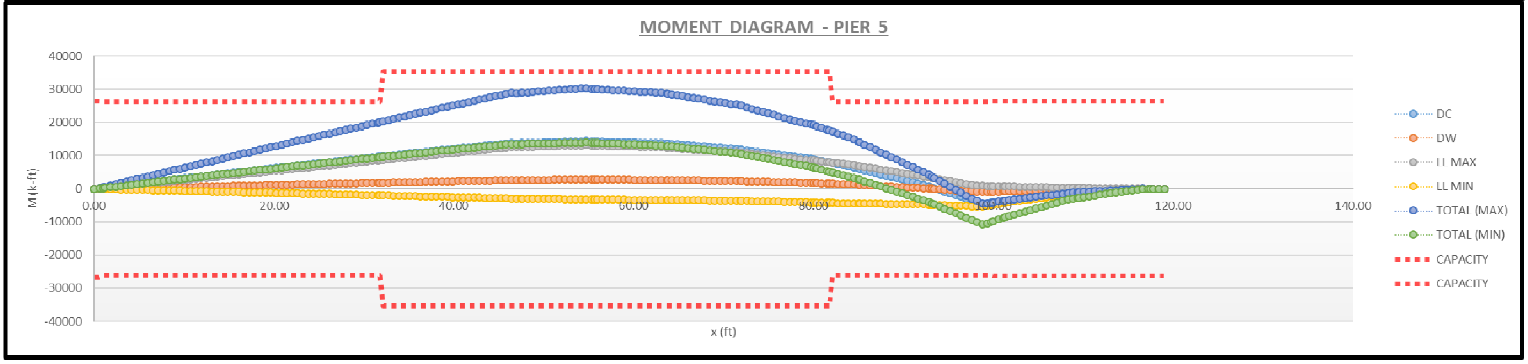
MOMENT AND SHEAR DIAGRAMS FOR CONTROLLING LEGAL VEHICLE (EV3)AT EACH PIER CONTD.


SHEAR DIAGRAM - PIER 7

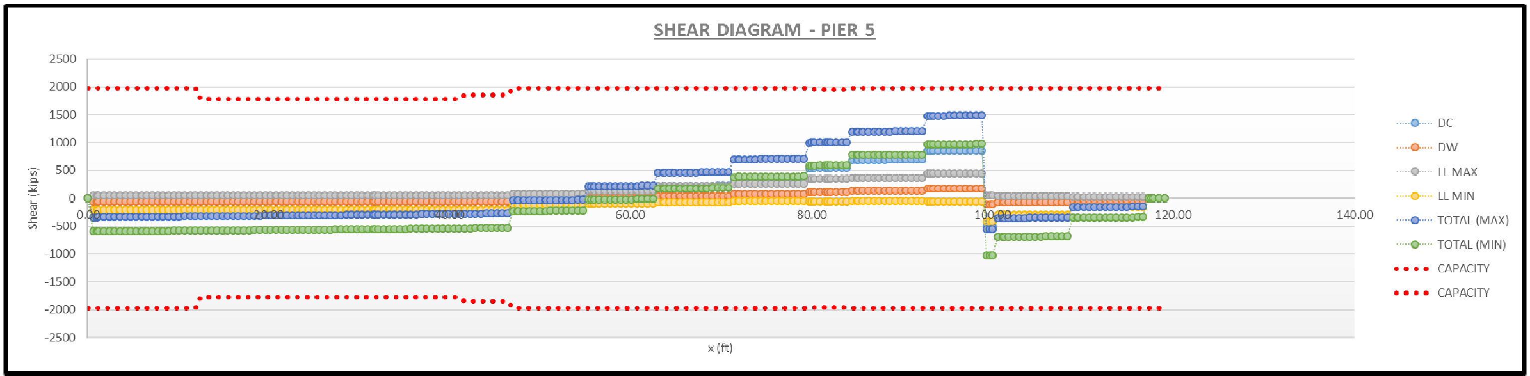

MOMENT DIAGRAM - PIER 7

MOMENT AND SHEAR DIAGRAM FOR CONROLLING HL-93 (INV)


SHEAR DIAGRAM - PIER 5


MOMENT DIAGRAM - PIER 5

MOMENT AND SHEAR DIAGRAM FOR CONROLLING HL-93 (OPR)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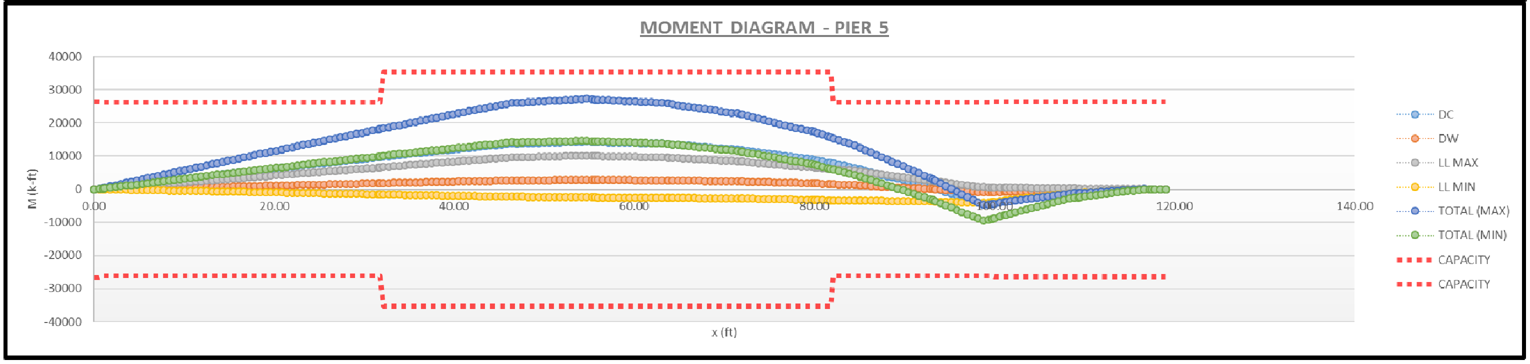
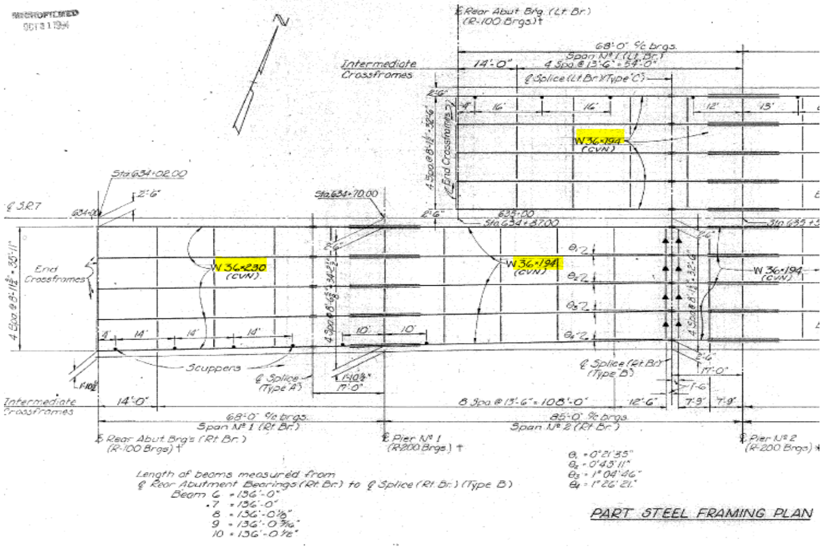
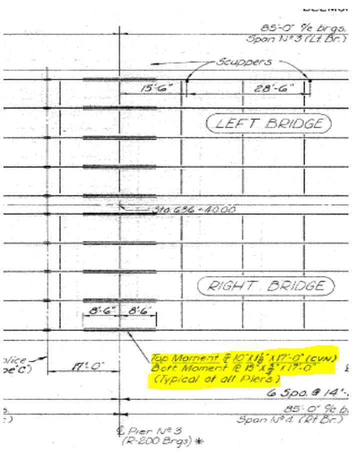
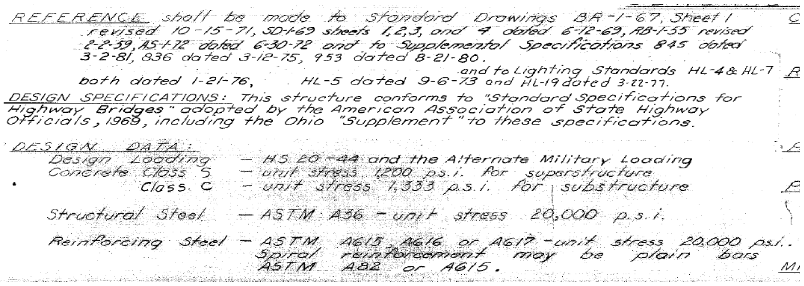

MOMENT DIAGRAM - PIER 5

APPENDIX B – MIDAS UNIT LOADS

EXISTING FRAMING PLAN AND MATERIAL SPECIFICATIONS (FROM RECORD PLANS)



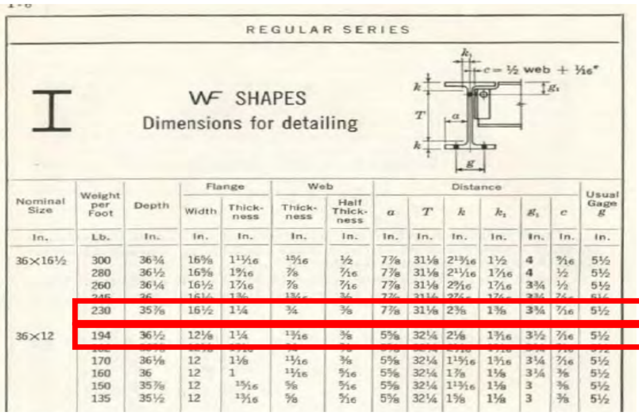



REFERENCE shall be made to Standard Drawings BR-1-67, Sheet 1 revised 10-15-71, SD-1-69 sheets 1,2,3, and 4 dated 6-12-69, RB-1-55 revised 2-2-59, AS-1-72 dated 6-30-72 and to Supplemental Specifications 845 dated 3-2-81, 836 dated 3-12-75, 953 dated 8-21-80. and to Lighting Standards HL-4 & HL-7 both dated 1-21-76, HL-5 dated 9-6-73 and HL-19 dated 3-22-77.

DESIGN SPECIFICATIONS: This structure conforms to "Standard Specifications for Highway Bridges" adopted by the American Association of State Highway Officials, 1969, including the Ohio "Supplement" to these specifications.

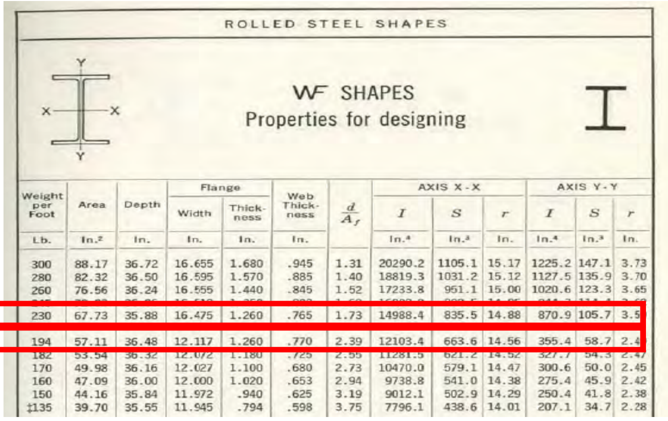
DESIGN DATA:
Design Loading — HS 20-44 and the Alternate Military Loading
Concrete Class S — unit stress 1,200 p.s.i. for superstructure
Class C — unit stress 1,333 p.s.i. for substructure
Structural Steel — ASTM A36 - unit stress 20,000 p.s.i.
Reinforcing Steel — ASTM A615, A616 or A617 - unit stress 20,000 p.s.i. Spiral reinforcement may be plain bars ASTM A82 or A615.

ARCHIVED AISC 1966 BEAM SHAPES


REGULAR SERIES

WF SHAPES
Dimensions for detailing

Nominal Size	Weight per Foot	Depth	Flange Width	Flange Thickness	Web Thickness	Web Half Thickness	Distance a	Distance T	Distance k	Distance k1	Distance g1	Distance c	Usual Gage g
In.	Lb.	In.	In.	In.	In.	In.	In.	In.	In.	In.	In.	In.	In.
36×16½	300	36¾	16⅝	1 11/16	15/16	½	7⅞	31⅛	2 13/16	1½	4	9/16	5½
	280	36½	16⅝	1 9/16	⅞	7/16	7⅞	31⅛	2 11/16	1 7/16	4	½	5½
	260	36¼	16½	1 7/16	⅞	7/16	7⅞	31⅛	2 9/16	1 7/16	3¾	½	5½
	245	36	16½	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	230	35⅞	16½	1¼	¾	⅜	7⅞	31⅛	2⅜	1⅜	3¾	7/16	5½
36×12	194	36½	12⅛	1¼	13/16	⅜	5⅝	32¼	2⅛	1 3/16	3½	7/16	5½
	182	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	170	36⅛	12	1⅛	11/16	⅜	5⅝	32¼	1 15/16	1 3/16	3¼	7/16	5½
	160	36	12	1	11/16	5/16	5⅝	32¼	1⅞	1⅛	3¼	⅜	5½
	150	35⅞	12	15/16	⅝	5/16	5⅝	32¼	1 13/16	1⅛	3	⅜	5½
	135	35½	12	13/16	⅝	5/16	5⅝	32¼	1⅝	1⅛	3	⅜	5½


ROLLED STEEL SHAPES

WF SHAPES
Properties for designing

Weight per Foot	Area	Depth	Flange Width	Flange Thickness	Web Thickness	$\frac{d}{A_f}$	Axis X-X I	Axis X-X S	Axis X-X r	Axis Y-Y I	Axis Y-Y S	Axis Y-Y r
Lb.	In.^{2}	In.	In.	In.	In.		In.^{4}	In.^{3}	In.	In.^{4}	In.^{3}	In.
300	88.17	36.72	16.655	1.680	.945	1.31	20290.2	1105.1	15.17	1225.2	147.1	3.73
280	82.32	36.50	16.595	1.570	.885	1.40	18819.3	1031.2	15.12	1127.5	135.9	3.70
260	76.56	36.24	16.555	1.440	.845	1.52	17233.8	951.1	15.00	1020.6	123.3	3.65
245	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
230	67.73	35.88	16.475	1.260	.765	1.73	14988.4	835.5	14.88	870.9	105.7	3.5
194	57.11	36.48	12.117	1.260	.770	2.39	12103.4	663.6	14.56	355.4	58.7	2.4
182	53.54	36.32	12.072	1.180	.725	2.55	11281.5	621.2	14.52	327.7	54.3	2.47
170	49.98	36.16	12.027	1.100	.680	2.73	10470.0	579.1	14.47	300.6	50.0	2.45
160	47.09	36.00	12.000	1.020	.653	2.94	9738.8	541.0	14.38	275.4	45.9	2.42
150	44.16	35.84	11.972	.940	.625	3.19	9012.1	502.9	14.29	250.4	41.8	2.38
‡135	39.70	35.55	11.945	.794	.598	3.75	7796.1	438.6	14.01	207.1	34.7	2.28

MIDAS AND BRR INPUTS - PRIMARY MEMBER GEOMETRY AND "EFFECTIVE SECTION" OVER PIERS

MIDAS	SECTION	bf	tf	dw	tw	d
3	W36x230	16.47	1.26	33.38	0.76	35.9
4	W36x194	12.115	1.26	33.97	0.765	36.49

MOMENT PLATES OVER PIERS

Record P	bf (in)	tf (in)	L (ft)
TOP	10	1.125	17
BOTTOM	15	0.75	17

Midas	SECTION	bf	tf (top)	tf (bot)	dw	tw	d
5	W36X230M	16.47	1.94	1.94	33.38	0.76	37.27
6	W36X194M	12.12	2.19	2.19	33.97	0.77	38.35

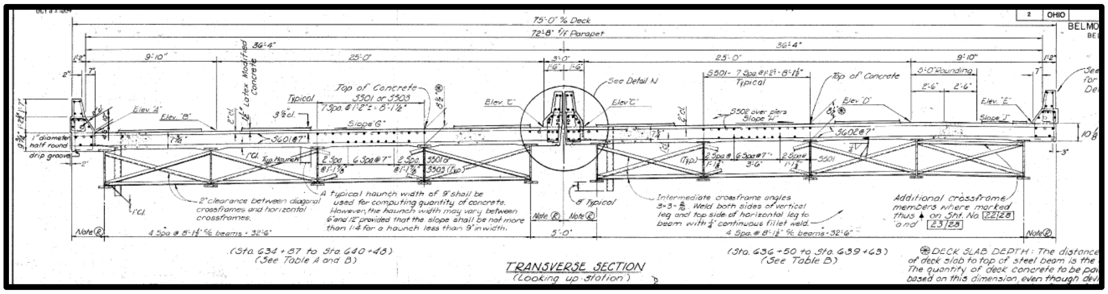
** - TOTAL FLANGE THICKNESS INCLUDING COVER PLATE AREA DISTRIBUTED OVER EXISTING FLANGE WIDTH

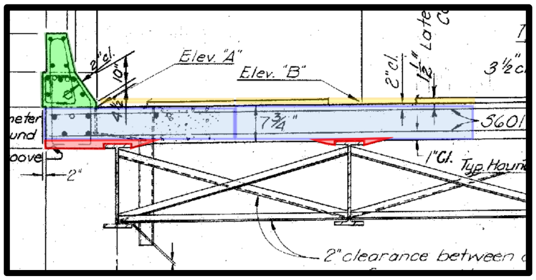
MEMBER UNIT WEIGHTS

MEMBER	AREA (sqin)	WT (#/FT)
3	66.87	227.55
4	56.52	192.31
5	89.37	304.12
6	79.02	268.88

** AN ADDITIONAL 10% HAS BEEN INCLUDED FOR DL ALLOWANCE ACCOUNTING FOR SPLICES, AND MISC STEEL

EXISTING DECK SECTION AND CALCULATED UNIT WEIGHTS





🟦 - SLAB WEIGHT (7.75" MIN +EX 1.5" LMC OVERLAY)
🟥 - HAUNCH WEIGHT
🟩 - BARRIER WEIGHT
🟨 - PROP 3" ASPHALT OVERLAY

* REFERENCE FRAMING PLAN FOR BEAM SPACINGS
* REFERENCE TYPICAL SECTION FOR DECK THICKNESS, VARIES FOR RIGHT BRIDGE WITHIN SPANS 1 AND 2

	TRIBUTARY DECK WIDTH (FT): DC-DECK									
	Beam	RA	RA - P1	P2	P3	P4	P5	P6	P7 - FA	FA
LEFT BRIDGE	1		6.56	6.56	6.56	6.56	6.56	6.79	6.57	6.59
	2		8.13	8.13	8.13	8.13	8.13	8.13	8.13	8.13
	3		8.13	8.13	8.13	8.13	8.13	8.13	8.13	8.13
	4		8.13	8.13	8.13	8.13	8.13	8.13	8.13	8.13
	5		6.48	6.48	6.48	6.48	6.48	6.48	6.48	6.48
RIGHT BRIDGE	6	6.91	6.69	6.48	6.48	6.48	6.48	6.48	6.48	
	7	8.98	8.55	8.13	8.13	8.13	8.13	8.13	8.13	
	8	8.98	8.55	8.13	8.13	8.13	8.13	8.13	8.13	
	9	8.98	8.55	8.13	8.13	8.13	8.13	8.13	8.13	
	10	6.36	6.15	6.56	6.56	6.56	6.56	6.48	6.57	

CALCULATED UNIT WEIGHT AT SUPPORTS

	TRIBUTARY DECK WEIGHT (#/FT): DC-DECK									
	Beam	RA	RA - P1	P2	P3	P4	P5	P6	P7 - FA	FA
LEFT BRIDGE	1		759	759	759	759	759	785	760	762
	2		939	939	939	939	939	939	939	939
	3		939	939	939	939	939	939	939	939
	4		939	939	939	939	939	939	939	939
	5		749	749	749	749	749	749	749	749
RIGHT BRIDGE	6	842	795	749	749	749	749	749	749	
	7	1094	1016	939	939	939	939	939	939	
	8	1094	1016	939	939	939	939	939	939	
	9	1094	1016	939	939	939	939	939	939	
	10	776	730	759	759	759	759	749	760	

APPLIED UNIT LOADS (AVERAGE UNIT WEIGHT BETWEEN SPANS)

	TRIBUTARY DECK WEIGHT (#/FT): DC-DECK								
	Beam	SPAN1	SPAN 2	SPAN 3	SPAN 4	SPAN 5	SPAN 6	SPAN 7	SPAN 8
LEFT BRIDGE	1		759	759	759	759	772	773	761
	2		939	939	939	939	939	939	939
	3		939	939	939	939	939	939	939
	4		939	939	939	939	939	939	939
	5		749	749	749	749	749	749	749
RIGHT BRIDGE	6	818	772	749	749	749	749	749	
	7	1055	978	939	939	939	939	939	
	8	1055	978	939	939	939	939	939	
	9	1055	978	939	939	939	939	939	
	10	753	745	759	759	759	754	755	

	HAUNCH AREA (SQ IN): DC-HAUNCH									
	Beam	RA	RA - P1	P2	P3	P4	P5	P6	P7 - FA	FA
LEFT BRIDGE	1		65.96	65.96	65.96	65.96	65.96	71.46	66.21	66.71
	2		27.18	27.18	27.18	27.18	27.18	27.18	27.18	27.18
	3		27.18	27.18	27.18	27.18	27.18	27.18	27.18	27.18
	4		27.18	27.18	27.18	27.18	27.18	27.18	27.18	27.18
	5		64.25	64.25	64.25	64.25	64.25	64.25	64.25	64.25
RIGHT BRIDGE	6	63.13	63.13	64.25	64.25	64.25	64.25	64.25	64.25	
	7	30.44	30.44	27.18	27.18	27.18	27.18	27.18	27.18	
	8	30.44	30.44	27.18	27.18	27.18	27.18	27.18	27.18	
	9	30.44	30.44	27.18	27.18	27.18	27.18	27.18	27.18	
	10	50.07	50.07	66.26	66.26	66.26	66.26	64.24	66.51	

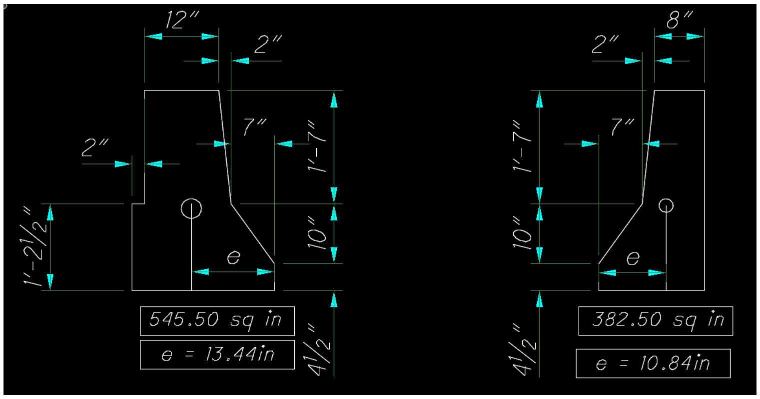
CALCULATED UNIT WEIGHT AT SUPPORTS

	HAUNCH UNIT WEIGHT (#/FT): DC-HAUNCH									
	Beam	RA	RA - P1	P2	P3	P4	P5	P6	P7 - FA	FA
LEFT BRIDGE	1		69	69	69	69	69	74	69	69
	2		28	28	28	28	28	28	28	28
	3		28	28	28	28	28	28	28	28
	4		28	28	28	28	28	28	28	28
	5		67	67	67	67	67	67	67	67
RIGHT BRIDGE	6	66	66	67	67	67	67	67	67	
	7	32	32	28	28	28	28	28	28	
	8	32	32	28	28	28	28	28	28	
	9	32	32	28	28	28	28	28	28	
	10	52	52	69	69	69	69	67	69	

APPLIED UNIT LOADS (AVERAGE UNIT WEIGHT BETWEEN SPANS)

	HAUNCH AREA (#/FT): DC-HAUNCH								
	Beam	SPAN1	SPAN 2	SPAN 3	SPAN 4	SPAN 5	SPAN 6	SPAN 7	SPAN 8
LEFT BRIDGE	1		69	69	69	69	72	72	69
	2		28	28	28	28	28	28	28
	3		28	28	28	28	28	28	28
	4		28	28	28	28	28	28	28
	5		67	67	67	67	67	67	67
RIGHT BRIDGE	6	66	66	67	67	67	67	67	
	7	32	30	28	28	28	28	28	
	8	32	30	28	28	28	28	28	
	9	32	30	28	28	28	28	28	
	10	52	61	69	69	69	68	68	

BARRIER AND PARAPET AREA (SQ IN): DC-BARR



BARRIER AND PARAPET UNIT WEIGHT (#/FT): DC-BARR

BARRIER 1 UNIT WEIGHT =	545.5 SQ IN,	568.23 #/FT
BARRIER 2 UNIT WEIGHT =	382.5 SQ IN,	398.44 #/FT

TOTAL BARRIER WT	966.67 #/FT	=	193 #/FT	EACH BEAM
# BEAMS	5 BEAMS			

TRAVEL WAY WIDTH (FT): DC-WS

	RA	RA - P1	P2	P3	P4	P5	P6	P7 - FA	FA
LEFT	0	34.25	34.25	34.25	34.25	34.25	34.25	34.25	34.25
RIGHT	36.91667	35.20833	34.25	34.25	34.25	34.25	34.25	34.25	

3" FWS UNIT LOAD PER BEAM (#/FT): DC-WS

	RA	RA - P1	P2	P3	P4	P5	P6	P7 - FA	FA
LEFT	0	248	248	248	248	248	248	248	248
RIGHT	268	255	248	248	248	248	248	248	0

3" FWS AVERAGE UNIT LOAD BETWEEN SUPPORTS (#/FT): DC-WS

	SPAN1	SPAN 2	SPAN 3	SPAN 4	SPAN 5	SPAN 6	SPAN 7	SPAN 8
LEFT	124	248	248	248	248	248	248	248
RIGHT	261	252	248	248	248	248	248	124

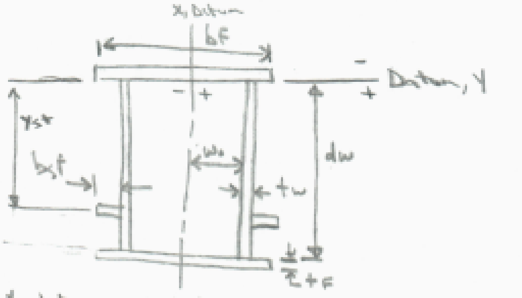
APPENDIX C – SAMPLE HAND CALCULATIONS

CAPACITY CALCULATIONS BELOW ARE PROVIDED FOR REFERENCE ONLY. ACTUAL FORMULAS AND VALUES ARE CALCULATED WITHIN A SEPERATE RATING SPREADSHEET AT EACH POINT OF INTEREST (POI) AND COMPARED AGAINST THE CONTROLLING MIDAS GENERATED LOADS TO ASSIGN A RATING FACTOR FOR EACH VEHICLE

PIER 2 AT LEFT COLUMN SUPPORT - STEEL PIER CAP SAMPLE CAPACITY HAND CALCULATIONS

Capacity Calculations – Steel Pier Cap

Shape Properties @ Support location (varies along length)



$b_f = 48"$
$t_f = 1.125"$
$d_w = 84"$
$t_w = .375"$
$W_o = 17.625"$
$b_{st} = 4.5"$
$t_{st} = .3125"$
$Y_{st} = 67.22"$

* datum selected as web depth varies along length

Area

$A_{tf} = 48" \times 1.125" = 54\ in^2$

$A_w = 84" \times .375" \times 2\ webs = 63\ in^2$

$A_{bf} = 48" \times 1.125" = 54\ in^2$

$A_{st} = 4.5" \times .3125" \times 2\ stiff = 2.81\ in^2$

Area Total $= 54\ in^2 + 63\ in^2 + 54\ in^2 + 2.81\ in^2 = 173.81\ in^2$

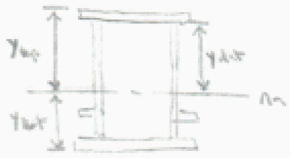
Centroid from Datum

$$y_{dat} = [54\ in^2 \times (-1.125"/2) + 63\ in^2 \times (84"/2) + 2.81\ in^2\,(67.22") + 54\ in^2\,(84" + 1.125"/2)] / 173.81\ in^2$$

$$= 42.41\ in$$

$$y_{bot} = \underbrace{(2 \times 1.125 + 84")}_{total\ depth} - (\underbrace{42.41"}_{y\ dat} + \underbrace{1.125"}_{tf}) = 42.72"$$

$$Y_{top} = \underbrace{42.41}_{ydat} + \underbrace{1.125"}_{tf} = 43.53"$$



* Ytop/Ybot taken at extreme top for comparison in flexural calcs, potential to take @ center of flange for additional capacity

Moment of Inertia Calculations

$I_{xx,\ tf/bf} = \frac{1}{12} \times 48 \times 1.125^3 = 5.70\ in^4$

$I_{xx,\ w} = \frac{1}{12} \times .375 \times 84^3 \times 2\ webs = 37044\ in^4$

$I_{xx,\ st} = \frac{1}{12} \times 4.5" \times .3125^3 = .011\ in^4$ (single stiffener)

$$I_{,\ x\text{-}x} = \Sigma I_{xx} + Ay^2$$

$$= 5.70\ in^4 + 54\ in^2 (42.41" + 1.125/2)^2 + 37044\ in^4 + 63\ in^2 (42.41 - 84"/2)^2 \ldots$$

$$+ 5.70\ in^4 + 84\ in^2\,(84" + 1.125/2 - 42.41")^2 + 2 \times .011 + 2.81 \times (67.22 - 42.41)^2$$

$$= 234463\ in^4$$

Moment of Inertia calcs, contd.

$I_{y\text{-}y,\ tf/bf} = \frac{1}{12} \times 48^3 \times 1.125 = 10368\ in^4$

$I_{y\text{-}y,\ w} = \frac{1}{12} \times .375^3 \times 84 = .369\ in^4$ (single web)

$I_{y\text{-}y,\ st} = \frac{1}{12} \times 4.5^3 \times .3125 = 2.37\ in^4$ (single stiff)

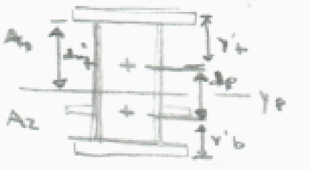
$I_{y\text{-}y} = 10368\ in^4 \times 2 + .369\ in^4 \times 2 + 63\ in^2 (17.625'' + .375/2)^2 + 2.37 \times 2 + 2.81 \left(17.625'' + .375'' + \frac{4.5}{2}\right)^2$

$= 41883\ in^3$

Section Modulus calcs

$S_{top} = \frac{I_{xx}}{y_{top}} = \frac{234463\ in^4}{43.53''} = 5386\ in^3$

$S_{bot} = \frac{I_{xx}}{y_{bot}} = \frac{234463\ in^4}{42.72''} = 5488.37\ in^3$



y_p occurs @ $A_1 = A_2$

- plastic section modulus (Z)

$d_{wt} = \left(\underbrace{\frac{173.81}{2}}_{\frac{A_T}{2}} - \underbrace{48'' \times 1.125}_{A_{tf}}\right) \Big/ \underbrace{(.375'' \times 2)}_{t_w} = 43.87\ in$

$y'_t = \left[54'' \times 1.125/2 + .375'' \times 43.87 \times 2 \times \left(\frac{43.87''}{2} + 1.125''\right)\right] \Big/ \underbrace{\left(\frac{173.81\ in^2}{2}\right)}_{AT/2} = 9.08''$

$y'_b = [\,54'' \times 1.125/2 + .375 \times (84'' - 43.87) \times 2 \times ((84 - 43.87)/2 + 1.125) + 2.81 \times (\underbrace{86.25''}_{d_t} - \underbrace{67.22''}_{y_{st}} + \underbrace{1.125}_{t_f})$

$\cdots / (173.81/2)$

$= 8.27''$

$d_p = \underbrace{84'' + 2 \times 1.125''}_{d_t} - 9.08'' - 8.27'' = 68.9''$

$Z = \frac{A}{2} \times d_p = 173.81\ in^2/2 \times 68.9'' = 5988\ in^3$

(reference Excel file for further clarification)

Capacity Calculation per AASHTO LRFD, 8th Edition

* per C6.11.1, noncomposite box sections shall be analyzed per 6.12.2.2.2

Flexural Resistance, 6.12.1.2

$M_r = \phi_f M_n$ (6.12.1.2.1-1)

where $\phi_f = 1.0$ (6.5.4.2)

M_n = lesser of equations 6.12.2.2.2-1 through 6.12.2.2.2-9

$$M_{n_1} = F_y S \left[1 - \frac{.064 F_y S \ell}{AE} \left(\frac{\Sigma\left(\frac{b}{t}\right)}{I_y} \right)^{.5} \right] \quad (6.12.2.2.2\text{-}1)$$

* note: this equation is used to satisfy the requirements of LTB as specified in C6.12.2.2.2 for rectangular shapes
additionally for this pier cap, the top Flange will be checked for Flexural capacity as it will be in tension under all load conditions. Bottom flange checks similar but use separate section modulus

$$\frac{b_f}{t_f} = (48'' - \underbrace{2\times 6''}_{\text{flange overhang}}) / (1.125'') = 32$$

$$\frac{d_w}{t_w} = \frac{84''}{.375} = 224$$

$$\Sigma\left(\frac{b}{t}\right) = \overbrace{2\times 32 + 2\times 224}^{\text{2 webs, 2 flanges}} = 512$$

$A = (84'' + 2\times 1.125''/2) \times (35.25'' + .375''\times 2/2) = 3037 \text{ in}^2$ (area of rectangle between flange centerlines excluding overhang)

$\ell = 22.4'$ (interior span), $25.3' \times 2 = 50.6'$ (exterior span ×2 for cantilever)

$= 50.6' \times 12 = 607.2''$ (exterior span unbraced length use conservatively @ support)

$$M_{n_1} = (36\text{ ksi})(5386\text{ in}^3)\left[1 - \frac{.064(36)(5386)(607'')}{3037 \times 29000}\left(\frac{512}{41883}\right)^{.5}\right] = \underline{192062\text{ k-in}}$$

$$M_{n_2} = M_p = F_y Z = 36\text{ ksi} \times 5988\text{ in}^3 = \underline{215,568\text{ k-in}}$$

$$\lambda_f = \frac{b_{fc}}{t_{fc}} = \frac{2\times 17.625}{1.125} = 31.33 \quad (6.12.2.2.2\text{-}5)$$

$$\lambda_{pf} = 1.12\sqrt{\frac{E}{F_y}} = 1.12\sqrt{\frac{29000}{36}} = 31.79 \quad (6.12.2.2.2\text{-}5)$$

$$\lambda_{rf} = 1.40\sqrt{\frac{E}{F_y}} = 1.40\sqrt{\frac{29000}{36}} = 39.74 \quad (6.12.2.2.2\text{-}6)$$

$\lambda_f < \lambda_{pf}$ ∴ compact flange, eqns 6.12.2.2.2-3 to 6.12.2.2.2-8 don't apply at this section

$$\lambda_{pw} = 2.42\sqrt{\frac{E}{F_y}} = 2.42\sqrt{\frac{29000}{36}} = 68.69 \qquad (6.12.2.2.2\text{-}10)$$

$$\frac{D}{t_w} = 224 > \lambda_{pw} \quad \therefore \text{ web local buckling applies}$$

$$M_{ny} = M_p - (M_p - F_y S)\left(.305\frac{D}{t_w}\sqrt{\frac{F_y}{E}} - .738\right) \leq M_p \qquad (6.12.2.2.2\text{-}9)$$

$$= 215568 - (215568 - 36 \times 5386)\left(.305(224)\sqrt{\frac{36}{29000}} - .738\right) = \underline{179398 \text{ k-in}}$$

In order for web local buckling to occur per 6.12.2.2.2-9, web bend buckling must occur as described within 6.11.9 (6.10.4.2.2-4) for service checks of rectangular members.

$$f_c \leq F_{crw} \qquad (6.10.4.2.2\text{-}4)$$

$$F_{crw} = \frac{0.9Ek}{\left(\frac{D}{t_w}\right)^2} \qquad (6.10.1.9.1\text{-}1)$$

k = buckling coefficient with long. stiffeners (6.10.1.9.2)

$$d_s = 84'' - 67.22'' = 16.78'' \qquad \text{(distance from long stiff to comp flg)}$$

$$D_c = 84'' - \underbrace{42.41''}_{y_{bar}} = 41.59'' \qquad \text{(web depth in compression)}$$

$$\frac{d_s}{D_c} = \frac{16.78}{41.59} = 0.40 \quad \therefore \; 6.10.1.9.2\text{-}1 \text{ controls}$$

$$k = \frac{5.17}{(d_s/D)^2} \geq \frac{9}{(D_c/D)^2} \qquad \underbrace{\frac{5.17}{(16.78/84)^2}}_{129.55} \geq \underbrace{\frac{9}{(41.59/84)^2}}_{36.71}$$

$k = 129.55$ controls

$$F_{crw} = \frac{0.9(29000)(129.55)}{(224)^2} = 67.4 < f_c$$

therefore web bend buckling does not apply as section yields before web buckles

$\phi M_n = 1.0 \times \min\left(193896 \text{ k-in},\ 215{,}568 \text{ k-in},\ \cancel{179336} \text{ k-in}\right)$

$= 193896 \text{ k-in} / 12 = 16158 \text{ k-ft}$

$\boxed{\phi M_n = 16158 \text{ k-ft}}$ @ support

Shear Resistance 6.12.1.2.3

$V_r = \phi V_n$ (6.12.1.2.3a-1, 6.11.9, 6.10.9)

$\phi_v = 1.0$ (6.5.4.2)

determine web stiffener applicability per 6.10.9.1

since a longitudinal stiffener exists, transverse stiffener spacing cannot exceed 1.5 D for interior panels

stiffener spacing @ support per record plans $= \underset{}{\overset{d_o}{\frown}} 6.93'' < 1.5 \times 84'' = 126''$
therefore member is considered stiffened

$$\frac{2Dt_w}{b_{ft}t_{ft} + b_{fc}t_{fc}} \le 2.5 \quad (6.10.9.3.2\text{-}1) \qquad \frac{2(84'' \times .375'' \times 2\text{ webs})}{2 \times 48 \times 1.125} = 1.17 < 2.5$$

$$V_n = V_p\left[C + \frac{.87(1-C)}{\sqrt{1+\left(\frac{d_o}{D}\right)^2}}\right] \quad (6.10.9.3.2\text{-}2)$$

where,

$V_p = .58\, F_{yw} D t_w$ (6.10.9.3.2-3)

C = shear buckling resistance ratio per 6.10.9.3.2-4 through 6.10.9.3.2-6
k = shear buckling coefficient per 6.10.9.3.2-7

$V_p = .58 \times 36 \text{ ksi} \times 84'' \times .375'' \times 2 \text{ webs} = 1315.44 \text{ K}$

$$k = 5 + \frac{5}{\left(\frac{d_o}{D}\right)^2} = 5 + \frac{5}{\left(\frac{6.93}{84}\right)^2} = 738$$

$\frac{D}{t_w} = \frac{84''}{.375} = 224$ since $\frac{D}{t_w} < 1.12\sqrt{\frac{Ek}{F_y}}$, $C = 1.0$

$\sqrt{\frac{Ek}{F_y}} = \sqrt{\frac{29000(738)}{36}} = 771.03$

$$V_n = 1315.44\ k\left[1.0 + \frac{.87(1-1)}{\sqrt{1+\left(\frac{6.9}{84}\right)^2}}\right] = 1315.4\ k$$

$$\boxed{\phi V_n = 1315.4\ k}$$ (both webs)

Fatigue Check

Flange CP welds are considered a fatigue category B' per 3.2, table 6.6.1.2.3-1, long stiffener termination locations are considered E per 4.2. Worst case category will be used at each location for analysis

As the steel pier caps are considered fracture critical members, per BDM 306.3.6, and infinite life analysis per AASHTO 6.6.1.2.3

Flange connections:

F_{TH} = 12 ksi per B'

Stress calculation

M (includes LF = 0.75 per BDM Table)

LL max 127.05 k-ft

LL min -1099 k-ft

ΔLL 1226 k-ft → 14713 k-in

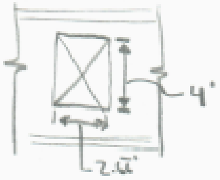
$$\Delta\sigma = \frac{My}{I_{xx}} = \frac{M}{S_{top}}$$

$$= \frac{14713 \text{ k-in}}{5386 \text{ in}^3} = 2.73 \text{ ksi}$$

9.67 ksi < 12 ksi (ok)

Access Hatch reductions due to Web Openings

(φVn and φMn calculated from spreadsheet)



@ mid support

$$\phi V_n = .58 \times 48" \times .375" \times 36 \text{ ksi} = 376 \text{ k}$$

$$\frac{V_n - 376}{V_n} = \frac{1315 - 376}{1315} = 0.71$$ max ratio @ opening

$$\phi M_n = 36 \text{ ksi} \times S_{opening} = 36 \times \frac{.375" \times 48^2}{6} / 12 = 432 \text{ k-ft}$$

$$\frac{M_n - 432}{M_n} = \frac{15464 - 432}{15464} = 0.97$$ max ratio @ opening

$$\frac{V_u}{V_n} = \frac{417}{1315.44} = .32$$ (ok)

$$\frac{M_u}{M_n} = \frac{13121}{15464} = .85 < .97$$ (ok)

Bearing Stiffener Capacity (6.10.11.2)

end bearing

$\phi_b = 1.0$ (6.5.4.2)

$R_{sb} = 1.4\ A_{pn}\ F_{ys}$ (6.10.11.2.3-2)

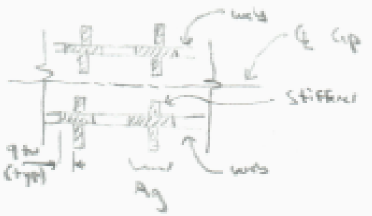
$A_{pn} = (6'' \times 2$ (each side of single web) $\times 1.125'') \times 2$ webs $\times 2$ stiff groups per beam

$= 54\ in^2$ ... excludes projected width of stiffener port holes

$\phi R_{sb} = 1.0\,(1.4 \times 54\ in^2 \times 36\ ksi) = \underline{2722}\ k$

axial capacity (6.9.2.1)

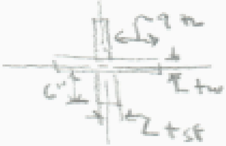
$P_o = F_y\ A_g$, where A_g is defined in 6.10.11.2.4 b due to welded webs connections



$A_g = [(2 \times 6'') \times (1.125'')] + [9 \times \underbrace{.375''}_{tw} \times .375'' \times 2]$

$= 16.03\ in^2$

$P_o = 36\ ksi \times 16.03\ in^2 = 577\ k$



$I_{min} = \underbrace{\frac{1}{12}((2 \times .375 \times 9)^3 \times .375)}_{9.61} + \underbrace{\frac{1}{12}(1.125^3 \times 12)}_{1.42}$

$= 11.16\ in^4$

$k = .75$ (4.6.2.5)

$l_u = .75 \times 36'' + .25 \times 84'' = 48''$ → (porthole is unbraced length added length provided for conservatism)

$r = \sqrt{I/A} = \sqrt{11.16/16.03} = .836$

$\frac{kl}{r} = \frac{.75 \times 48}{.836} = 43.06 < 140$ (ok)

$P_e = \frac{\pi^2 E}{\left(\frac{kl}{r}\right)^2} A_g$ (6.9.4.1.2-1)

$= \frac{\pi^2 \times 29000\ ksi}{(43.06)^2} \times 16.03 = 2474\ k$

$\frac{P_e}{P_o} = \frac{2474}{577} = 4.28 \Rightarrow P_n = \left[.658^{\left(\frac{P_o}{P_e}\right)}\right] P_o$

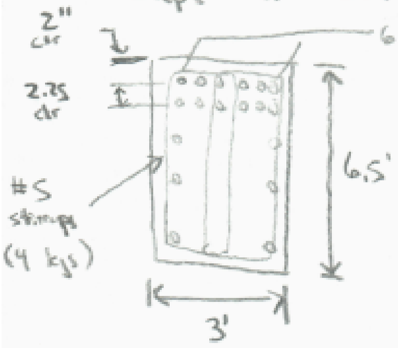
$\phi P_n = 1.0\left[.658^{\left(\frac{1}{4.28}\right)}\right] 577\ k = 523\ k \times 4$ total stiffener groups

$= 2092\ k$

PIER 2 AT LEFT COLUMN SUPPORT - STEEL PIER CAP SAMPLE CAPACITY HAND CALCULATIONS

Capacity Calculations – Concrete Pier Cap

Shape properties @ cantilever support, conservatively ignore variable steps at beam bearings. Positive moment capacity not calculated as negative moment will control by inspection.



$A_{st,\,top} = 2 \times 6 \times 1.56\,in^2 = 18.72\,in^2$

$$\bar{y}_{,\,top} = \frac{6 \times (2+.625" + \frac{1.41}{2}) + 6 \times (2 + .625 + 1.41 + 2.25" + \frac{1.41}{2})}{12}$$

$= 5.16"$

Flexural Capacity (5.6.3.2)

$\phi_F = 0.9$

$d = (6.5 \times 12) - 5.16" = 72.84"$

$c = \dfrac{A_s F_s}{\alpha_1 f'_c \beta_1 b}$ (5.6.3.1.2-4)

$\alpha_1 = .85$ (5.6.2.2)

$\beta_1 = .85$ (5.6.2.2)

$f'_c = 4$ ksi (per Class C in record plans)

$F_s = 40$ ksi (per BDM guidance for plan year)

$$c = \frac{18.72\,in^2 \times 40\,ksi}{.85\,(4\,ksi)\,.85 \times 36"} = 7.2"$$

$a = \beta_1 c = .85 \times 7.2" = 6.12"$

$\varepsilon_s = (d-c)(.003/c) = (72.84" - 7.2")(.003/7.2") = .027\ \frac{in}{in}$

since $\varepsilon_s > .005$ section is tension controlled

$\phi M_n = \phi A_{st} f_s \left(d - \frac{a}{2}\right) = 0.9\left[18.72\,in^2 \times 40\,ksi \left(72.84" - \frac{6.12}{2}\right)\right]/12$

$\boxed{\phi M_n = 3919\ k\text{-}ft}$

Shear Capacity

$\phi_v = 0.9$ (5.5.4.2)

$A_{v\,min} = .0316 \lambda \sqrt{f'_c} \dfrac{b_v s}{f_y}$ (5.7.2.5)

$= .0316 (1.0) \sqrt{4\,ksi} \times \dfrac{36 \times 8''}{40\,ksi} = .455\ in^2/ft$

$A_{v,act} = \dfrac{4 \times .31}{(8''/12)} = 1.86\ in^2/ft$

Since $A_{v,act} > A_{v\,min}$ the simplified procedure per 5.7.3.4.1 may be used, otherwise general procedure per 5.7.3.4.2 used for calculation of shear capacity

$\beta = 2.0$ (5.7.3.4.1)

$V_{n,max} = 0.25 f'_c b_v d_v$ (5.7.3.3-2)

$= .25 (4\,ksi)\ 36'' \times 69.78'' = 2512\ k$

$d_v = \max\left(d - \frac{a}{2},\ .9 \times d,\ .72\,h\right)$

① $72.84 - \dfrac{6.12}{2} = 69.76''$ ⟵ controls

② $.9 \times 72.84 = 65.6''$

③ $.72 \times 6.5' \times 12 = 56.16$

$\phi V_n = \phi (V_c + V_s)$

$V_c = .0316 \beta \lambda \sqrt{f'_c}\ b_v d_v$ (5.7.3.3-3)

$= .0316 (2.0)(1.0) \sqrt{4}\ 36'' \times 69.78'' = 317.5\ k$

$V_s = \dfrac{A_v f_y d_v (\overset{1}{\cancel{\cot(45)}} + \overset{0}{\cancel{\cot(90)}}) \sin \overset{1}{\cancel{(90)}}}{s}$

$= 1.86\ in^2/ft \times 40\ ksi \times 69.78'' = 5191$

$V_n = (318\,k + 5191\,k)$

$= 5509\ k$

$V_{n,max}$ controls

$\boxed{\phi V_n = .9 \times 2512 = 2260\,k}$