SDR ENGINEERING CONSULTANTS, INC.

Peer Review of Macarthur Drive Interchange Completion-Phase1

STATE PROJECT NO. 283-09-0114

Prepared for:

Louisiana Department of Transportation and Development

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Peer Review of Macarthur Drive Interchange Completion-Phase1

A. WORK SCOPE

This report presents the Plan Review of the LA State Project, MacArthur Drive Interchange Completion – Phase 1 (S.P. #283-09-0114). The review work consists of three parts as follows:

- General comments addressing the problems in design and detailing,
- Structural analysis of the Trapezoidal Girders, including the flexural, shear checking for the beam design, and the checking for beam dapped end.
- Structural checking for the design of Pier Cap Ledge.

B. PEER REVIEW- GENERAL COMMENTS

Following are general comments listed by category:

1. New and Old Cap Connection Details

The provided detail is too complicated for no reason. Drill horizontally into existing cap and install #9- 18" long bars (2 to 4 bars should be sufficient). Drilling 9" into existing cap and using epoxy should be sufficient but the designer needs to check. Roughen the surface of the existing cap to expose the aggregate and improve bond quality between the old and new sections.

2. Bearing Seat Elevations

Each box girder is supported over four pads which could create a rocking of the beam and extreme care is required in setting the seat elevations. This is similar to a table with unequal legs. The bearing type specified is very stiff and will not provide sufficient compensation to eliminate the rocking problem. An appropriate construction tolerance should be established and the proper bearing pads should be selected accordingly.

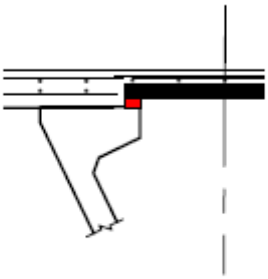
3. Deck Construction Sequence

As mentioned above, the elevations of the four point supports are critical. The deck construction sequence should be provided to avoid any un-balanced loads that could result in altering the seat elevations.

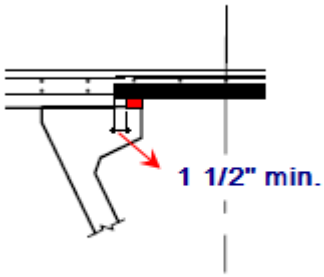
4. Bedding Detail For Stay-in-Place Deck Panels

Sheet # 547 shows the cross section details and the shown bedding of the stay in place precast panel is poor and will lead to undesired cracking of the deck. Please provide details for the

bedding of the precast panel (see below). Proper bedding of the stay-in-place panels is extremely important and a note should be added to alert the CEI to check and ensure proper bedding of these panels. We recommend using other alternatives to the prestressed stay in place panels.



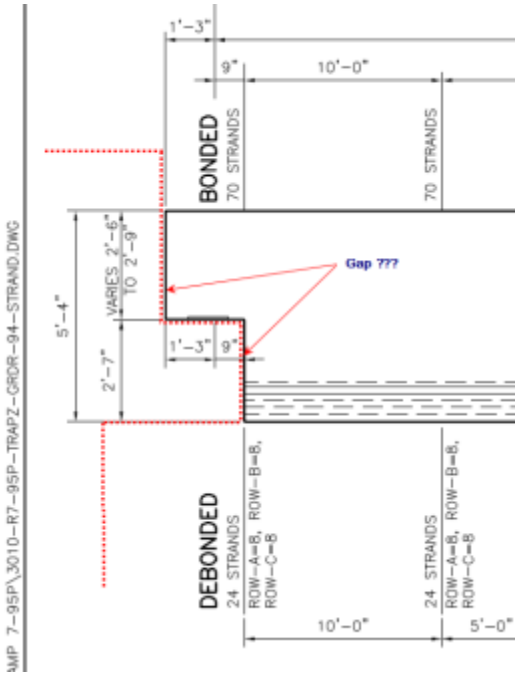
Bad Detail

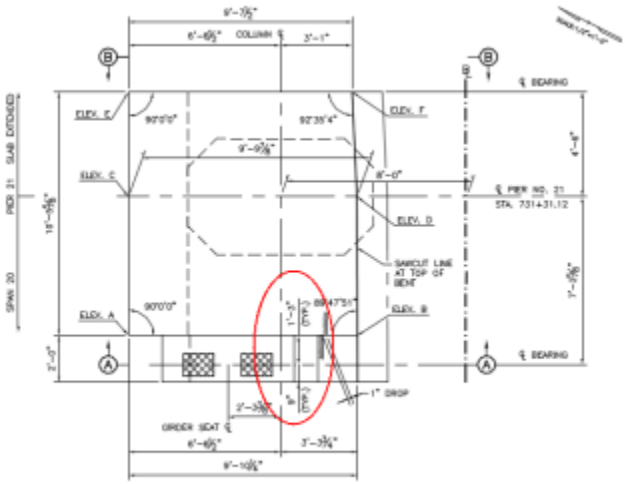


Good Detail

5. Gaps Between Girder and Pier Cap

The plans show no gaps in any direction between the girder and pier cap





The typical girder seat and typical trapezoidal girder end section show no gap between the side of the box and the pier cap (see comments on sheet 317 (attached)). Please note that the typical depth of the box on sheet 317 does not match the box cross section details, please correct.

6. Storm Drainage

- 90° bends will clog. Eliminate all 90° bends and replace with 45° bends.
- Remove drainage from inside the box.

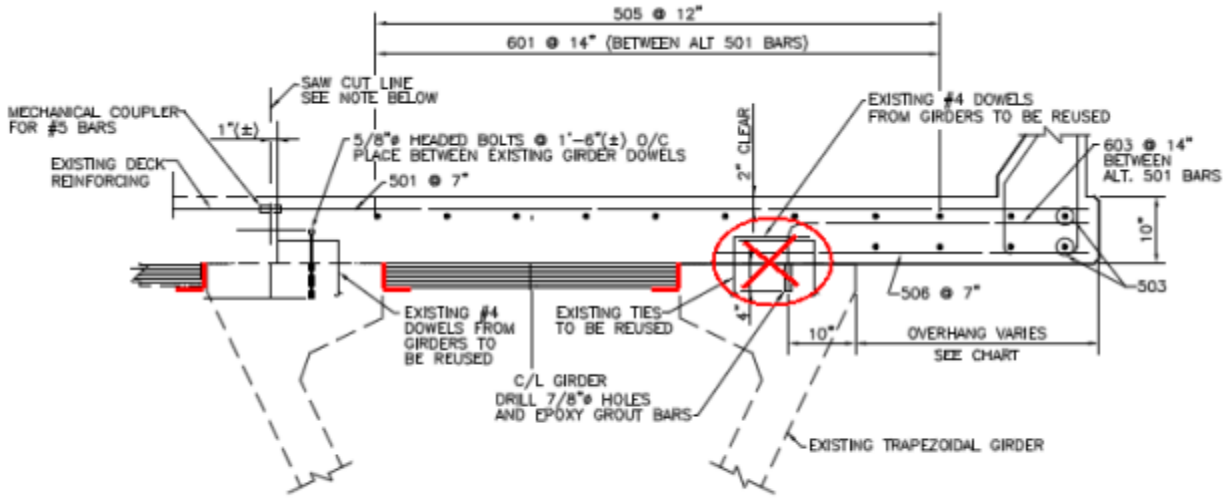
7. Pier Reinforcement Details

Sheet 370 (typical example) shows curved steel bars. Provide a note stating that these #11 bars need to be pre-bend at the yard, avoid bending at the job site.

8. Rail Attachment Details

The plans as shown do not have enough embedment without drilling holes into the box and inserting the steel at varying depth to provide the required embedment as shown in Sheet # 545. This details as shown are not practical and will result in significant labor to drill these holes with the associated increased cost and tight quality control. We understand that the department is looking into using stay in place steel form in the majority of the bridge. We strongly concur with this decision and recommend changing this steel embedment details since it will not be required if stay-in-place steel forms are used.

If there is a need to use the precast stay-in-place panel, we strongly recommend changing the shown details and hanging the panel between the top flanges of the box as shown by the sketch inserted in Sheet # 545. Please note that using this hanging detail will be even simpler if stay-in-place steel forms are used. We believe that even with the cost associated with hanging the panel it will be cheaper than the shown details since the steel development issue will be solved and the need to drill holes is eliminated.



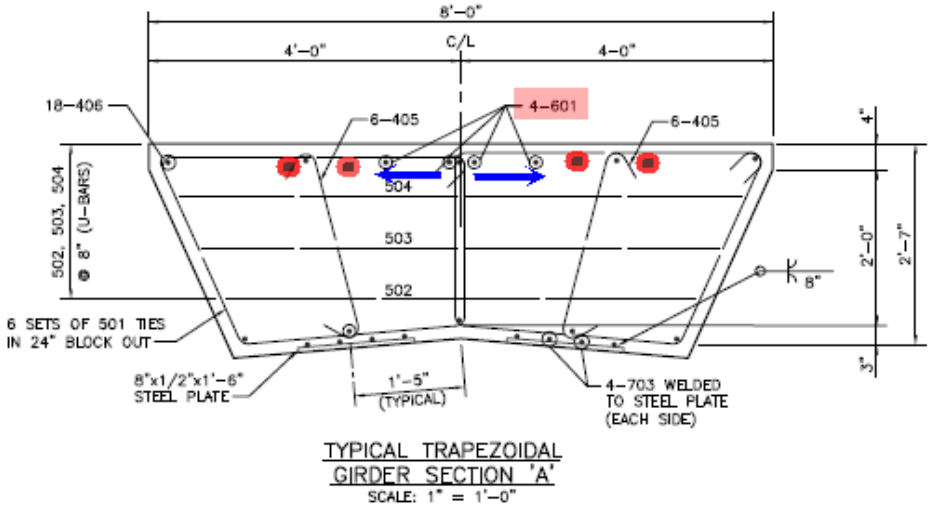
SECTION THRU CANTILEVER DECK SLAB
SCALE: 1"=1'-0"

9. Deck Thickness

The deck thickness shown is 7.5 inches. Does this thickness include the ½" sacrificial concrete for finishing the top surface or is it included?

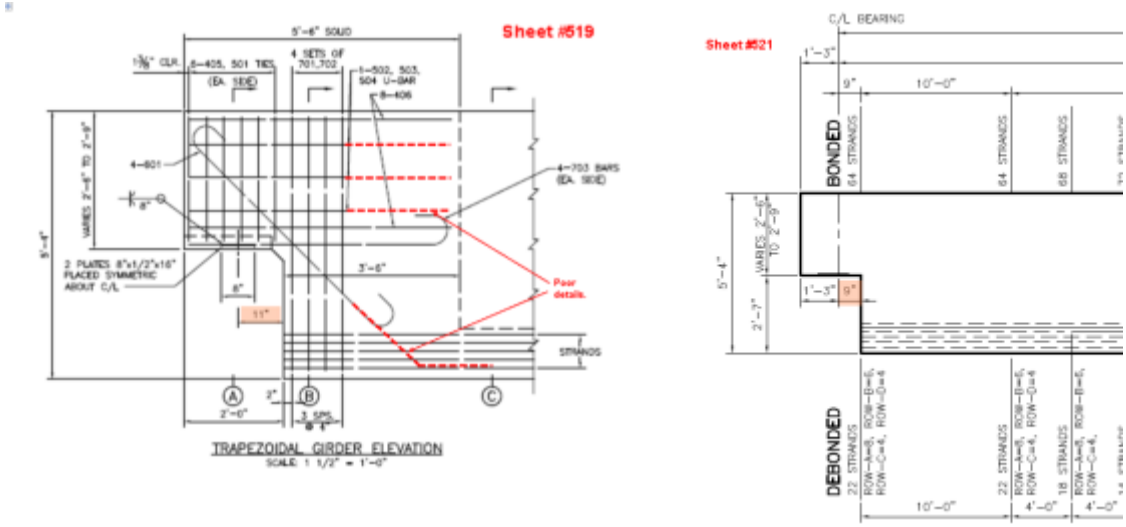
10. Reinforcement Detailing

Sheet # 519: Inclined bars 4-601 are not well-distributed. The bars should be placed closely to the bearing location having critical load path.



11. Problem of Consistency

- Sheet # 519: the distance from bearing CL to the inner face of ledge is 11''. While it is 9'' in Sheets # 521 to # 525.



- Beam length (Joint CL to joint CL) for span 15 to span 25 in Sheet # 526, as below, are not consistent with B2B Length in Sheet #521 to #525

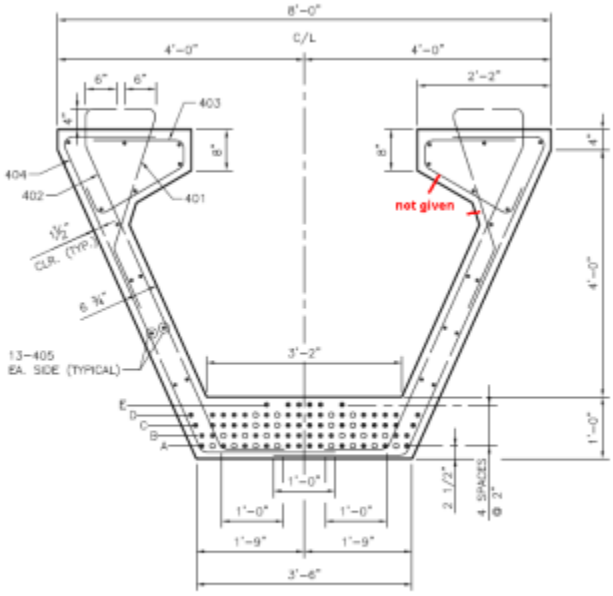
Sheet # 526

RAMP 7 SPAN NO.	GIRDER NO.	℄ JOINT TO ℄ JOINT MEASURED HORIZ.	LENGTH OF GIRDER HORIZONTAL PROJECTION	TRUE LENGTH OF GIRDER	GIRDER TYPE	STRAND PATTERN	CONCRETE STRENGTH @ RELEASE	28 DAY CONCRETE STRENGTH	TIE DOWN POINT
		Lc	L_H	L_T			f'ci(ksi)	f'c(ksi)	≥
S15	G15A	118'-4 ¼"	120'-7 ¼"	120'-8 ¾"	TRAPZ.	94-2	5.6	7.0	
S15	G15B	118'-4 ¼"	120'-7 ¼"	119'-11 ¾"	TRAPZ.	94-2	5.6	7.0	
S16	G16A	123'-9 ⅜"	126'-0 ⅜"	126'-0 ⅝"	TRAPZ.	96-3	5.6	7.0	
S16	G16B	123'-9 ⅜"	126'-0 ⅜"	126'-0 ⅝"	TRAPZ.	96-3	5.6	7.0	
S17	G17	124'-0"	126'-3"	126'-3"	TRAPZ.	96-3	5.6	7.0	
S18	G18	121'-3 ⅛"	123'-6 ⅛"	123'-6 ⅛"	TRAPZ.	96-3	5.6	7.0	
S19	G19	123'-6 ¾"	125'-10 ½"	125'-10 ½"	TRAPZ.	96-3	5.6	7.0	
S20	G20	121'-11"	124'-2 ¾"	124'-3 ⅜"	TRAPZ.	96-3	5.6	7.0	
S21	G21	132'-9 ⅝"	135'-0 ¾"	135'-2 ¼"	TRAPZ.	100-4	5.6	7.0	
S22	G22	132'-6 ¾"	134'-9 ¾"	135'-0 ⅛"	TRAPZ.	100-4	5.6	7.0	
S23	G23	116'-7"	118'-10 ⅛"	119'-0 ¼"	TRAPZ.	94-2	5.6	7.0	
S24	G24	138'-11 ¼"	141'-2 ¼"	141'-4 ¼"	TRAPZ.	114-5	5.6	7.0	
S25	G25	102'-0 ⅝"	104'-3 ⅝"	104'-5 ⅛"	TRAPZ.	86-1	5.6	7.0	

Sheet #521-#525: B2B Length is 102,116,124,132,139 ft, respectively.

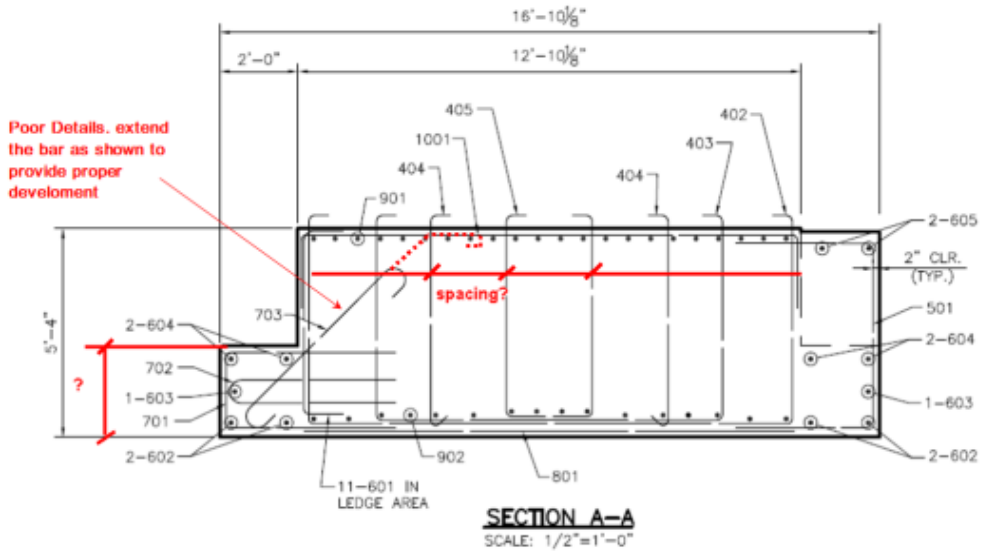
12. Dimension Missing/Error

- Sheet # 520: Dimension needed, see below.

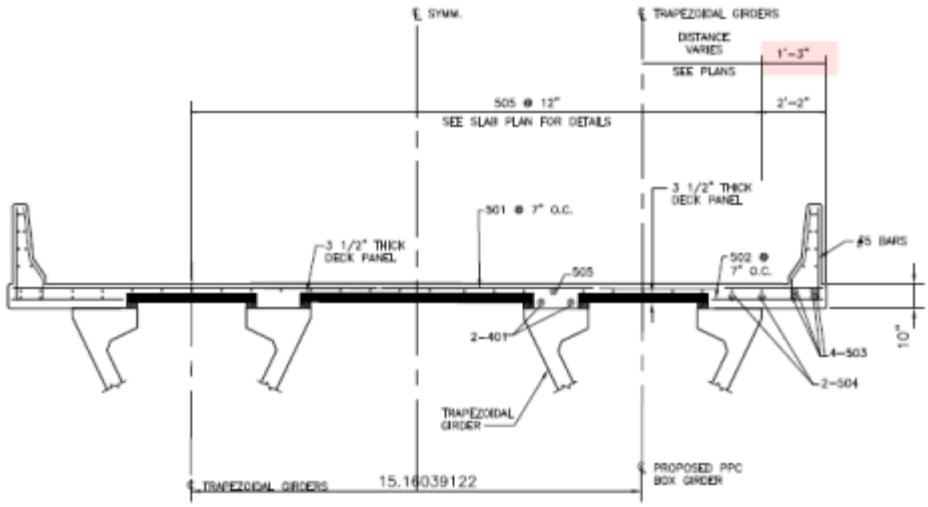


TYPICAL TRAPEZOIDAL GIRDER SECTION 'C'
SCALE: 1" = 1'-0"

- Sheet # 319: Dimensions needed-see below.



- Sheets #547 and #935: dimension for the barrier width (1'-3'') is not shown correctly, see below.



C. ANALYSIS OF TRAPEZOIDAL GIRDERS

1. Limit of Debonded strands

AASHTO LRFD Specification states: *The number of partially debonded strands should not exceed 25 percent of the total number of strands.* The following beams violate this requirement; however, we believe the difference is insignificant and should be allowed.

- Sheet # 521: debond 22 strands out of 86 strands. 22/86= 25.6 >25% NS
- Sheet # 522: debond 24 strands out of 94 strands. 24/94= 25.53 >25% NS
- Sheet # 525: debond 30 strands out of 114 strands. 30/114= 26.31>25% NS

NS: Not Significant

While the debonding limit is exceeded we believe that the increase is not significant to affect the overall performance of the girders.

2. Flexural And Shear Ratings

Flexural and shear ratings of ramp 7 - spans #15, #16 and #24 are performed by Smart Bridge Suite (SBS) based on the 95% Plan, to check the strength and serviceability. Assumptions and details of the load rating analysis are given in Appendix A1. In the analysis, Span 24 is modeled as bridge with five girders, while spans 15&16 are modeled as 2-girder bridge with 15.16' spacing between girders.

2.1 Analysis for strength and service:

Span #15:	LASDV #5	Service III is not satisfied for (R.F.=0.95).
Span #16:	HL93	Service III is not satisfied for (R.F.=0.70)
	LASDV 3	Service III is not satisfied for (R.F.=0.95)
	LASDV 4	Service III is not satisfied for (R.F.=0.85)
	LASDV 5	Service III is not satisfied for (R.F.=0.69)
	LASDV 7	Service III is not satisfied for (R.F.=0.94)
	LASDV 8	Service III is not satisfied for (R.F.=0.91)
Span #24:	HL93	Service III is not satisfied for (R.F.=0.65)
	LASDV 4	Service III is not satisfied for (R.F.=0.98)
	LASDV 5	Service III is not satisfied for (R.F.=0.78)
	LASDV 8	Service III is not satisfied for (R.F.=0.96)

Results Summary; *In general, Service III is not met for trapezoidal girders and the designer should consider the possibility of adding more strands, however, the debonding level is already exceeded and harping the strands or increasing the section size might be necessary.*

2.2 Analysis of the Beam dapped ends:

The beam dapped ends are analyzed using Strut and Tie Modeling (STM) method for checking the strength and reinforcement details. The procedure and results are presented in Appendix A2

Results Summary; *The provided reinforcement does not meet loading requirements. Additional and better detailing of the reinforcement is required.*

D. ANALYSIS OF PIER CAP LEDGE

The beam dapped end is analyzed by the Strut and Tie Modeling (STM) method for checking the strength and detailing. The analysis procedure and results are presented in Appendix B.

Results Summary; *The provided reinforcement does not meet loading requirements. Additional and better detailing of the reinforcement is required.*

APPENDIX A ANALYSIS OF TRAPEZOIDAL GIRDERS

A1 Flexural and Shear Checking

Smart Bridge Suite (SBS) Program is used for the analysis of the Trapezoidal Girders, including the ones with B2B length of 118ft, 124ft and 139ft. The girder framing plan, strands pattern, typical bridge deck section, and prestressed girder details are taken from the plan, as shown below.

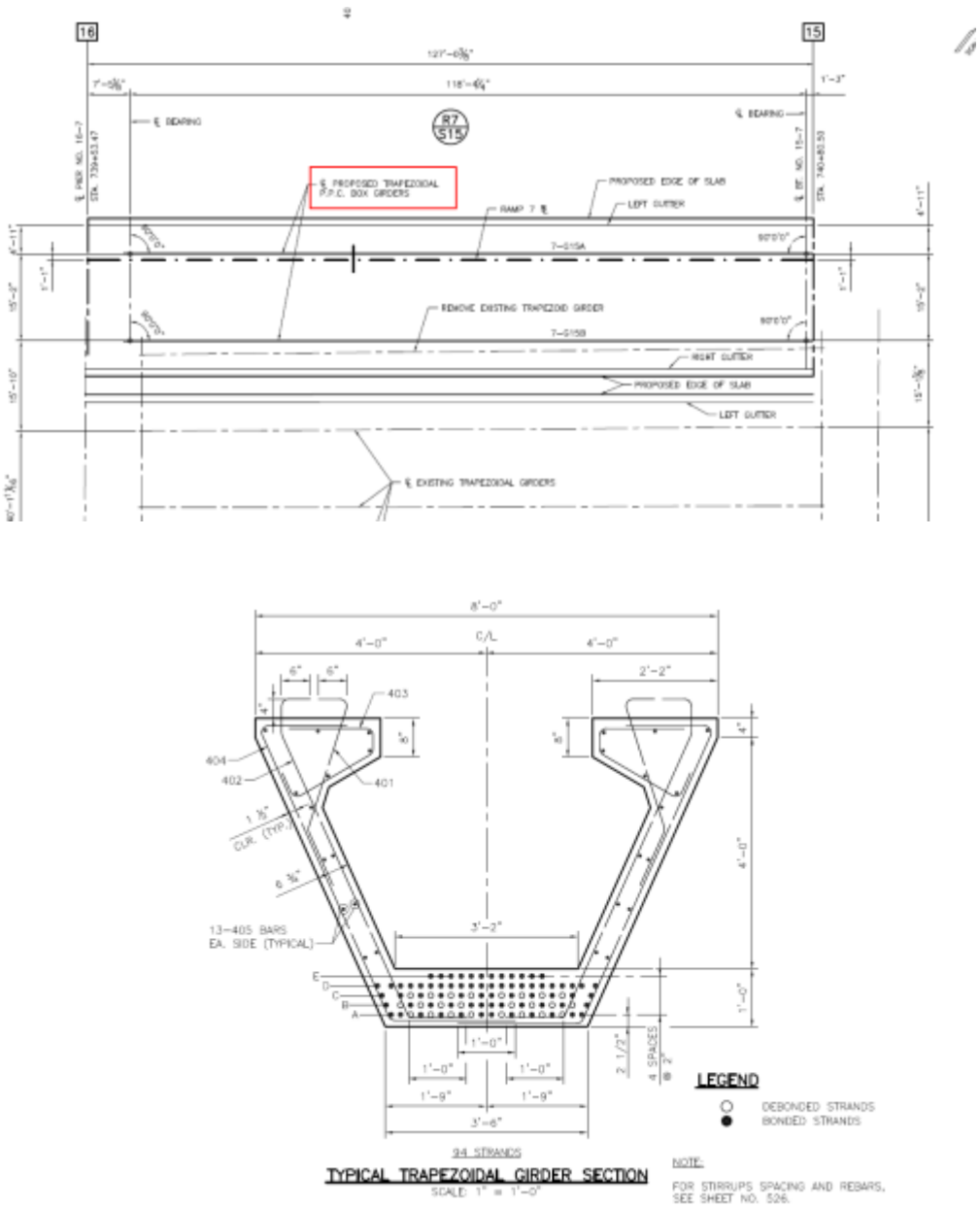


Fig. A.1 Girder Framing and Strands Pattern (118 ft B2B, Span 15)

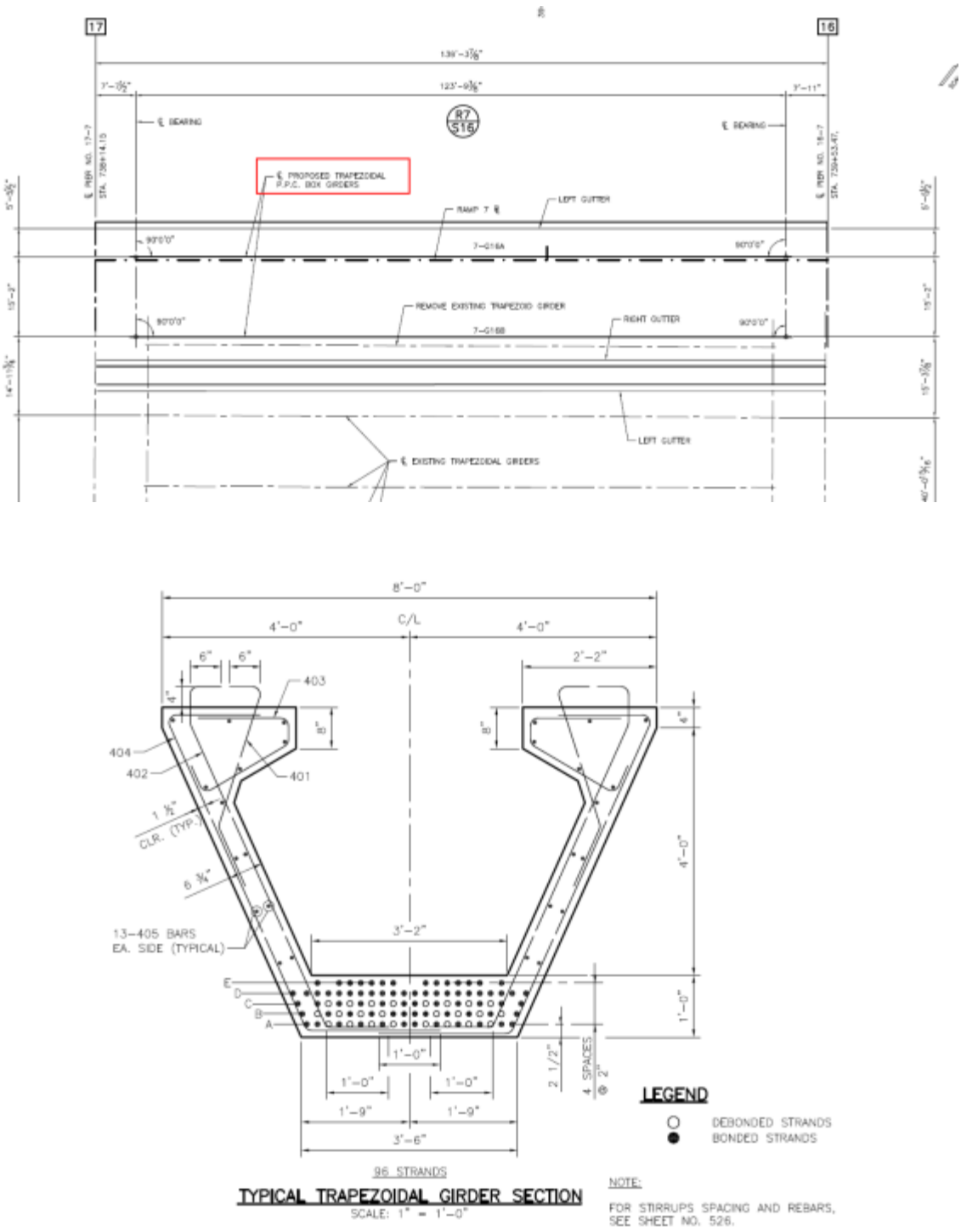
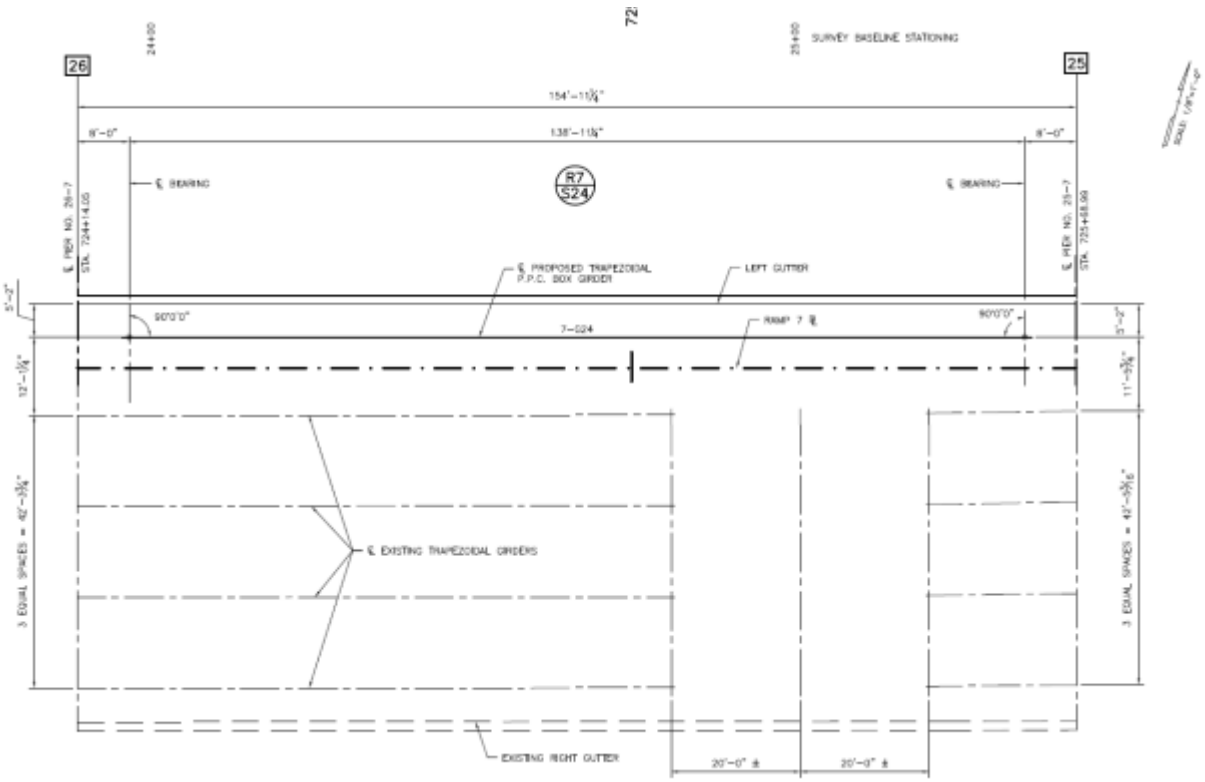


Fig. A.2 Girder Framing and Strands Pattern (124 ft B2B, Span 16)



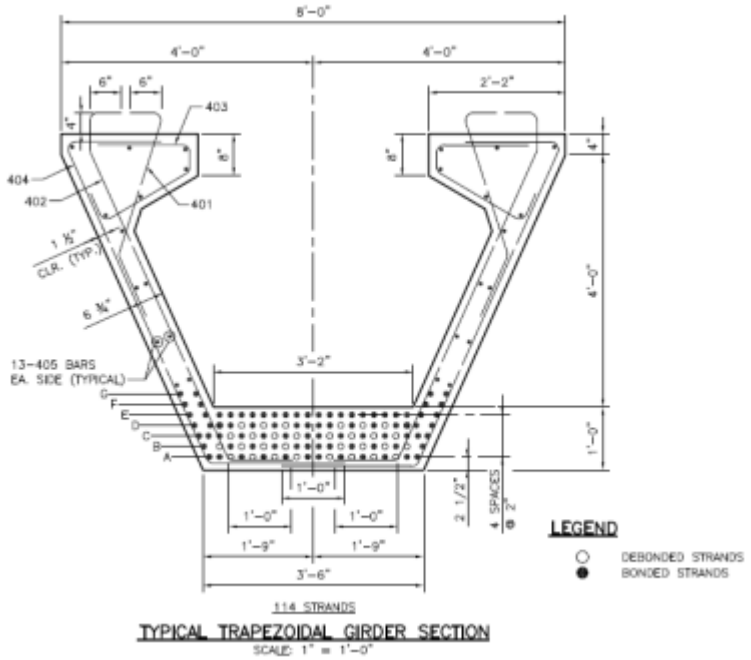


Fig. A.3 Girder Framing and Strands Pattern (139 ft B2B, Span 24)

Ramp 7 -Spans 15 & 16 consists of two trapezoidal girders spaced at 15.16 feet on centers, respectively, while Ramp 7- Span24 has only one trapezoidal girder added for widening.

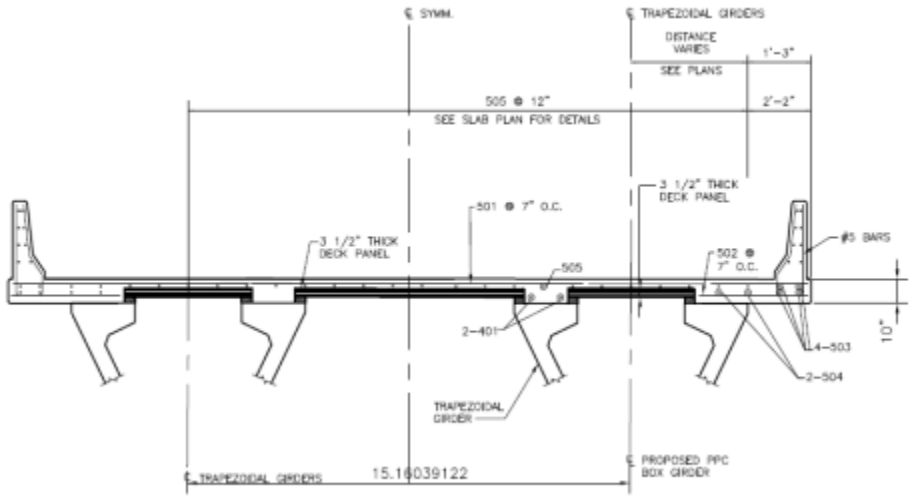


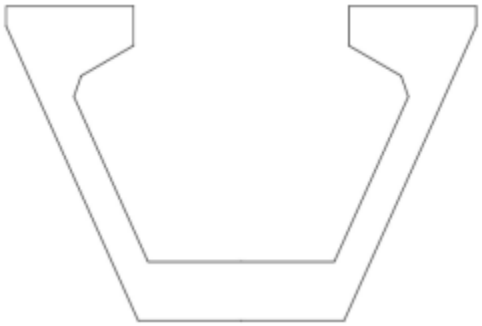
Fig. A.4 Typical Deck Section (Spans 15 &16)

The details of materials properties, strand type and stirrups spacing, etc. are summarized in the plan Sheet #526, as partially shown below.

Table A.1 Prestressed Girder Schedule

RAMP 7 SPAN NO.	GIRDER NO.	℄ JOINT TO ℄ JOINT MEASURED HORIZ.	LENGTH OF GIRDER HORIZONTAL PROJECTION	TRUE LENGTH OF GIRDER	GIRDER TYPE	STRAND PATTERN	CONCRETE STRENGTH @ RELEASE	28 DAY CONCRETE STRENGTH	TIE DOWN PORT	STIRRUP SPACING	STIRRUP SPACING	STIRRUP SPACING	STIRRUP SPACING	SPACING BETWEEN GIRDERS
		Lc	L_H	L_T			f'c(ksi)	f'c(ksi)	#	N1 SPA. @ S1 = W1	N2 SPA. @ S2 = W2	N3 SPA. @ S3 = W3	N4 SPA. @ S4 = W4	S
S15	G15A	118'-4 ¼"	120'-7 ¼"	120'-8 ¾"	TRAPZ.	94-2	5.6	7.0		13 @ 4" = 4'-4"	73 @ 5" = 30'-5"	46 @ 6" = 24'-0"		5 1/8"
S15	G15B	118'-4 ¼"	120'-7 ¼"	119'-11 ¾"	TRAPZ.	94-2	5.6	7.0		13 @ 4" = 4'-4"	73 @ 5" = 30'-5"	46 @ 6" = 24'-0"		5 1/8"
S16	G16A	123'-9 ⅜"	126'-0 ⅜"	126'-0 ⅜"	TRAPZ.	96-3	5.6	7.0		55 @ 4" = 18'-4"	66 @ 5" = 27'-6"	32 @ 6" = 16'-0"		0 11/16"
S16	G16B	123'-9 ⅜"	126'-0 ⅜"	126'-0 ⅜"	TRAPZ.	96-3	5.6	7.0		55 @ 4" = 18'-4"	66 @ 5" = 27'-6"	32 @ 6" = 16'-0"		0 11/16"
S17	G17	124'-0"	126'-3"	126'-3"	TRAPZ.	96-3	5.6	7.0		55 @ 4" = 18'-4"	66 @ 5" = 27'-6"	32 @ 6" = 16'-0"		2"
S18	G18	121'-3 ⅛"	123'-6 ⅛"	123'-6 ⅛"	TRAPZ.	96-3	5.6	7.0		55 @ 4" = 18'-4"	66 @ 5" = 27'-6"	29 @ 6" = 14'-6"		3 5/8"
S19	G19	123'-6 ⅜"	125'-10 ⅜"	125'-10 ⅜"	TRAPZ.	96-3	5.6	7.0		55 @ 4" = 18'-4"	66 @ 5" = 27'-6"	31 @ 6" = 15'-6"		5 3/8"
S20	G20	121'-11"	124'-2 ⅜"	124'-3 ⅜"	TRAPZ.	96-3	5.6	7.0		55 @ 4" = 18'-4"	66 @ 5" = 27'-6"	30 @ 6" = 15'-0"		1 1/2"
S21	G21	132'-9 ⅜"	135'-0 ⅜"	135'-2 ⅜"	TRAPZ.	100-4	5.6	7.0		41 @ 4" = 13'-8"	66 @ 5" = 27'-6"	37 @ 6" = 18'-6"	8 @ 9" = 6'-0"	8 7/8"
S22	G22	132'-6 ⅜"	134'-9 ⅜"	135'-0 ⅜"	TRAPZ.	100-4	5.6	7.0		41 @ 4" = 13'-8"	66 @ 5" = 27'-6"	37 @ 6" = 18'-6"	8 @ 9" = 6'-0"	7 3/8"
S23	G23	116'-7"	118'-10 ⅜"	119'-0 ⅜"	TRAPZ.	94-2	5.6	7.0		13 @ 4" = 4'-4"	73 @ 5" = 30'-5"	46 @ 6" = 23'-0"		6 1/2"
S24	G24	138'-11 ⅜"	141'-2 ⅜"	141'-4 ⅜"	TRAPZ.	114-5	5.6	7.0		47 @ 4" = 15'-8"	74 @ 5" = 30'-10"	41 @ 6" = 20'-6"	4 @ 7" = 2'-4"	1 5/8"
S25	G25	102'-0 ⅜"	104'-3 ⅜"	104'-5 ⅜"	TRAPZ.	86-1	5.6	7.0		67 @ 5" = 27'-11"	46 @ 6" = 23'-0"			1 5/16"

Figure A.5 shows the section properties of the trapezoidal girder. Note that the trapezoidal girder section is not available in SBS for which *an equivalent section* is generated in SBS, as shown in Figure A.6. This equivalent section has the same section properties and has no effect on the final design.



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----------------   REGIONS   ----------------

Area:                          1766.84 sq in
Perimeter:                     394.88 in
Bounding box:              X: -47.97  --  47.97 in
                           Y: 0.00  --  64.00 in
Centroid:                  X: 0.00 in
                           Y: 32.58 in
Moments of inertia:        X: 2753502.61 sq in sq in
                           Y: 1483741.07 sq in sq in
Product of inertia:       XY: 0.00 sq in sq in
Radii of gyration:         X: 39.48 in
                           Y: 28.98 in
Principal moments (sq in sq in) and X-Y directions about
centroid:
                           I: 877559.19 along [1.00 0.00]
                           J: 1483741.07 along [0.00 1.00]
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Fig. A.5 Section Properties of trapezoidal girder

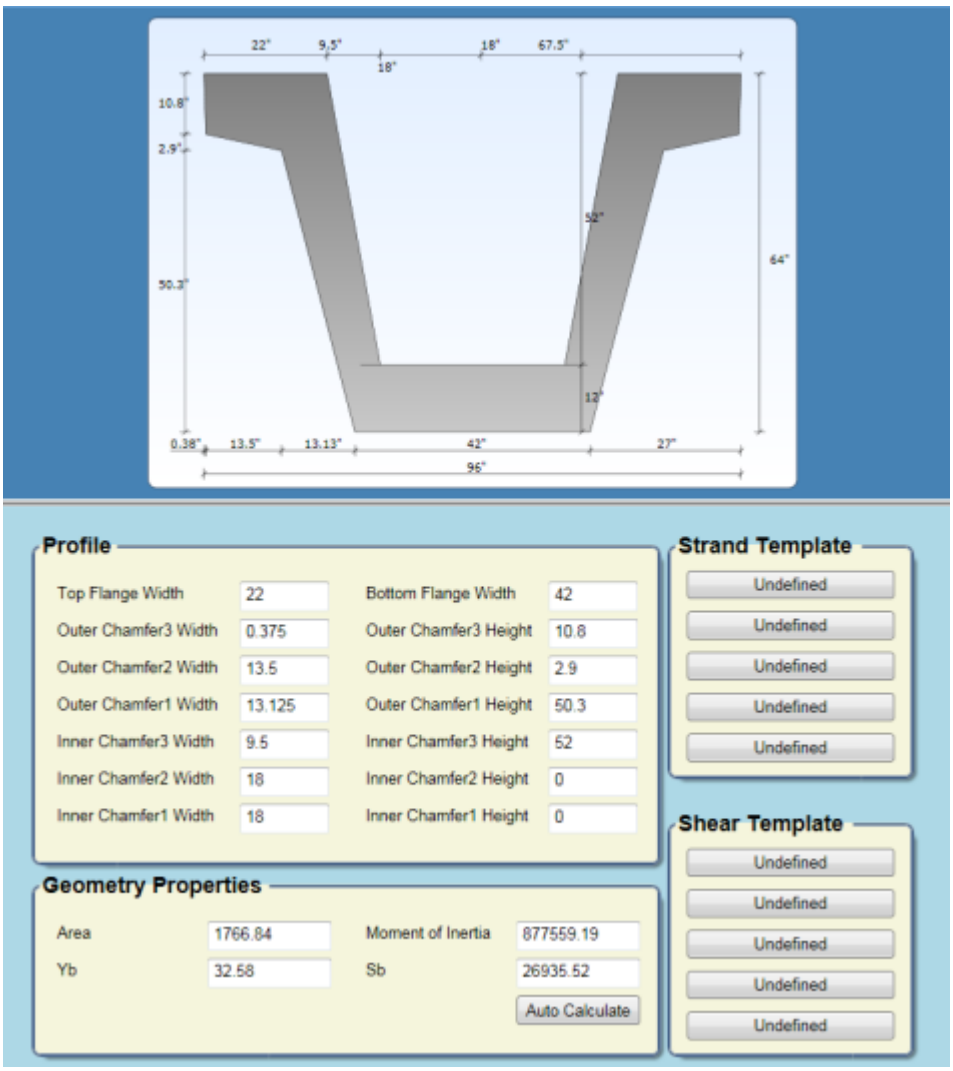


Figure A.6 Equivalent section used in SBS

Load Rating Assumptions

The load ratings were performed based on AASHTO LRFD specifications 2007 with 2009 Interim. The following assumptions were used in the analysis:

- Allowable concrete tension stress (Inv.): $6\sqrt{f'_c}$
- Allowable concrete tension stress (Op.): $7.5\sqrt{f'_c}$
- The materials properties used in the analytical investigation are shown in Table A.2 below.

Table A.2 Materials Properties used in load rating

Concrete		
Concrete Element	Girder	Deck
Concrete Strength at Prestress Transfer	5600 psi	N/A
Concrete Elastic Modulus at Transfer	4536.75 ksi	N/A
28 Days Concrete Strength	7000psi	4000 psi
28 Days Concrete Elastic Modulus	5072.24 ksi	3834.25 ksi
Concrete Weight	150 lbs/ft^3	150 lbs/ft^3
Prestressing Strands		
Strand Diameter	0.5 inch	
Strand Type	Low Relaxation	
Strand Strength	270 ksi	
Yield strength	243 ksi	
Strand Elastic Modulus	28,500 ksi	
Steel Stirrups		
Size of steel bar	#4	
Yield strength	60 ksi	
Elastic Modulus	29,000 ksi	

Ramp 7 Traffic Data from the Plan

2011 ADT=5472; 2031 ADT=8131

D=100%; K=10%; T=13% then, ***ADTT=8131*13%=1057.03***

In accordance with Table A.3 (Permit Load Factors) specified in LADOTD "The policies and guidelines for bridge rating and evaluation" (March 3, 2009), the load factors (Strength II) for LASDVs can be interpolated as shown in Table A.3.

Table A.3 Strength II Load Factors for LASDVs

Loading	Load Factor (Based on ADTT=1057.03)
LASDV-1	1.54
LASDV-2	1.60
LASDV-3	1.34
LASDV-4	1.26
LASDV-5	1.20
LASDV-6	1.20
LASDV-7	1.20
LASDV-8	1.20

Table A.4 gives the load factors for the Service III limit state used in the load rating.

Table A.4 Service III Load Factors for LASDVs

Loading	Load Factor
HL-93 (Inv.)	1.0
LASDV-1	0.9
LASDV-2	0.9
LASDV-3	0.9
LASDV-4	0.9
LASDV-5	0.9
LASDV-6	0.9
LASDV-7	0.9
LASDV-8	0.9

In addition, in accordance with Table 3 (Permit Load Factors) specified in LADOTD "The policies and guidelines for bridge rating and evaluation" (March 3, 2009), Two or more lanes Distribution Factors are used for the rating of LASDVs.

With the above information, the ramp 7 - spans 15, 16, 24 are rated by SBS.

Table A.5 Load Rating Summary --Span 15 (Based on sheet 522, 116' span length B2B)

Vehicle	Limit State	Weight in Tons	Load Factor			DF	RF	Tons	Location
			LL	DL	DW				
HL93	Moment Strength I (Inv)	36	1.75	1.25	1.5	1.19	1.58	56.99	Span 1 Beam 1(0.500 L)
	Moment Strength I (Opt)	36	1.35	1.25	1.5	1.19	2.05	73.88	Span 1 Beam 1(0.500 L)
	Shear Strength I (Inv)	36	1.75	1.25	1.5	1.277	7.30	262.74	Span 1 Beam 1(0.041 L)
	Shear Strength I (Opt)	36	1.35	1.25	1.5	1.277	9.46	340.59	Span 1 Beam 1(0.041 L)
	Stress Service III (Inv)	36	1	1	1	1.19	1.00	36.10	Span 1 Beam 1(0.500 L)
	Stress Service I (Opt)	36	1	1	1	1.19	2.73	98.39	Span 1 Beam 1(0.500 L)
LASDV 1	Moment Strength II	54	1.54	1.25	1.5	0.923	2.21	119.34	Span 1 Beam 1(0.500 L)
	Shear Strength II	54	1.54	1.25	1.5	1.277	7.76	419.04	Span 1 Beam 1(0.041 L)
	Stress Service III	54	0.9	1	1	0.923	1.50	81.16	Span 1 Beam 1(0.500 L)
	Stress Service I	54	1	1	1	0.923	3.36	181.66	Span 1 Beam 1(0.500 L)
LASDV 2	Moment Strength II	44	1.6	1.25	1.5	0.923	2.87	126.28	Span 1 Beam 1(0.500 L)
	Shear Strength II	44	1.6	1.25	1.5	1.277	9.79	430.76	Span 1 Beam 1(0.959 L)
	Stress Service III	44	0.9	1	1	0.923	2.02	89.06	Span 1 Beam 1(0.500 L)
	Stress Service I	44	1	1	1	0.923	4.53	199.32	Span 1 Beam 1(0.500 L)
LASDV 3	Moment Strength II	66.35	1.34	1.25	1.5	0.923	2.17	143.98	Span 1 Beam 1(0.500 L)
	Shear Strength II	66.35	1.34	1.25	1.5	1.277	7.41	491.65	Span 1 Beam 1(0.959 L)
	Stress Service III	66.35	0.9	1	1	0.923	1.28	84.99	Span 1 Beam 1(0.500 L)
	Stress Service I	66.35	1	1	1	0.923	2.87	190.16	Span 1 Beam 1(0.500 L)
LASDV 4	Moment Strength II	71.25	1.26	1.25	1.5	0.923	2.07	147.49	Span 1 Beam 1(0.500 L)
	Shear Strength II	71.25	1.26	1.25	1.5	1.277	7.42	528.68	Span 1 Beam 1(0.041 L)
	Stress Service III	71.25	0.9	1	1	0.923	1.15	81.94	Span 1 Beam 1(0.500 L)
	Stress Service I	71.25	1	1	1	0.923	2.57	183.40	Span 1 Beam 1(0.500 L)
LASDV 5	Moment Strength II	104.5	1.2	1.25	1.5	0.923	1.80	187.58	Span 1 Beam 1(0.500 L)
	Shear Strength II	104.5	1.2	1.25	1.5	1.277	6.64	693.78	Span 1 Beam 1(0.959 L)
	Stress Service III	104.5	0.9	1	1	0.923	0.95	99.28	Span 1 Beam 1(0.500 L)
	Stress Service I	104.5	1	1	1	0.923	2.13	222.17	Span 1 Beam 1(0.500 L)

Table A.5 Load Rating Summary --Span 15 (Based on sheet 522, 116' span length B2B)

Vehicle	Limit State	Weight in Tons	Load Factor			DF	RF	Tons	Location
			LL	DL	DW				
LASDV 6	Moment Strength II	90	1.2	1.25	1.5	0.923	2.65	238.86	Span 1 Beam 1(0.600 L)
	Shear Strength II	90	1.2	1.25	1.5	1.277	7.89	709.74	Span 1 Beam 1(0.041 L)
	Stress Service III	90	0.9	1	1	0.923	1.41	127.26	Span 1 Beam 1(0.500 L)
	Stress Service I	90	1	1	1	0.923	3.08	277.10	Span 1 Beam 1(0.600 L)
LASDV 7	Moment Strength II	130	1.2	1.25	1.5	0.923	2.50	324.68	Span 1 Beam 1(0.500 L)
	Shear Strength II	130	1.2	1.25	1.5	1.277	7.63	991.36	Span 1 Beam 1(0.041 L)
	Stress Service III	130	0.9	1	1	0.923	1.32	171.76	Span 1 Beam 1(0.500 L)
	Stress Service I	130	1	1	1	0.923	2.96	384.38	Span 1 Beam 1(0.500 L)
LASDV 8	Moment Strength II	120	1.2	1.25	1.5	0.923	2.44	292.55	Span 1 Beam 1(0.500 L)
	Shear Strength II	120	1.2	1.25	1.5	1.277	7.52	901.81	Span 1 Beam 1(0.959 L)
	Stress Service III	120	0.9	1	1	0.923	1.29	154.76	Span 1 Beam 1(0.500 L)
	Stress Service I	120	1	1	1	0.923	2.87	344.60	Span 1 Beam 1(0.600 L)

It can be seen that only the load rating factor for LASDV #5 under service III limit state is 0.95 which is lower the minimum acceptable RF of 1.0.

Table A.6 Load Rating Summary --Span 16 (Based on sheet 523, 124' span length B2B)

Vehicle	Limit State	Weight in Tons	Load Factor			DF	RF	Tons	Location
			LL	DC	DW				
HL93	Moment Strength I (Inv)	36	1.75	1.25	1.5	1.194	1.33	47.77	Span 1 Beam 1(0.500 L)
	Moment Strength I (Opt)	36	1.35	1.25	1.5	1.194	1.72	61.92	Span 1 Beam 1(0.500 L)
	Shear Strength I (Inv)	36	1.75	1.25	1.5	1.275	7.99	287.72	Span 1 Beam 1(0.042 L)
	Shear Strength I (Opt)	36	1.35	1.25	1.5	1.275	10.36	372.97	Span 1 Beam 1(0.042 L)
	Stress Service III (Inv)	36	1	1	1	1.194	0.70	25.17	Span 1 Beam 1(0.500 L)
	Stress Service I (Opt)	36	1	1	1	1.194	2.46	88.53	Span 1 Beam 1(0.500 L)
LASDV 1	Moment Strength II	54	1.54	1.25	1.5	0.909	1.93	103.97	Span 1 Beam 1(0.500 L)
	Shear Strength II	54	1.54	1.25	1.5	1.275	8.64	466.56	Span 1 Beam 1(0.958 L)
	Stress Service III	54	0.9	1	1	0.909	1.12	60.26	Span 1 Beam 1(0.500 L)
	Stress Service I	54	1	1	1	0.909	3.14	169.61	Span 1 Beam 1(0.500 L)
LASDV 2	Moment Strength II	44	1.6	1.25	1.5	0.909	2.48	109.07	Span 1 Beam 1(0.500 L)
	Shear Strength II	44	1.6	1.25	1.5	1.275	10.86	477.81	Span 1 Beam 1(0.958 L)
	Stress Service III	44	0.9	1	1	0.909	1.49	65.65	Span 1 Beam 1(0.500 L)
	Stress Service I	44	1	1	1	0.909	4.20	184.76	Span 1 Beam 1(0.500 L)
LASDV 3	Moment Strength II	66.35	1.34	1.25	1.5	0.909	1.88	124.72	Span 1 Beam 1(0.500 L)
	Shear Strength II	66.35	1.34	1.25	1.5	1.275	8.25	547.18	Span 1 Beam 1(0.958 L)
	Stress Service III	66.35	0.9	1	1	0.909	0.95	62.90	Span 1 Beam 1(0.500 L)
	Stress Service I	66.35	1	1	1	0.909	2.67	176.96	Span 1 Beam 1(0.500 L)
LASDV 4	Moment Strength II	71.25	1.26	1.25	1.5	0.909	1.80	128.18	Span 1 Beam 1(0.500 L)
	Shear Strength II	71.25	1.26	1.25	1.5	1.275	8.25	587.51	Span 1 Beam 1(0.042 L)
	Stress Service III	71.25	0.9	1	1	0.909	0.85	60.80	Span 1 Beam 1(0.500 L)
	Stress Service I	71.25	1	1	1	0.909	2.40	171.07	Span 1 Beam 1(0.500 L)
LASDV 5	Moment Strength II	104.5	1.2	1.25	1.5	0.909	1.54	160.57	Span 1 Beam 1(0.500 L)
	Shear Strength II	104.5	1.2	1.25	1.5	1.275	7.27	759.29	Span 1 Beam 1(0.958 L)
	Stress Service III	104.5	0.9	1	1	0.909	0.69	72.51	Span 1 Beam 1(0.500 L)
	Stress Service I	104.5	1	1	1	0.909	1.95	204.04	Span 1 Beam 1(0.500 L)

Table A.6 Load Rating Summary --Span 16 (Based on sheet 523, 124' span length B2B)

Vehicle	Limit State	Weight in Tons	Load Factor			DF	RF	Tons	Location
			LL	DC	DW				
LASDV 6	Moment Strength II	90	1.2	1.25	1.5	0.909	2.25	202.53	Span 1 Beam 1(0.500 L)
	Shear Strength II	90	1.2	1.25	1.5	1.275	8.62	776.06	Span 1 Beam 1(0.042 L)
	Stress Service III	90	0.9	1	1	0.909	1.02	91.46	Span 1 Beam 1(0.500 L)
	Stress Service I	90	1	1	1	0.909	2.78	250.27	Span 1 Beam 1(0.400 L)
LASDV 7	Moment Strength II	130	1.2	1.25	1.5	0.909	2.08	270.01	Span 1 Beam 1(0.500 L)
	Shear Strength II	130	1.2	1.25	1.5	1.275	8.09	1051.18	Span 1 Beam 1(0.042 L)
	Stress Service III	130	0.9	1	1	0.909	0.94	121.94	Span 1 Beam 1(0.500 L)
	Stress Service I	130	1	1	1	0.909	2.64	343.12	Span 1 Beam 1(0.500 L)
LASDV 8	Moment Strength II	120	1.2	1.25	1.5	0.909	2.01	241.19	Span 1 Beam 1(0.500 L)
	Shear Strength II	120	1.2	1.25	1.5	1.275	8.08	969.13	Span 1 Beam 1(0.958 L)
	Stress Service III	120	0.9	1	1	0.909	0.91	108.92	Span 1 Beam 1(0.500 L)
	Stress Service I	120	1	1	1	0.909	2.55	306.50	Span 1 Beam 1(0.500 L)

It can be seen that load rating factor for HL-93 (Inv.) and LASDV #3,4,5,7,8 under service III limit state are lower than 1.0, while the rating factors under Strength limit state are greater than 1.0.

Table A.7 Load Rating Summary --Span 24 (based on sheet 525, 139' span length B2B)

Vehicle	Limit State	Weight in Tons	Load Factor			DF	RF	Tons	Location
			LL	DL	DW				
HL93	Moment Strength I (Inv)	36	1.75	1.25	1.5	1.192	1.28	46.19	Span 1 Beam 1(0.500 L)
	Moment Strength I (Opt)	36	1.35	1.25	1.5	1.192	1.66	59.87	Span 1 Beam 1(0.500 L)
	Shear Strength I (Inv)	36	1.75	1.25	1.5	1.192	7.78	280.05	Span 1 Beam 1(0.964 L)
	Shear Strength I (Opt)	36	1.35	1.25	1.5	1.192	10.08	363.03	Span 1 Beam 1(0.964 L)
	Stress Service III (Inv)	36	1	1	1	1.192	0.65	23.49	Span 1 Beam 1(0.500 L)
	Stress Service I (Opt)	36	1	1	1	1.192	1.95	70.22	Span 1 Beam 1(0.500 L)
LASDV 1	Moment Strength II	54	1.54	1.25	1.5	0.75	2.33	125.90	Span 1 Beam 1(0.500 L)
	Shear Strength II	54	1.54	1.25	1.5	1.052	9.74	525.72	Span 1 Beam 1(0.700 L)
	Stress Service III	54	0.9	1	1	0.75	1.29	69.50	Span 1 Beam 1(0.500 L)
	Stress Service I	54	1	1	1	0.81	2.88	155.57	Span 1 Beam 3(0.500 L)
LASDV 2	Moment Strength II	44	1.6	1.25	1.5	0.75	2.97	130.61	Span 1 Beam 1(0.500 L)
	Shear Strength II	44	1.6	1.25	1.5	1.052	12.33	542.52	Span 1 Beam 1(0.036 L)
	Stress Service III	44	0.9	1	1	0.75	1.70	74.89	Span 1 Beam 1(0.500 L)
	Stress Service I	44	1	1	1	0.81	3.81	167.68	Span 1 Beam 3(0.500 L)
LASDV 3	Moment Strength II	66.35	1.34	1.25	1.5	0.75	2.26	150.21	Span 1 Beam 1(0.500 L)
	Shear Strength II	66.35	1.34	1.25	1.5	1.052	9.33	618.84	Span 1 Beam 1(0.700 L)
	Stress Service III	66.35	0.9	1	1	0.75	1.09	72.12	Span 1 Beam 1(0.500 L)
	Stress Service I	66.35	1	1	1	0.81	2.43	161.50	Span 1 Beam 3(0.500 L)
LASDV 4	Moment Strength II	71.25	1.26	1.25	1.5	0.75	2.18	155.12	Span 1 Beam 1(0.500 L)
	Shear Strength II	71.25	1.26	1.25	1.5	1.052	9.36	666.97	Span 1 Beam 1(0.700 L)
	Stress Service III	71.25	0.9	1	1	0.75	0.98	70.04	Span 1 Beam 1(0.500 L)
	Stress Service I	71.25	1	1	1	0.81	2.20	156.82	Span 1 Beam 3(0.500 L)
LASDV 5	Moment Strength II	104.5	1.2	1.25	1.5	0.75	1.82	189.77	Span 1 Beam 1(0.500 L)
	Shear Strength II	104.5	1.2	1.25	1.5	1.052	8.04	840.18	Span 1 Beam 1(0.036 L)
	Stress Service III	104.5	0.9	1	1	0.75	0.78	81.61	Span 1 Beam 1(0.500 L)
	Stress Service I	104.5	1	1	1	0.81	1.75	182.67	Span 1 Beam 3(0.500 L)

Table A.7 Load Rating Summary --Span 24 (based on sheet 525, 139' span length B2B)

Vehicle	Limit State	Weight in Tons	Load Factor			DF	RF	Tons	Location
			LL	DL	DW				
LASDV 6	Moment Strength II	90	1.2	1.25	1.5	0.75	2.56	230.71	Span 1 Beam 1(0.500 L)
	Shear Strength II	90	1.2	1.25	1.5	1.052	9.50	855.28	Span 1 Beam 1(0.964 L)
	Stress Service III	90	0.9	1	1	0.75	1.10	99.24	Span 1 Beam 1(0.500 L)
	Stress Service I	90	1	1	1	0.81	2.43	218.73	Span 1 Beam 3(0.400 L)
LASDV 7	Moment Strength II	130	1.2	1.25	1.5	0.75	2.32	301.39	Span 1 Beam 1(0.500 L)
	Shear Strength II	130	1.2	1.25	1.5	1.052	8.48	1102.75	Span 1 Beam 1(0.964 L)
	Stress Service III	130	0.9	1	1	0.75	1.00	129.65	Span 1 Beam 1(0.500 L)
	Stress Service I	130	1	1	1	0.81	2.22	288.76	Span 1 Beam 3(0.400 L)
LASDV 8	Moment Strength II	120	1.2	1.25	1.5	0.75	2.23	267.72	Span 1 Beam 1(0.500 L)
	Shear Strength II	120	1.2	1.25	1.5	1.052	8.72	1045.83	Span 1 Beam 1(0.036 L)
	Stress Service III	120	0.9	1	1	0.75	0.96	115.16	Span 1 Beam 1(0.500 L)
	Stress Service I	120	1	1	1	0.81	2.15	257.83	Span 1 Beam 3(0.500 L)

Note: Span 1 Beam 1 represents the New beam.

It can be seen that load rating factor for HL-93 (Inv.) and LASDV #4,5,8 under service III limit state are lower than 1.0, while the rating factors under Strength limit state are greater than 1.0.

A2 Dapped End Checking

The trapezoidal girders were designed with a typical dapped end as shown in Figure A2.1.

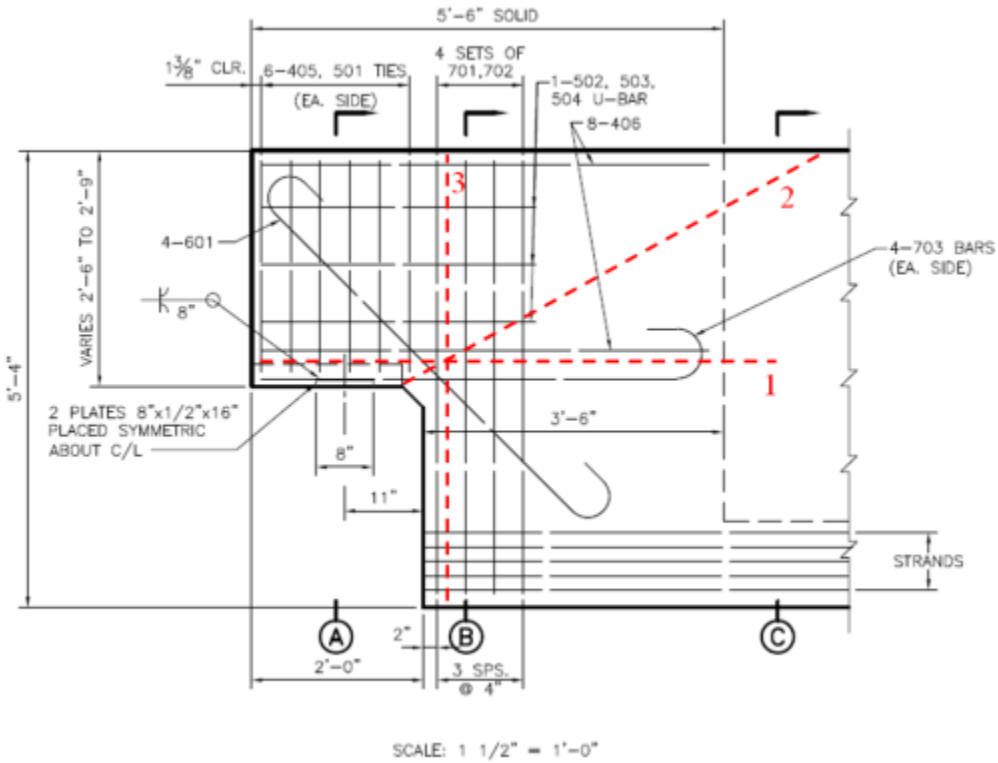


Figure A2.1 Typical Trapezoidal Girder Elevation

For this dapped end, three sections need to be checked. Section 1 and section 3 are subject to tension, while section 2 is the possible critical shear cracking plane. By referring to the plan, #4, 5, 6 and 7 deformed bars are used. Table A2.1 shows the nominal cross section area for the steel bars.

Table A2.1 Steel bar nominal cross section area, in^2

#4	0.20
#5	0.31
#6	0.44
#7	0.60

A2.1.1 Shear check for section 2 in beam dapped end

The shear capacity is determined based on the AASHTO LFRD Specification 2010. Figure A2.2 shows the details of girder dapped end with the shear failure plane highlighted by the dashed red line. The shear failure plane starts from the inside edge of the bearing with a failure angle, θ = **29°**, assumed based on the AASHTO LRFD Specifications Article 5.8.3.4.2.

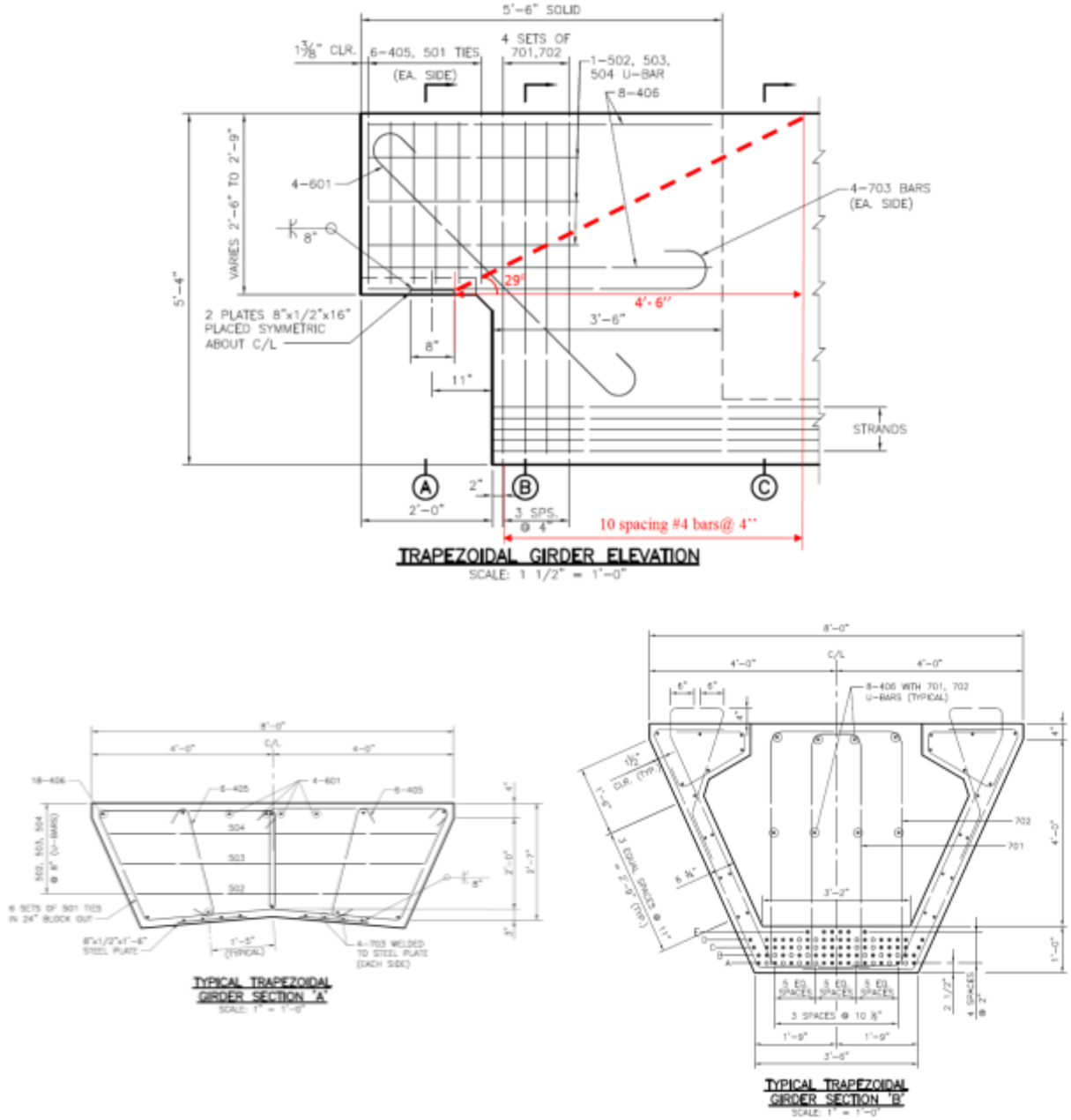


Figure A2.2 Dapped End Details and Critical Shear Failure Plane

Any shear reinforcement crossing the shear failure plane contributes to the shear capacity of the dapped end. As can be seen from Figure A2.2, three components are contributing to the shear capacity of the dapped end, as follows:

1. Vertical shear reinforcements
2. Inclined reinforcements
3. Concrete

Any of the above reinforcements crossing the failure plane should be counted in determining the shear capacity of the dapped ends.

Table A2.2 shows the details of each reinforcement component contributing to the shear capacity.

Table A2.2 Shear Contribution of Reinforcements/strands

	Steel detail		**Steel Area, in^2**
Vertical Bars	One set of 405,501 tie (see Section "A"):	1 set of (2-leg 405; 4-leg 501)	1*(2*0.20+4*0.31)=1.64
	4 sets of 701,702 (see section "B"):	4 sets of (2-leg 701; 2-leg 702)	4*(2*0.60+2*0.60)=9.6
	11 sets of 401,401 (see section "B") :	11 sets of (2-leg 401; 2-leg 402)	11*(2*0.20+2*0.20)=8.8
Inclined Bars	4 sets of 601	4 sets of 1-leg 601	4*0.44*cos(45°)=1.24
	Total		**21.28**
Hence, Shear Resistance due to steel reinforcements: Vs=Φ*As*Fy=0.9*21.28*60=*1149.00 kips*			

Table A2.3 Ultimate Shear Load Vu At Shear Critical Section, Factored(DL +LL), kips

Loading	Vu , @critical shear section (124' span)	Vu , @critical shear section (139' span)
HL-93	558.70	**554.14**
LASDV-1	538.50	497.94
LASDV-2	487.58	456.21
LASDV-3	550.38	508.03
LASDV-4	550.41	508.40
LASDV-5	***585.62***	545.54
LASDV-6	539.00	506.01
LASDV-7	555.57	532.14
LASDV-8	555.90	525.64

From Smart Bridge Suite, the ultimate Shear Load V_u at Shear Critical Section under factored dead load and live load can be retrieved and calculated, as shown in Table B.10. As can be seen, the control V_u=585.62 kips for 124' Span.

Hence, it can be seen that Vu < Vs indicating the beam end has sufficient shear resistance for safe use, even without taking the concrete shear contribution into account.

A2.1.2 Tension check for section 1&3 in beam dapped end

Section 1 and section 3, as shown in Figure B.1, are checked by an advanced approach, Strut-and-tie modeling (STM) in this report.

STM is a popular approach that has been used to design discontinuity regions (D-regions) in reinforced and/or prestressed concrete structures. The STM approach reduces complex states of stress within a D-region into a truss comprised of uniaxial stress paths. Each stress path is deemed as a STM member. The forces in each of the truss members can be determined using basic truss theory. Members subjected to tensile stresses are called ties, while compression members are called struts. By using STM approach, strength of a structural element can be estimated and detailed. Appropriate reinforcement should be placed at tie location.

For a dapped (ledged) end, the typical STM Models are shown in Figure A2.3.

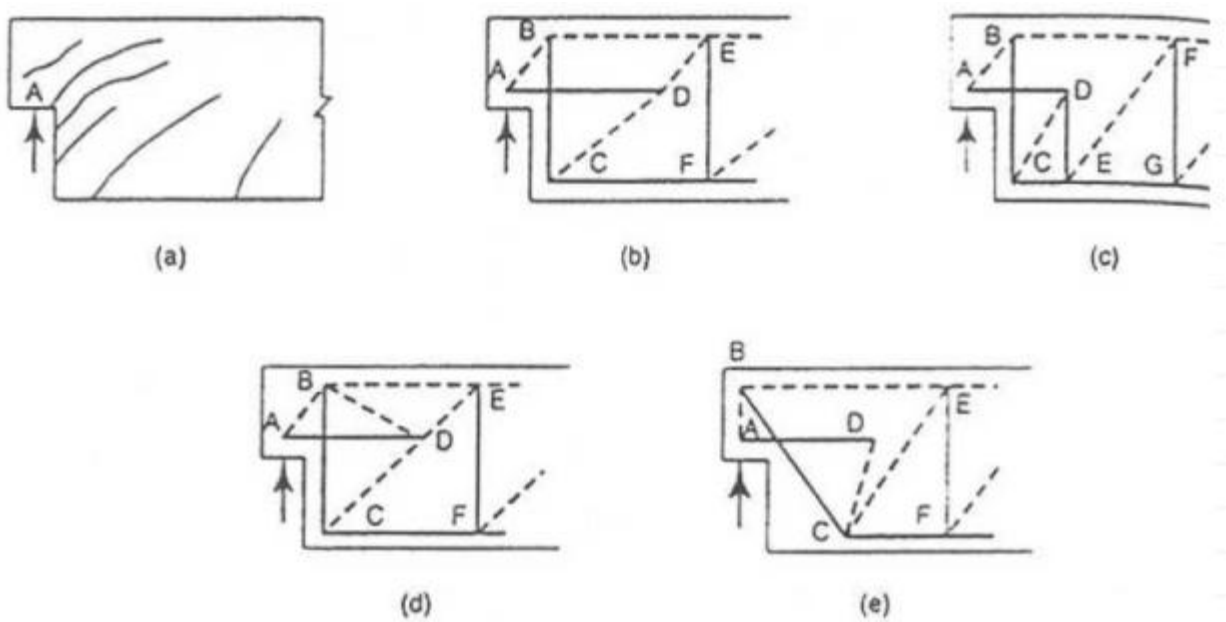
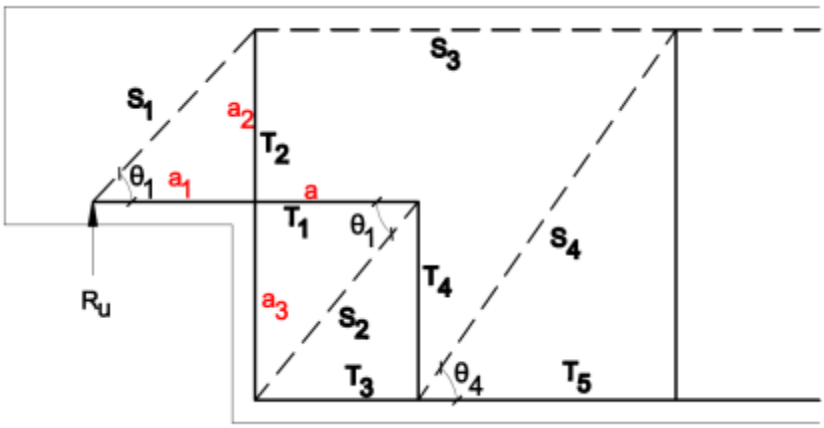


Figure A2.3 Typical STM Models for a dapped end

Among the above 4 models, (c) is most commonly used since it represent the critical controlling model , which is utilized in this report to check the reinforcement design, as shown in Figure A2.4.



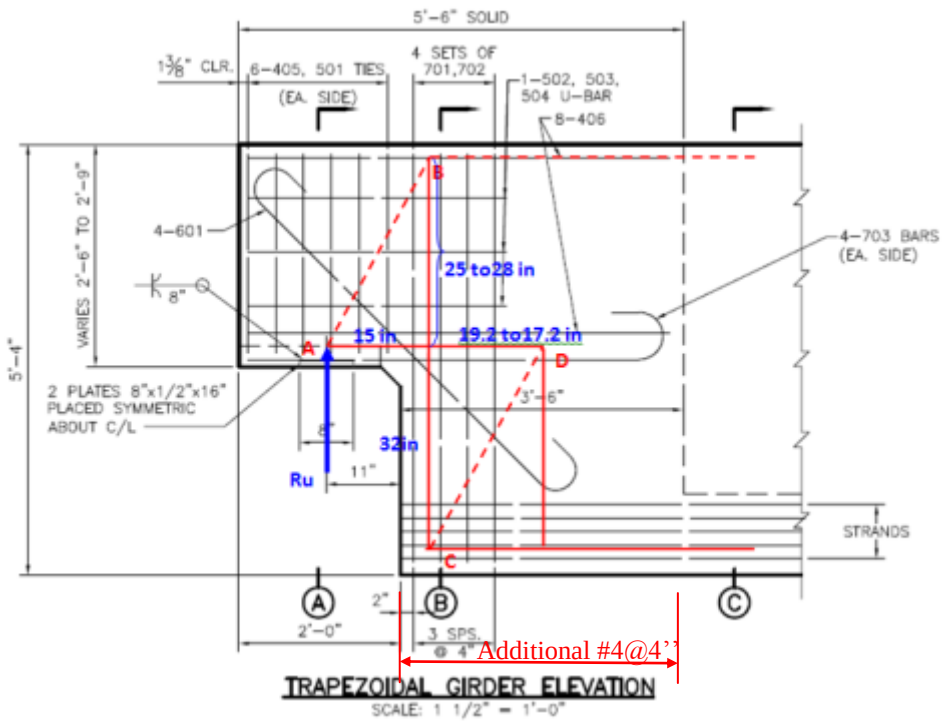


Figure A2.4 STM Model (2) for Beam dapped end

Where: R_u is the ultimate reaction under factored dead load and live load, as shown in Table A2.4.

Table A2.4 Reaction R_u under Factored(DL +LL), kips

Loading	R_u (124' span)	R_u (139' span)
HL-93	601.70	590.27
LASDV-1	577.37	528.63
LASDV-2	524.51	485.84
LASDV-3	589.63	539.37
LASDV-4	589.73	539.79
LASDV-5	***630.02***	580.50
LASDV-6	582.03	539.25
LASDV-7	605.06	570.70
LASDV-8	600.81	560.99

Note: The above values of dead load and live load are the shear force at section x=0 from SBS.

As can be seen the control value for the R_u=630.02 kips.

Based on the strut-tie truss in Figure 10, tension force in Member in T_1 and T_2 can be determined as below.

T_1= R_u*15/28=337.51 kips or T_1=Ru*15/25=378.01 kips (control)
T_2= R_u=630.02kips

Nominal Tension Capacity for T_1, T_2

13. **Vs @ T_1**

	Steel details		**Steel Area, in^2**
Horizontal Bars	4 set of 406	4 set of (1-leg 406;)	4*(0.20)=0.8
	8 sets of 703	8 sets of (1-leg 703)	8*(0.60)=4.8
Inclined Bars	4 sets of 601	4 sets of 1-leg 601	4*0.44*cos(45°)=1.244
	Total		**6.84**

Tensile capacity due to steel reinforcements:
Cs_1=Φ*A_s*f_y=0.9*6.844*60=369.5 kips < T_1 (=378.01 kips) N.G.

Recommend:

- *Use 4-No.7 bar instead of 4-601 for the inclined bars.*

	Steel details		**Steel Area, in^2**
Horizontal Bars	4 set of 406	4 set of (1-leg 406;)	4*(0.20)=0.8
	8 sets of 703	8 sets of (1-leg 703)	8*(0.60)=4.8
Inclined Bars	4 sets of 701	4 sets of 1-leg 701	4*0.6*cos(45°)=1.70
	Total		**7.3**

Hence, C's_1=Φ*A'$_s$*f_y=0.9*7.3*60=394.2 kips > T_1 (=378.01 kips) O.K.

14. **Vs @ T_2 (***assumed all 3 sets of 701, 702 engaged***)**

	Steel details		**Steel Area, in^2**
VerticalBars	3 sets of 701	3 set of (2-leg 701)	3*2*(0.60)=3.6
	3 sets of 702	3 set of (2-leg 702)	3*2*(0.60)=3.6
	3 sets of 401	3 set of (2-leg 401)	3*2*(0.20)=1.2
	3 sets of 402	3 sets of (2-leg 402)	3*2*(0.20)=1.2
Inclined Bars	4 sets of 601	4 sets of 1-leg 601	4*0.44*sin(45°)=1.244
	Total		**10.844**

Cs_2 =Φ*As*Fy=0.9*60*10.844=585.58 kips < T_2 (=630.02 kips) N.G.

No. 8 bar is recommended to substitute the 701, 702 bars.

APPENDIX B ANALYSIS OF PIER CAP LEDGE

Figure B.1 shows the typical details for the pier cap ledge. Similar to the dapped end of the trapezoidal girder, the pier ledge needs to be checked by STM approach.

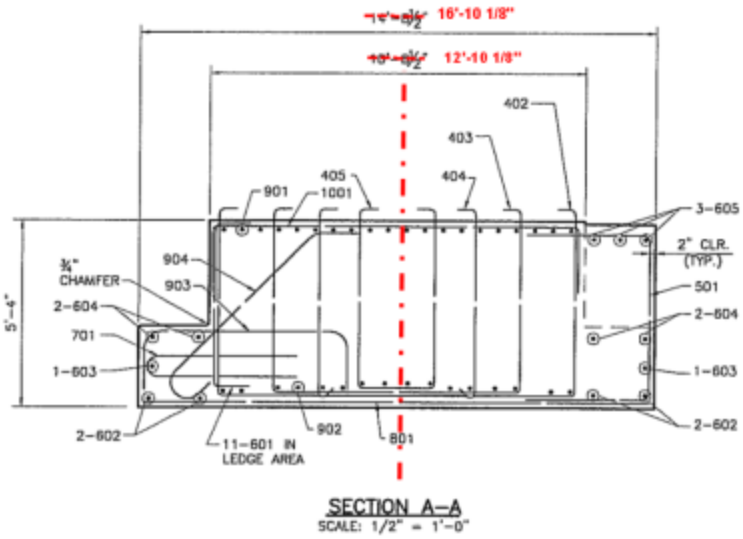
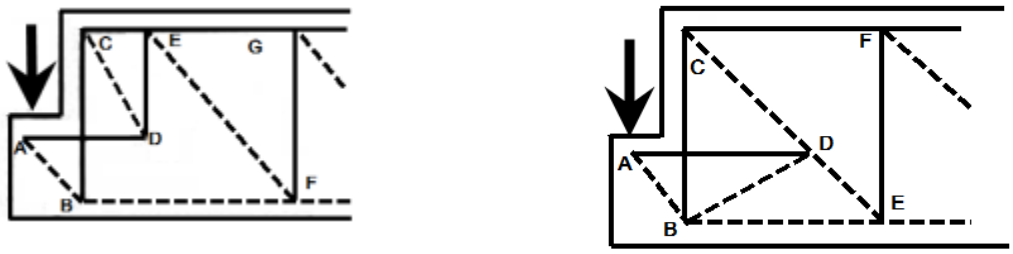


Figure B.1 Typical Details for the Pier Cap Ledge (Cap # 16)

Models (c) and (d) in Figure B.2 are utilized to check the strength and details of pier cap ledge in this report. Figures B.3 and Figure B.4 show the two detailed strut-and-tie models, in which dashed and solid line represents the strut and tie member, respectively.



(C) (D)

B.2 Typical STM Models for Cap Ledger

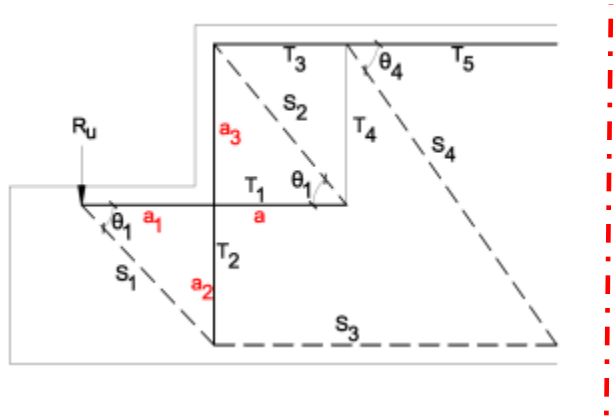


Figure B.3 Strut and Tie Model for Pier Ledge

Based on Figure B.2 and Figure B.3, the geometry of the strut-tie truss can be determined, as follows.

a_1=	17 in	a=	27.8 in
a_2=	22 in	a_3=	36 in

The corresponding forces in strut and tie members can be solved by using truss theory, as shown in Table B. 1.

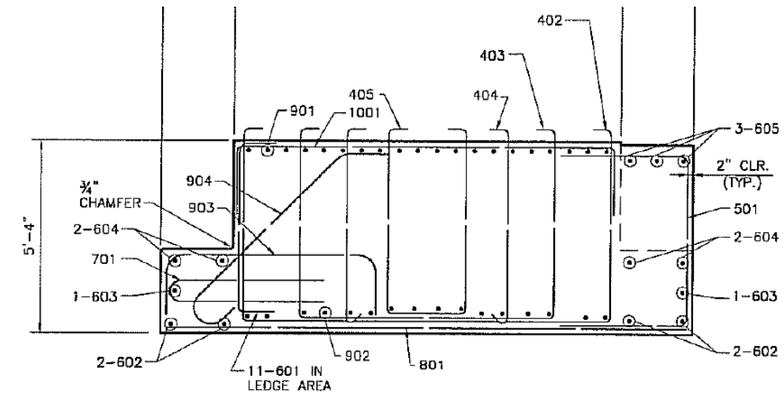
Table B.1 Forces in strut and Tie members

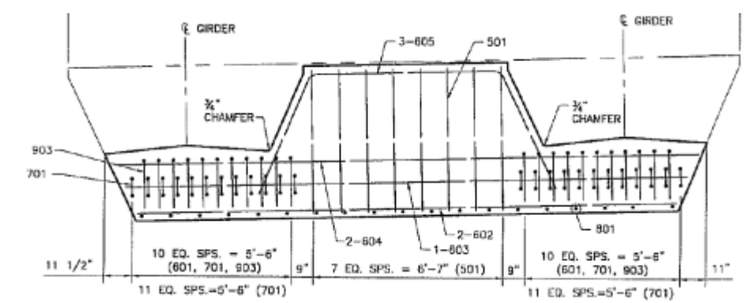
Member	Angle, θ	Force, kips
S1	52.3	796.2
S2	52.3	796.2
S3	0.0	486.9
S4	52.1	798.4
T1	0.0	486.9
T2	90.0	630.0
T3	0.0	486.9
T4	90.0	630.0
T5	0.0	977.4

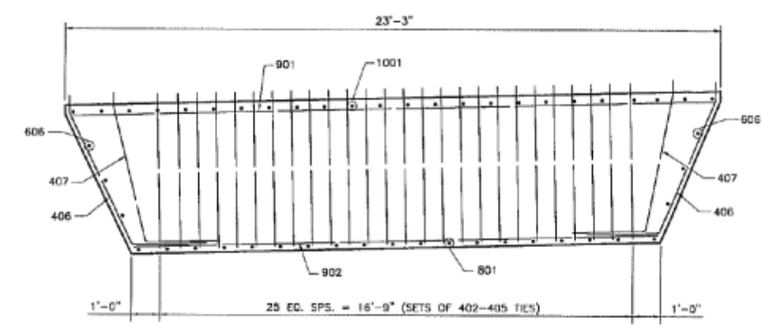
Table B.2 gives the determination of tension resistance for tie members based on 95% plan, which is compared with the required force. As can be seen that the reinforcement at locations T4, and T5 are not sufficient, and they need to be increased and distributed efficiently.

Table B.2 Resistance and Required Force for Tie Members

Member	Resistance	Required **(Calculated by Strut and Tie Method)**
T1	Bars 903 and the inclined bar 904. Hence, It has: 11-903 (1 leg)+ + 11-904 (Inclined), steel area = $11*1.0+11*1.0*\cos(45^o)=18.78\ in^2$ **Shear Resistance due to steel reinforcements, Vs:** **Vs=Φ*As*Fy** =0.9*18.78 *60=1013.96 kips	$\mathbf{F_{T1}}$=486.9kips < Vs (1013.96 kips), O.K.
T2	bars 402, 601 and the inclined bar 904. The number of 402 bar can be determined as below: *25 spacing 16'-9'' (402-405): spacing=16'-9''/25=0.67'=8''* *Width of one beam support: 5'-6'': 5'6'' /8''=8 spacings, which mean 9 sets (402-405).* *Hence,* It has: 11-601 + 11-904 (Inclined) + 9-402, steel area = $11*0.44+11*1.0*\cos(45^o)+9*0.2= 14.42in^2$ **Shear Resistance due to steel reinforcements, Vs:** **Vs=Φ*As*Fy** = 0.9*14.42 *60=778.52 kips	$\mathbf{F_{T2}}$=630.02 kips < Vs (778.52 kips), O.K.
T3	1001 bar, 12 bars engaged (24 in total) $12*1.27=15.24in^2$ **Shear Resistance due to steel reinforcements, Vs:** **Vs=Φ*As*Fy** = 0.9*15.24 *60=822.96 kips	$\mathbf{F_{T3}}$=486.9 kips < Vs (822.96 kips), O.K.
T4	*25 spacing 16'-9'' (402-405): spacing=16'-9''/25=0.67'=8''* *Width of one beam support: 5'-6'': 5'6'' /8''=8 spacings, which mean 9 sets (402-405).* *Hence,* It has: 9-404 and 9-403, steel area =2*9*0.2= $3.6in^2$ **Shear Resistance due to steel reinforcements, Vs:** **Vs=Φ*As*Fy** = 0.9*3.6*60=194.4 kips	$\mathbf{F_{T4}}$=630.02 kips > Vs (194.4 kips), **N.G.** Recommend: 2 set of 11-701 at 4''at T4 location.
T5	1001 bar, 12 bars engaged **(24 Bars in total)** $12*1.27=15.24in^2$ **Shear Resistance due to steel reinforcements, Vs:** **Vs=Φ*As*Fy** = 0.9*15.24 *60=822.96 kips	$\mathbf{F_{T5}}$=977.4 kips > Vs (822.96 kips), **N.G.** Recommend: 30 bars in total at T5 location. And close spacing near the bearing.







Another strut-and-tie model is shown in Figure B.4.

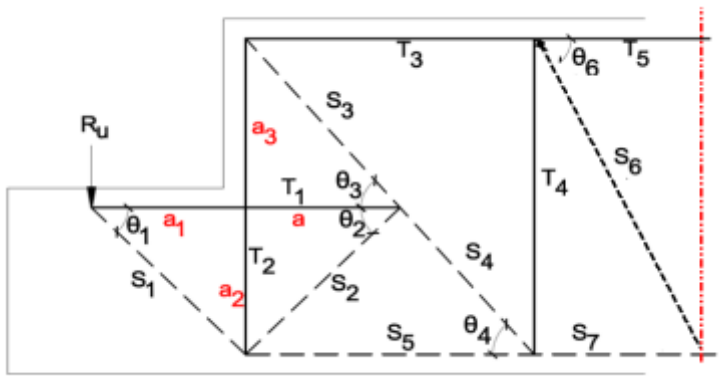


Figure B.4 Strut and Tie Model (2) for Pier Ledger

Based on Figure B.2 and Figure B.4, the geometry of the strut-tie truss is determined as follows.

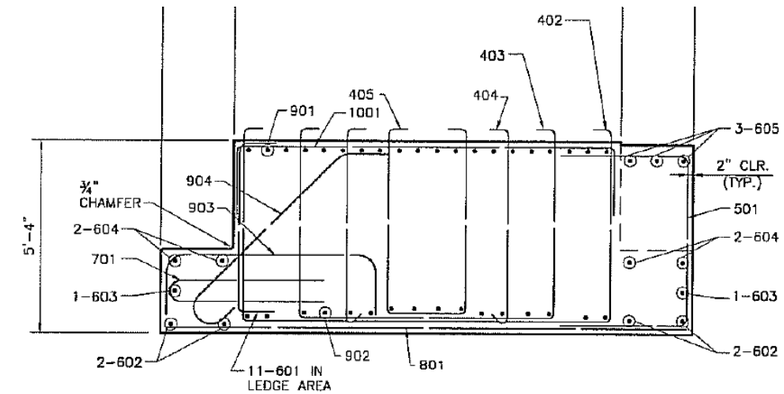
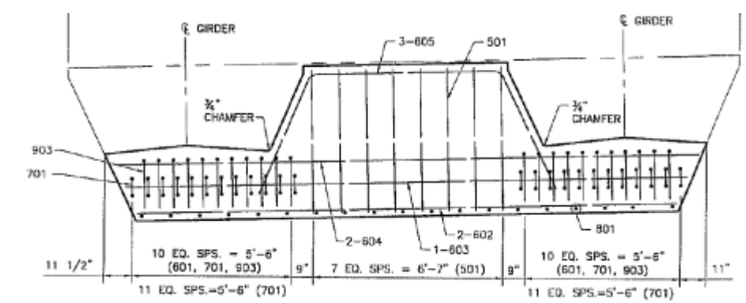
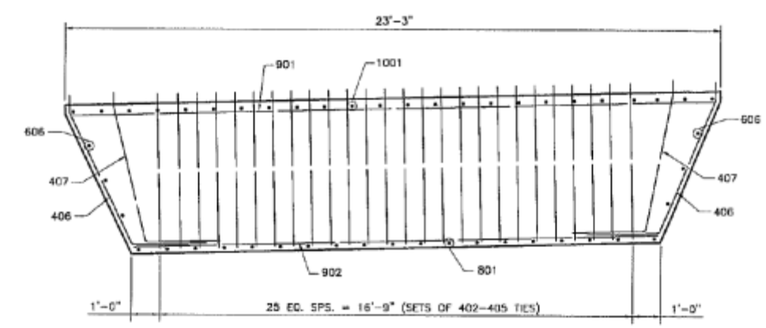
a_1=	17in	a=	25in
a_2=	22in	a_3=	36in

The corresponding forces in strut and tie members can be solved by using truss theory, as shown in Table B.3.

Table B.3 Forces in strut and Tie members

Member	Angle, θ	Force, kips
S1	52.3	796.2
S2	41.3	402.5
S3	55.2	1090.7
S4	55.2	766.9
S5	0.0	184.7
S6	60.5	502.5
S7	0.0	622.1
T1	0.0	486.9
T2	90.0	895.8
T3	0.0	622.1
T4	90.0	437.4
T5	0.0	869.4

Table B.4 gives the determination of tension resistance for tie members based on 95% plan, which is compared with the required force. As can be seen that the reinforcement at locations T2, T4, and T5 are not sufficient, and they need to be improved and detailed.

Table B.4 Resistance and required force for tie members

Member	Resistance	Required **(Calculated by Strut and Tie Method)**
T1	Bars 903 and the inclined bar 904. Hence, It has: 11-903 (1 leg)+ + 11-904 (Inclined), steel area = $11*1.0+11*1.0*\cos(45^o)=18.78\ in^2$ **Shear Resistance due to steel reinforcements, Vs:** **Vs=Φ*As*Fy** =0.9*18.78 *60=1013.96 kips	F_{T1}=486.9kips < Vs (1013.96 kips), O.K.
T2	bars 402, 601 and the inclined bar 904. The number of 402 bar can be determined as below: *25 spacing 16'-9'' (402-405): spacing=16'-9''/25=0.67'=8''* *Width of one beam support: 5'-6'': 5'6'' /8''=8 spacings, which mean 9 sets (402-405).* *Hence,* It has: 11-601 + 11-904 (Inclined) + 9-402, steel area = $11*0.44+11*1.0*\cos(45^o)+9*0.2= 14.42in^2$ **Shear Resistance due to steel reinforcements, Vs:** **Vs=Φ*As*Fy** = 0.9*14.42 *60=778.52 kips	F_{T2}=895.8 kips> Vs (778.52 kips), **N.G.** Recommend: Add one more set of 11-601 @ 4''next to the existing one.
T3	1001 bar, 12 bars engaged (24 in total) $12*1.27=15.24in^2$ **Shear Resistance due to steel reinforcements, Vs:** **Vs=Φ*As*Fy** = 0.9*15.24 *60=822.96 kips	F_{T3}=622.1kips < Vs (822.96 kips), O.K.
T4	*25 spacing 16'-9'' (402-405): spacing=16'-9''/25=0.67'=8''* *Width of one beam support: 5'-6'': 5'6'' /8''=8 spacings, which mean 9 sets (402-405).* *Hence,* It has: 9-404, steel area =9*0.2= $1.8in^2$ **Shear Resistance due to steel reinforcements, Vs:** **Vs=Φ*As*Fy** = 0.9*1.8*60=97.2 kips	F_{T4}=437.4 kips > Vs (97.2 kips), **N.G.** Recommend: 2 set of 11-701 @ 4''at T4 location.
T5	1001 bar, 12 bars engaged **(24 Bars in total)** $12*1.27=15.24in^2$ **Shear Resistance due to steel reinforcements, Vs:** **Vs=Φ*As*Fy** = 0.9*15.24 *60=822.96 kips	F_{T5}=869.4 kips > Vs (822.96 kips), **N.G.** Recommend: 30 bars in total at T5 location. And close spacing near the bearing.