Portsmouth Bypass
An Appalachian Development Highway

STATE OF OHIO DEPARTMENT OF TRANSPORTATION

Stage 2 Submission

SCI-TR234-0122
TR234 over CSXT Railroad
Bridge Calculations

Portsmouth Bypass, Phase 1

SCI-823-6.81
PID 19415

July 31, 2009

PREPARED FOR:
HDR Engineering, Inc.
9987 Carver Road, Suite 200
Cincinnati, Ohio 45242
513-984-7500

Ohio Department of Transportation
District 9
650 Eastern Avenue
Chillicothe, Ohio 45601

PREPARED BY:
KZF Design, Inc.
655 Eden Park Drive
Cincinnati, Ohio 45202
513-621-6211

HDR

ONE COMPANY | *Many Solutions®*

STAGE 2 DETAILED DESIGN

TABLE OF CONTENTS

A. GEOMETRY/LAYOUT

	AT	AU	AV	AW	AX	AY	AZ	BA	BB	BC
1						Project	SCI-TR234-0122 over CSXT Railroad			
2						Item	Superstructure			
3						Design	Deck Elevations			
4						Originator	DAT		Date	13-Jul-09
5						Checker			Date	
6		<-- PROFILE INPUT -->								
7	PVI	STA	ELEV	VCL	G1	G2	START	A	B	C
8		36+00.000	662.865		N.A.	-1.0000%	3600.000		-1.0000%	662.865
9	1	38+20.540	660.660		-1.0000%	-2.0017%	3820.540		-1.0000%	660.660
10							3820.540		-2.0017%	660.660
11	2	38+66.500	659.740		-2.0017%	-2.0000%	3866.500		-2.0017%	659.740
12							3866.500		-2.0000%	659.740
13	3	39+66.500	657.740	200	-2.0000%	0.4900%	3866.500	6.2250E-05	-2.0000%	659.740
14							4066.500		0.4900%	658.230
15	4	40+66.500	658.230		0.4900%		4066.500		0.4900%	658.230
16							4066.500			658.230
17										
18										
19										
20										
21										
22										
23										
24	UP	<-- WB SUPERELEVATION -->								
25	PVI	STA	SUPER	VCL	G1	G2	START	A	B	C
26					N.A.	0.0006%			0.0006%	
27	1	35+00.000	0.0200		0.0006%		3500.000		0.0006%	0.0200
28							3500.000			0.0200
29	2	41+00.000	0.0200				4100.000			0.0200
30							4100.000			0.0200
31										
32										
33										
34										
35										
36										
37										
38										
39										
40										
41										
42										
43	DOWN	<-- EB SUPERELEVATION -->								
44	PVI	STA	SUPER	VCL	G1	G2	START	A	B	C
45					N.A.	-0.0006%			-0.0006%	
46	1	35+00.000	-0.0200		-0.0006%		3500.000		-0.0006%	-0.0200
47							3500.000			-0.0200
48	2	41+00.000	-0.0200				4100.000			-0.0200
49							4100.000			-0.0200
50										
51										
52										
53										
54										

* P.G.L. = Centerline Constr. & SCI-TR234-0122 over CSXT Railroad
** D = EB (Down) , U = WB (Up)
*** (+) = RT of P.G.L.
Skew = 0

Project	SCI-TR234-0122 over CSXT Railroad
Item	Superstructure
Design	Deck Elevations
Originator	DAT Date 13-Jul-09
Checker	Date

Row	A	B	C	D	E	F	G	H	J	L	M	N	O	R	S	T	U	V	W	X	Y	AA	AC	AD	AE	AF	AI
7		*	**	OFFSETS ***					OFFSETS ***	OFFSETS ***					SUPER-	Deck Elevations						Deck Elevations	Deck Elevations				Rt edge
8	Location	STATION	Dir.	Lt edge	Beam 1	Beam 2	Beam 3	Beam 4	PGL	Beam 5	Beam 6	Beam 7	Beam 8	Rt edge	ELEV	P.G.L.	Lt edge	Beam 1	Beam 2	Beam 3	Beam 4	PGL	Beam 5	Beam 6	Beam 7	Beam 8	
9	Begin	36+69.500	U	-29.500											2.0000%	662.170	662.760										
10	Bridge	36+69.500														662.170						662.170					661.580
11	Limit	36+69.500	D											29.500	-2.0000%	662.170											
12		36+70.500	U	-29.500											2.0000%	662.160	661.570										
13		36+70.500	U		-27.125										2.0000%	662.160		661.618									
14		36+70.500	U			-19.375									2.0000%	662.160			661.773								
15		36+70.500	U				-11.625								2.0000%	662.160				661.928							
16		36+70.500	U					-3.875							2.0000%	662.160					662.083						
17	Centerline																										
18	Bearings	36+70.500														662.160						662.160					
19	Rear Abut.																										
20		36+70.500	D							3.875					-2.0000%	662.160							662.083				
21		36+70.500	D								11.625				-2.0000%	662.160								661.928			
22		36+70.500	D									19.375			-2.0000%	662.160									661.773		
23		36+70.500	D										27.125		-2.0000%	662.160										661.618	
24																											
25																											661.570
26		36+70.500	D											29.500	-2.0000%	662.160											
27		36+85.438	U	-29.500											2.0000%	662.011	661.421										
28		36+85.438	U		-27.125										2.0000%	662.011		661.469									
29		36+85.438	U			-19.375									2.0000%	662.011			661.624								
30		36+85.438	U				-11.625								2.0000%	662.011				661.779							
31		36+85.438	U					-3.875							2.0000%	662.011					661.934						
32	1/8 Pt.																										
33	Span 1	36+85.438														662.011						662.011					
34																											
35		36+85.438	D							3.875					-2.0000%	662.011							661.934				
36		36+85.438	D								11.625				-2.0000%	662.011								661.779			
37		36+85.438	D									19.375			-2.0000%	662.011									661.624		
38		36+85.438	D										27.125		-2.0000%	662.011										661.469	
39																											
40																											661.421
41		36+85.438	D											29.500	-2.0000%	662.011											
42		37+00.375	U	-29.500											2.0000%	661.862	661.272										
43		37+00.375	U		-27.125										2.0000%	661.862		661.319									
44		37+00.375	U			-19.375									2.0000%	661.862			661.474								
45		37+00.375	U				-11.625								2.0000%	661.862				661.629							
46		37+00.375	U					-3.875							2.0000%	661.862					661.784						
47	2/8 Pt.																										
48	Span 1	37+00.375														661.862						661.862					
49																											
50		37+00.375	D							3.875					-2.0000%	661.862							661.784				
51		37+00.375	D								11.625				-2.0000%	661.862								661.629			
52		37+00.375	D									19.375			-2.0000%	661.862									661.474		
53		37+00.375	D										27.125		-2.0000%	661.862										661.319	
54																											
55																											661.272
56		37+00.375	D											29.500	-2.0000%	661.862											
57		37+15.313	U	-29.500											2.0000%	661.712	661.122										
58		37+15.313	U		-27.125										2.0000%	661.712		661.170									
59		37+15.313	U			-19.375									2.0000%	661.712			661.325								
60		37+15.313	U				-11.625								2.0000%	661.712				661.480							
61		37+15.313	U					-3.875							2.0000%	661.712					661.635						
62	3/8 Pt.																										
63	Span 1	37+15.313														661.712						661.712					
64																											
65		37+15.313	D							3.875					-2.0000%	661.712							661.635				
66		37+15.313	D								11.625				-2.0000%	661.712								661.480			
67		37+15.313	D									19.375			-2.0000%	661.712									661.325		
68		37+15.313	D										27.125		-2.0000%	661.712										661.170	
69																											
70																											

535502-screed elevations (SCI-TR234-0122).xls

* P.G.L. = Centerline Constr. & SCI-TR234-0122 over CSXT Railroad
** D = EB (Down) , U = WB (Up)
*** (+) = RT of P.G.L.
Skew = 0

Project: SCI-TR234-0122 over CSXT Railroad
Item: Superstructure
Design: Deck Elevations
Originator: DAT Date: 13-Jul-09
Checker: Date:

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF	AG	AH	AI
7		*	**	OFFSETS ***						OFFSETS ***		OFFSETS ***							SUPER-	Deck Elevations							Deck Elevations		Deck Elevations						
8	Location	STATION	Dir.	Lt edge	Beam 1	Beam 2	Beam 3	Beam 4		PGL		Beam 5	Beam 6	Beam 7	Beam 8			Rt edge	ELEV	P.G.L.	Lt edge	Beam 1	Beam 2	Beam 3	Beam 4		PGL		Beam 5	Beam 6	Beam 7	Beam 8			Rt edge
71		37+15.313	D															29.500	-2.0000%	661.712															661.122
72		37+30.250	U	-29.500															2.0000%	661.563	660.973														
73		37+30.250	U		-27.125														2.0000%	661.563		661.020													
74		37+30.250	U			-19.375													2.0000%	661.563			661.175												
75		37+30.250	U				-11.625												2.0000%	661.563				661.330											
76		37+30.250	U					-3.875											2.0000%	661.563					661.485										
77	4/8 Pt.																																		
78	Span 1	37+30.250																		661.563							661.563								
79																																			
80		37+30.250	D									3.875							-2.0000%	661.563									661.485						
81		37+30.250	D										11.625						-2.0000%	661.563										661.330					
82		37+30.250	D											19.375					-2.0000%	661.563											661.175				
83		37+30.250	D												27.125				-2.0000%	661.563												661.020			
84																																			
85																																			660.973
86		37+30.250	D															29.500	-2.0000%	661.563															
87		37+45.188	U	-29.500															2.0000%	661.414	660.824														
88		37+45.188	U		-27.125														2.0000%	661.414		660.871													
89		37+45.188	U			-19.375													2.0000%	661.414			661.026												
90		37+45.188	U				-11.625												2.0000%	661.414				661.181											
91		37+45.188	U					-3.875											2.0000%	661.414					661.336										
92	5/8 Pt.																																		
93	Span 1	37+45.188																		661.414							661.414								
94																																			
95		37+45.188	D									3.875							-2.0000%	661.414									661.336						
96		37+45.188	D										11.625						-2.0000%	661.414										661.181					
97		37+45.188	D											19.375					-2.0000%	661.414											661.026				
98		37+45.188	D												27.125				-2.0000%	661.414												660.871			
99																																			
100																																			660.824
101		37+45.188	D															29.500	-2.0000%	661.414															
102		37+60.125	U	-29.500															2.0000%	661.264	660.674														
103		37+60.125	U		-27.125														2.0000%	661.264		660.722													
104		37+60.125	U			-19.375													2.0000%	661.264			660.877												
105		37+60.125	U				-11.625												2.0000%	661.264				661.032											
106		37+60.125	U					-3.875											2.0000%	661.264					661.187										
107	6/8 Pt.																																		
108	Span 1	37+60.125																		661.264							661.264								
109																																			
110		37+60.125	D									3.875							-2.0000%	661.264									661.187						
111		37+60.125	D										11.625						-2.0000%	661.264										661.032					
112		37+60.125	D											19.375					-2.0000%	661.264											660.877				
113		37+60.125	D												27.125				-2.0000%	661.264												660.722			
114																																			
115																																			660.674
116		37+60.125	D															29.500	-2.0000%	661.264															
117		37+75.063	U	-29.500															2.0000%	661.115	660.525														
118		37+75.063	U		-27.125														2.0000%	661.115		660.572													
119		37+75.063	U			-19.375													2.0000%	661.115			660.727												
120		37+75.063	U				-11.625												2.0000%	661.115				660.882											
121		37+75.063	U					-3.875											2.0000%	661.115					661.037										
122	7/8 Pt.																																		
123	Span 1	37+75.063																		661.115							661.115								
124																																			
125		37+75.063	D									3.875							-2.0000%	661.115									661.037						
126		37+75.063	D										11.625						-2.0000%	661.115										660.882					
127		37+75.063	D											19.375					-2.0000%	661.115											660.727				
128		37+75.063	D												27.125				-2.0000%	661.115												660.572			
129																																			
130																																			660.525
131		37+75.063	D															29.500	-2.0000%	661.115															
132		37+90.000	U	-29.500															2.0000%	660.965	660.375														

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF	AG	AH	AI
1	* P.G.L. = Centerline Constr. & SCI-TR234-0122 over CSXT Railroad																						Project	SCI-TR234-0122 over CSXT Railroad											
2	** D = EB (Down) , U = WB (Up)																						Item	Superstructure											
3	*** (+) = RT of P.G.L.																						Design	Deck Elevations											
4	Skew = 0																						Originator	DAT		Date	13-Jul-09								
5																							Checker			Date									
6																																			
7		*	**			OFFSETS ***				OFFSETS ***				OFFSETS ***					SUPER-				Deck Elevations				Deck Elevations				Deck Elevations				
8	Location	STATION	Dir.	Lt edge	Beam 1	Beam 2	Beam 3	Beam 4		PGL		Beam 5	Beam 6	Beam 7	Beam 8			Rt edge	ELEV	P.G.L.	Lt edge	Beam 1	Beam 2	Beam 3	Beam 4		PGL		Beam 5	Beam 6	Beam 7	Beam 8			Rt edge
133		37+90.000	U		-27.125														2.0000%	660.965		660.423													
134		37+90.000	U			-19.375													2.0000%	660.965			660.578												
135		37+90.000	U				-11.625												2.0000%	660.965				660.733											
136		37+90.000	U					-3.875											2.0000%	660.965					660.888										
137	Centerline																																		
138	Bearings	37+90.000																		660.965							660.965								
139	Fwd. Abut.																																		
140		37+90.000	D									3.875							-2.0000%	660.965									660.888						
141		37+90.000	D										11.625						-2.0000%	660.965										660.733					
142		37+90.000	D											19.375					-2.0000%	660.965											660.578				
143		37+90.000	D												27.125				-2.0000%	660.965												660.423			
144																																			
145																																			
146		37+90.000	D															29.500	-2.0000%	660.965															660.375
147	End	37+91.000	U	-29.500															2.0000%	660.955	660.365														
148	Bridge	37+91.000																		660.955							660.955								
149	Limit	37+91.000	D															29.500	-2.0000%	660.955															660.365

212 * P.G.L. = Centerline Constr. & SCI-TR234-0122 over CSXT Railroad
213 ** D = EB (Down) , U = WB (Up)
214 *** (+) = RT of P.G.L.
215 Skew = 0
216
217

Project: SCI-TR234-0122 over CSXT Railroad
Item: Superstructure
Design: Screed Elevations
Originator: DAT 13-Jul-09
Checker:

Row	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF	AG	AH	AI	AJ
218		*	**	OFFSETS ***						OFFSETS ***		OFFSETS ***							SUPER-	Screed Elevations							Screed Elevations		Screed Elevations							Dead Load Deflection
219	Location	STATION	Dir.	Lt edge	Beam 1	Beam 2	Beam 3	Beam 4		PGL		Beam 5	Beam 6	Beam 7	Beam 8			Rt edge	ELEV	P.G.L.	Lt edge	Beam 1	Beam 2	Beam 3	Beam 4		PGL		Beam 5	Beam 6	Beam 7	Beam 8			Rt edge	(in.)
220	Begin	36+69.500	U	-29.500															2.0000%	662.170	662.780															
221	Bridge	36+69.500																		662.170							662.170									
222	Limit	36+69.500	D															29.500	-2.0000%	662.170															661.580	
223		36+70.500	U	-29.500															2.0000%	662.160	661.570															
224		36+70.500	U		-27.125														2.0000%	662.160		661.618														
225		36+70.500	U			-19.375													2.0000%	662.160			661.773													
226		36+70.500	U				-11.625												2.0000%	662.160				661.926												
227		36+70.500	U					-3.875											2.0000%	662.160					662.083											
228	Centerline																																			
229	Bearings	36+70.500																		662.160							662.160									
230	Rear Abut.																																			
231		36+70.500	D									3.875							-2.0000%	662.160									662.083							
232		36+70.500	D										11.625						-2.0000%	662.160										661.926						
233		36+70.500	D											19.375					-2.0000%	662.160											661.773					
234		36+70.500	D												27.125				-2.0000%	662.160												661.518				
235																																				
236																																				
237		36+70.500	D															29.500	-2.0000%	662.160															661.570	
238		36+85.438	U	-29.500															2.0000%	662.011	661.467															
239		36+85.438	U		-27.125														2.0000%	662.011		661.515														
240		36+85.438	U			-19.375													2.0000%	662.011			661.670													
241		36+85.438	U				-11.625												2.0000%	662.011				661.825												
242		36+85.438	U					-3.875											2.0000%	662.011					661.980											
243	1/8 Pt.																									0.046										
244	Span 1	36+85.438																		662.011							662.057									0.552
245																												0.046								
246		36+85.438	D									3.875							-2.0000%	662.011									661.980							
247		36+85.438	D										11.625						-2.0000%	662.011										661.825						
248		36+85.438	D											19.375					-2.0000%	662.011											661.670					
249		36+85.438	D												27.125				-2.0000%	662.011												661.515				
250																																	0.046			
251																																		0.046		
252		36+85.438	D															29.500	-2.0000%	662.011															661.467	
253		37+00.375	U	-29.500															2.0000%	661.862	661.351															
254		37+00.375	U		-27.125														2.0000%	661.862		661.398														
255		37+00.375	U			-19.375													2.0000%	661.862			661.553													
256		37+00.375	U				-11.625												2.0000%	661.862				661.708												
257		37+00.375	U					-3.875											2.0000%	661.862					661.863											
258	2/8 Pt.																									0.079										
259	Span 1	37+00.375																		661.862							661.941									0.947
260																												0.079								
261		37+00.375	D									3.875							-2.0000%	661.862									661.863							
262		37+00.375	D										11.625						-2.0000%	661.862										661.708						
263		37+00.375	D											19.375					-2.0000%	661.862											661.553					
264		37+00.375	D												27.125				-2.0000%	661.862												661.398				
265																																	0.079			
266																																		0.079		
267		37+00.375	D															29.500	-2.0000%	661.862															661.351	
268		37+15.313	U	-29.500															2.0000%	661.712	661.221															
269		37+15.313	U		-27.125														2.0000%	661.712		661.268														
270		37+15.313	U			-19.375													2.0000%	661.712			661.423													
271		37+15.313	U				-11.625												2.0000%	661.712				661.578												
272		37+15.313	U					-3.875											2.0000%	661.712					661.733											
273	3/8 Pt.																									0.099										
274	Span 1	37+15.313																		661.712							661.811									1.183
275																												0.099								
276		37+15.313	D									3.875							-2.0000%	661.712									661.733							
277		37+15.313	D										11.625						-2.0000%	661.712										661.578						
278		37+15.313	D											19.375					-2.0000%	661.712											661.423					
279		37+15.313	D												27.125				-2.0000%	661.712												661.268				
280																																	0.099			
281																																		0.099		
282		37+15.313	D															29.500	-2.0000%	661.712															661.221	
283		37+30.250	U	-29.500															2.0000%	661.563	661.078															
284		37+30.250	U		-27.125														2.0000%	661.563		661.126														
285		37+30.250	U			-19.375													2.0000%	661.563			661.281													
286		37+30.250	U				-11.625												2.0000%	661.563				661.436												

212 * P.G.L. = Centerline Constr. & SCI-TR234-0122 over CSXT Railroad
213 ** D = EB (Down) , U = WB (Up)
214 *** (+) = RT of P.G.L.
215 Skew = 0

Project: SCI-TR234-0122 over CSXT Railroad
Item: Superstructure
Design: Screed Elevations
Originator: DAT 13-Jul-09
Checker:

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S
218		*	**	OFFSETS ***						OFFSETS ***				OFFSETS ***					SUPER-
219	Location	STATION	Dr.	Lt edge	Beam 1	Beam 2	Beam 3	Beam 4		PGL		Beam 5	Beam 6	Beam 7	Beam 8			Rt edge	ELEV
287		37+30.250	U					-3.875											2.0000%
288	4/8 Pt.																		
289	Span 1	37+30.250																	
290																			
291		37+30.250	D									3.875							-2.0000%
292		37+30.250	D										11.625						-2.0000%
293		37+30.250	D											19.375					-2.0000%
294		37+30.250	D												27.125				-2.0000%
295																			
296																			
297		37+30.250	D															29.500	-2.0000%
298		37+45.188	U	-29.500															2.0000%
299		37+45.188	U		-27.125														2.0000%
300		37+45.188	U			-19.375													2.0000%
301		37+45.188	U				-11.625												2.0000%
302		37+45.188	U					-3.875											2.0000%
303	5/8 Pt.																		
304	Span 1	37+45.188																	
305																			
306		37+45.188	D									3.875							-2.0000%
307		37+45.188	D										11.625						-2.0000%
308		37+45.188	D											19.375					-2.0000%
309		37+45.188	D												27.125				-2.0000%
310																			
311																			
312		37+45.188	D															29.500	-2.0000%
313		37+60.125	U	-29.500															2.0000%
314		37+60.125	U		-27.125														2.0000%
315		37+60.125	U			-19.375													2.0000%
316		37+60.125	U				-11.625												2.0000%
317		37+60.125	U					-3.875											2.0000%
318	6/8 Pt.																		
319	Span 1	37+60.125																	
320																			
321		37+60.125	D									3.875							-2.0000%
322		37+60.125	D										11.625						-2.0000%
323		37+60.125	D											19.375					-2.0000%
324		37+60.125	D												27.125				-2.0000%
325																			
326																			
327		37+60.125	D															29.500	-2.0000%
328		37+75.063	U	-29.500															2.0000%
329		37+75.063	U		-27.125														2.0000%
330		37+75.063	U			-19.375													2.0000%
331		37+75.063	U				-11.625												2.0000%
332		37+75.063	U					-3.875											2.0000%
333	7/8 Pt.																		
334	Span 1	37+75.063																	
335																			
336		37+75.063	D									3.875							-2.0000%
337		37+75.063	D										11.625						-2.0000%
338		37+75.063	D											19.375					-2.0000%
339		37+75.063	D												27.125				-2.0000%
340																			
341																			
342		37+75.063	D															29.500	-2.0000%
343		37+90.000	U	-29.500															2.0000%
344		37+90.000	U		-27.125														2.0000%
345		37+90.000	U			-19.375													2.0000%
346		37+90.000	U				-11.625												2.0000%
347		37+90.000	U					-3.875											2.0000%
348	Centerline																		
349	Bearings	37+90.000																	
350	Fwd. Abut.																		
351		37+90.000	D									3.875							-2.0000%
352		37+90.000	D										11.625						-2.0000%

	T	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF	AG	AH	AI	AJ
217																	Dead Load
218				Screed Elevations				Screed Elevations				Screed Elevations					Deflection
219	P.G.L.	Lt edge	Beam 1	Beam 2	Beam 3	Beam 4		PGL		Beam 5	Beam 6	Beam 7	Beam 8			Rt edge	(in.)
287	661.563					661.591											
288							0.105										
289	661.563							661.668									1.262
290									0.105								
291	661.563									661.591							
292	661.563										661.436						
293	661.563											661.251					
294	661.563												661.126				
295														0.105			
296															0.105		
297	661.563															661.078	
298	661.414	660.824															
299	661.414		660.871														
300	661.414			661.026													
301	661.414				661.181												
302	661.414					661.336											
303																	
304	661.414							661.414									1.183
305																	
306	661.414									661.336							
307	661.414										661.181						
308	661.414											661.026					
309	661.414												660.871				
310																	
311																	
312	661.414															660.824	
313	661.264	660.753															
314	661.264		660.801														
315	661.264			660.956													
316	661.264				661.111												
317	661.264					661.266											
318							0.079										
319	661.264							661.343									0.947
320									0.079								
321	661.264									661.266							
322	661.264										661.111						
323	661.264											660.956					
324	661.264												660.801				
325														0.079			
326															0.079		
327	661.264															660.753	
328	661.115	660.571															
329	661.115		660.618														
330	661.115			660.773													
331	661.115				660.928												
332	661.115					661.063											
333							0.046										
334	661.115							661.161									0.562
335									0.046								
336	661.115									661.063							
337	661.115										660.928						
338	661.115											660.773					
339	661.115												660.618				
340														0.046			
341															0.046		
342	661.115															660.571	
343	660.965	660.375															
344	660.965		660.423														
345	660.965			660.578													
346	660.965				660.733												
347	660.965					660.888											
348																	
349	660.965							660.965									
350																	
351	660.965									660.888							
352	660.965										660.733						

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S
212		* P.G.L. = Centerline Constr. & SCI-TR234-0122 over CSXT Railroad																	
213		** D = EB (Down) , U = WB (Up)																	
214		*** (+) = RT of P.G.L.																	
215	Skew = 0																		
216																			
217																			
218		*	**	OFFSETS ***						OFFSETS ***				OFFSETS ***					SUPER-
219	Location	STATION	Dir.	Lt edge	Beam 1	Beam 2	Beam 3	Beam 4		PGL		Beam 5	Beam 6	Beam 7	Beam 8			Rt edge	ELEV
353		37+90.000	D											19.375					-2.0000%
354		37+90.000	D												27.125				-2.0000%
355																			
356																			
357		37+90.000	D															29.500	-2.0000%
358	End	37+91.000	U	-29.500															2.0000%
359	Bridge	37+91.000																	
360	Limit	37+91.000	D															29.500	-2.0000%

	T	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF	AG	AH	AI	AJ
212				Project	SCI-TR234-0122 over CSXT Railroad												
213				Item	Superstructure												
214				Design	Screed Elevations												
215				Originator	DAT			13-Jul-09									
216				Checker													
217																	Dead Load
218				Screed Elevations				Screed Elevations				Screed Elevations					Deflection
219	P.G.L.	Lt edge	Beam 1	Beam 2	Beam 3	Beam 4		PGL		Beam 5	Beam 6	Beam 7	Beam 8			Rt edge	(in.)
353	660.965											660.578					
354	660.965												660.423				
355																	
356																	
357	660.965															660.375	
358	660.955	660.365															
359	660.965							660.955									
360	660.955															660.365	

	AM	AN	AO	AP	AQ	AR	AS	AT	AU	AV	AW	AX	AY	AZ	BA	BB	BC	BD	BE	BF	BG	BH	BI	BJ
210																				Project	SCI-TR234-0122 over CSXT Railroad			
211																				Item	Superstructure			
212																				Design	Abutment Beam Seat Elevations			
213																				Originator	DAT		Date	13-Jul-09
214																				Checker			Date	
215																								
216																								
217																								
218			Deck Elevations								THICKNESS (in.)						Abutment Beam Seat Elevations							
219	Location	STATION	Beam 1	Beam 2	Beam 3	Beam 4	Beam 5	Beam 6	Beam 7	Beam 8	deck	haunch	HP14x73	beam	brgs.	load plates	Beam 1	Beam 2	Beam 3	Beam 4	Beam 5	Beam 6	Beam 7	Beam 8
220		36+70.500	661.618								8.500	4.980	5.038	72.000	3.962	3.000	653.495							
221		36+70.500		661.773							8.500	4.980	6.898	72.000	3.962	3.000		653.495						
222	Centerline	36+70.500			661.928						8.500	4.980	8.758	72.000	3.962	3.000			653.495					
223	Bearings	36+70.500				662.083					8.500	4.980	10.618	72.000	3.962	3.000				653.495				
224	Rear Abut.	36+70.500					662.083				8.500	4.980	10.618	72.000	3.962	3.000					653.495			
225		36+70.500						661.928			8.500	4.980	8.758	72.000	3.962	3.000						653.495		
226		36+70.500							661.773		8.500	4.980	6.898	72.000	3.962	3.000							653.495	
227		36+70.500								661.618	8.500	4.980	5.038	72.000	3.962	3.000								653.495
228		37+90.000	660.423								8.500	4.980	5.038	72.000	3.962	3.000	652.300							
229		37+90.000		660.578							8.500	4.980	6.898	72.000	3.962	3.000		652.300						
230	Centerline	37+90.000			660.733						8.500	4.980	8.758	72.000	3.962	3.000			652.300					
231	Bearings	37+90.000				660.888					8.500	4.980	10.618	72.000	3.962	3.000				652.300				
232	Fwd. Abut.	37+90.000					660.888				8.500	4.980	10.618	72.000	3.962	3.000					652.300			
233		37+90.000						660.733			8.500	4.980	8.758	72.000	3.962	3.000						652.300		
234		37+90.000							660.578		8.500	4.980	6.898	72.000	3.962	3.000							652.300	
235		37+90.000								660.423	8.500	4.980	5.038	72.000	3.962	3.000								652.300

* P.G.L. = Centerline Constr. & SCI-TR234-0122 over CSXT Railroad
** D = EB (Down), U = WB (Up)
*** (+) = RT of P.G.L.
Skew = 0

Project	SCI-TR234-0122 over CSXT Railroad
Item	Superstructure
Design	Deck Elevations
Originator	DAT — Date 05-Feb-09
Checker	Date

Row	Location	* STATION	** Dir.	OFFSETS *** Lt edge	Beam 1	Beam 2	Beam 3	Beam 4	Beam 5	Beam 6	Beam 7	Beam 8	Rt edge	SUPER-ELEV	Deck Elevations P.G.L.	Lt edge	Beam 1	Beam 2	Beam 3	Beam 4	Deck Elevations PGL	Deck Elevations Beam 5	Beam 6	Beam 7	Beam 8	Rt edge
9	Begin	36+69.500	U	-29.500										2.0000%	662.170	662.760										
10	Bridge	36+69.500													662.170						662.170					661.580
11	Limit	36+69.500	D										29.500	-2.0000%	662.170											
12		36+70.500	U	-29.500										2.0000%	662.160	661.570										
13		36+70.500	U		-27.125									2.0000%	662.160		661.618									
14		36+70.500	U			-19.375								2.0000%	662.160			661.773								
15		36+70.500	U				-11.625							2.0000%	662.160				661.928							
16		36+70.500	U					-3.875						2.0000%	662.160					662.083						
17	Centerline														662.160						662.160					
18	Bearings	36+70.500																								
19	Rear Abut.																					662.083				
20		36+70.500	D						3.875					-2.0000%	662.160								661.928			
21		36+70.500	D							11.625				-2.0000%	662.160									661.773		
22		36+70.500	D								19.375			-2.0000%	662.160										661.618	
23		36+70.500	D									27.125		-2.0000%	662.160											
24																										
25																										661.570
26		36+70.500	D										29.500	-2.0000%	662.160											
27		36+85.438	U	-29.500										2.0000%	662.011	661.421										
28		36+85.438	U		-27.125									2.0000%	662.011		661.469									
29		36+85.438	U			-19.375								2.0000%	662.011			661.624								
30		36+85.438	U				-11.625							2.0000%	662.011				661.779							
31		36+85.438	U					-3.875						2.0000%	662.011					661.934						
32	1/8 Pt.														662.011						662.011					
33	Span 1	36+85.438																								
34																						661.934				
35		36+85.438	D						3.875					-2.0000%	662.011								661.779			
36		36+85.438	D							11.625				-2.0000%	662.011									661.624		
37		36+85.438	D								19.375			-2.0000%	662.011										661.469	
38		36+85.438	D									27.125		-2.0000%	662.011											
39																										
40																										661.421
41		36+85.438	D										29.500	-2.0000%	662.011											
42		37+00.375	U	-29.500										2.0000%	661.862	661.272										
43		37+00.375	U		-27.125									2.0000%	661.862		661.319									
44		37+00.375	U			-19.375								2.0000%	661.862			661.474								
45		37+00.375	U				-11.625							2.0000%	661.862				661.629							
46		37+00.375	U					-3.875						2.0000%	661.862					661.784						
47	2/8 Pt.														661.862						661.862					
48	Span 1	37+00.375																								
49																						661.784				
50		37+00.375	D						3.875					-2.0000%	661.862								661.629			
51		37+00.375	D							11.625				-2.0000%	661.862									661.474		
52		37+00.375	D								19.375			-2.0000%	661.862										661.319	
53		37+00.375	D									27.125		-2.0000%	661.862											
54																										
55																										661.272
56		37+00.375	D										29.500	-2.0000%	661.862											
57		37+15.313	U	-29.500										2.0000%	661.712	661.122										
58		37+15.313	U		-27.125									2.0000%	661.712		661.170									
59		37+15.313	U			-19.375								2.0000%	661.712			661.325								
60		37+15.313	U				-11.625							2.0000%	661.712				661.480							
61		37+15.313	U					-3.875						2.0000%	661.712					661.635						
62	3/8 Pt.														661.712						661.712					
63	Span 1	37+15.313																								
64																						661.635				
65		37+15.313	D						3.875					-2.0000%	661.712								661.480			
66		37+15.313	D							11.625				-2.0000%	661.712									661.325		
67		37+15.313	D								19.375			-2.0000%	661.712										661.170	
68		37+15.313	D									27.125		-2.0000%	661.712											
69																										
70																										

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R	S	T	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF	AG	AH	AI
1	* P.G.L. = Centerline Constr. & SCI-TR234-0122 over CSXT Railroad																						Project	SCI-TR234-0122 over CSXT Railroad											
2	** D = EB (Down) , U = WB (Up)																						Item	Superstructure											
3	*** (+) = RT of P.G.L.																						Design	Deck Elevations											
4	Skew = 0																						Originator	DAT		Date	05-Feb-09								
5																							Checker			Date									
6																																			
7		*	**	OFFSETS ***						OFFSETS ***		OFFSETS ***							SUPER-	Deck Elevations							Deck Elevations		Deck Elevations						
8	Location	STATION	Dir.	Lt edge	Beam 1	Beam 2	Beam 3	Beam 4		PGL		Beam 5	Beam 6	Beam 7	Beam 8			Rt edge	ELEV	P.G.L.	Lt edge	Beam 1	Beam 2	Beam 3	Beam 4		PGL		Beam 5	Beam 6	Beam 7	Beam 8			Rt edge
71		37+15.313	D															29.500	-2.0000%	661.712															661.122
72		37+30.250	U	-29.500															2.0000%	661.563	660.973														
73		37+30.250	U		-27.125														2.0000%	661.563		661.020													
74		37+30.250	U			-19.375													2.0000%	661.563			661.175												
75		37+30.250	U				-11.625												2.0000%	661.563				661.330											
76		37+30.250	U					-3.875											2.0000%	661.563					661.485										
77	4/8 Pt.																																		
78	Span 1	37+30.250																		661.563							661.563								
79																																			
80		37+30.250	D									3.875							-2.0000%	661.563									661.485						
81		37+30.250	D										11.625						-2.0000%	661.563										661.330					
82		37+30.250	D											19.375					-2.0000%	661.563											661.175				
83		37+30.250	D												27.125				-2.0000%	661.563												661.020			
84																																			
85																																			
86		37+30.250	D															29.500	-2.0000%	661.563															660.973
87		37+45.188	U	-29.500															2.0000%	661.414	660.824														
88		37+45.188	U		-27.125														2.0000%	661.414		660.871													
89		37+45.188	U			-19.375													2.0000%	661.414			661.026												
90		37+45.188	U				-11.625												2.0000%	661.414				661.181											
91		37+45.188	U					-3.875											2.0000%	661.414					661.336										
92	5/8 Pt.																																		
93	Span 1	37+45.188																		661.414							661.414								
94																																			
95		37+45.188	D									3.875							-2.0000%	661.414									661.336						
96		37+45.188	D										11.625						-2.0000%	661.414										661.181					
97		37+45.188	D											19.375					-2.0000%	661.414											661.026				
98		37+45.188	D												27.125				-2.0000%	661.414												660.871			
99																																			
100																																			
101		37+45.188	D															29.500	-2.0000%	661.414															660.824
102		37+60.125	U	-29.500															2.0000%	661.264	660.674														
103		37+60.125	U		-27.125														2.0000%	661.264		660.722													
104		37+60.125	U			-19.375													2.0000%	661.264			660.877												
105		37+60.125	U				-11.625												2.0000%	661.264				661.032											
106		37+60.125	U					-3.875											2.0000%	661.264					661.187										
107	6/8 Pt.																																		
108	Span 1	37+60.125																		661.264							661.264								
109																																			
110		37+60.125	D									3.875							-2.0000%	661.264									661.187						
111		37+60.125	D										11.625						-2.0000%	661.264										661.032					
112		37+60.125	D											19.375					-2.0000%	661.264											660.877				
113		37+60.125	D												27.125				-2.0000%	661.264												660.722			
114																																			
115																																			
116		37+60.125	D															29.500	-2.0000%	661.264															660.674
117		37+75.063	U	-29.500															2.0000%	661.115	660.525														
118		37+75.063	U		-27.125														2.0000%	661.115		660.572													
119		37+75.063	U			-19.375													2.0000%	661.115			660.727												
120		37+75.063	U				-11.625												2.0000%	661.115				660.882											
121		37+75.063	U					-3.875											2.0000%	661.115					661.037										
122	7/8 Pt.																																		
123	Span 1	37+75.063																		661.115							661.115								
124																																			
125		37+75.063	D									3.875							-2.0000%	661.115									661.037						
126		37+75.063	D										11.625						-2.0000%	661.115										660.882					
127		37+75.063	D											19.375					-2.0000%	661.115											660.727				
128		37+75.063	D												27.125				-2.0000%	661.115												660.572			
129																																			
130																																			
131		37+75.063	D															29.500	-2.0000%	661.115															660.525
132		37+90.000	U	-29.500															2.0000%	660.965	660.375														

* P.G.L. = Centerline Constr. & SCI-TR234-0122 over CSXT Railroad
** D = EB (Down) , U = WB (Up)
*** (+) = RT of P.G.L.
Skew = 0

Project: SCI-TR234-0122 over CSXT Railroad
Item: Superstructure
Design: Deck Elevations
Originator: DAT   Date: 05-Feb-09
Checker: Date:

	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q	R
7		*	**			OFFSETS ***				OFFSETS ***				OFFSETS ***				
8	Location	STATION	Dir.	Lt edge	Beam 1	Beam 2	Beam 3	Beam 4		PGL		Beam 5	Beam 6	Beam 7	Beam 8			Rt edge
133		37+90.000	U		-27.125													
134		37+90.000	U			-19.375												
135		37+90.000	U				-11.625											
136		37+90.000	U					-3.875										
137	Centerline																	
138	Bearings	37+90.000																
139	Fwd. Abut.																	
140		37+90.000	D									3.875						
141		37+90.000	D										11.625					
142		37+90.000	D											19.375				
143		37+90.000	D												27.125			
144																		
145																		
146		37+90.000	D															29.500
147	End	37+91.000	U	-29.500														
148	Bridge	37+91.000																
149	Limit	37+91.000	D															29.500

	S	T	U	V	W	X	Y	Z	AA	AB	AC	AD	AE	AF	AG	AH	AI
7	SUPER-			Deck Elevations					Deck Elevations				Deck Elevations				
8	ELEV	P.G.L.	Lt edge	Beam 1	Beam 2	Beam 3	Beam 4		PGL		Beam 5	Beam 6	Beam 7	Beam 8			Rt edge
133	2.0000%	660.965		660.423													
134	2.0000%	660.965			660.578												
135	2.0000%	660.965				660.733											
136	2.0000%	660.965					660.888										
137																	
138		660.965							660.965								
139																	
140	-2.0000%	660.965									660.888						
141	-2.0000%	660.965										660.733					
142	-2.0000%	660.965											660.578				
143	-2.0000%	660.965												660.423			
144																	
145																	
146	-2.0000%	660.965															660.375
147	2.0000%	660.955	660.365														
148		660.955							660.955								
149	-2.0000%	660.955															660.365

535502-screed elevations (SCI-TR234-0122).xls

	AM	AN	AO	AP	AQ	AR	AS	AT	AU	AV	AW	AX	AY	AZ	BA	BB	BC	BD	BE	BF	BG	BH	BI	BJ
210																				Project	SCI-TR234-0122 over CSXT Railroad			
211																				Item	Superstructure			
212																				Design	Abutment Beam Seat Elevations			
213																				Originator	DAT		Date	05-Feb-09
214																				Checker			Date	
215																								
216																								
217																								
218			Deck Elevations								THICKNESS (in.)						Abutment Beam Seat Elevations							
219	Location	STATION	Beam 1	Beam 2	Beam 3	Beam 4	Beam 5	Beam 6	Beam 7	Beam 8	deck	haunch	HP14x73	beam	brgs.	load plates	Beam 1	Beam 2	Beam 3	Beam 4	Beam 5	Beam 6	Beam 7	Beam 8
220		36+70.500	661.618								8.500	2.000	5.000	72.000	4.000	3.000	653.743							
221		36+70.500		661.773							8.500	2.000	6.860	72.000	4.000	3.000		653.743						
222	Centerline	36+70.500			661.928						8.500	2.000	8.720	72.000	4.000	3.000			653.743					
223	Bearings	36+70.500				662.083					8.500	2.000	10.580	72.000	4.000	3.000				653.743				
224	Rear Abut.	36+70.500					662.083				8.500	2.000	10.580	72.000	4.000	3.000					653.743			
225		36+70.500						661.928			8.500	2.000	8.720	72.000	4.000	3.000						653.743		
226		36+70.500							661.773		8.500	2.000	6.860	72.000	4.000	3.000							653.743	
227		36+70.500								661.618	8.500	2.000	5.000	72.000	4.000	3.000								653.743
228		37+90.000	660.423								8.500	2.000	5.000	72.000	4.000	3.000	652.548							
229		37+90.000		660.578							8.500	2.000	6.860	72.000	4.000	3.000		652.548						
230	Centerline	37+90.000			660.733						8.500	2.000	8.720	72.000	4.000	3.000			652.548					
231	Bearings	37+90.000				660.888					8.500	2.000	10.580	72.000	4.000	3.000				652.548				
232	Fwd. Abut.	37+90.000					660.888				8.500	2.000	10.580	72.000	4.000	3.000					652.548			
233		37+90.000						660.733			8.500	2.000	8.720	72.000	4.000	3.000						652.548		
234		37+90.000							660.578		8.500	2.000	6.860	72.000	4.000	3.000							652.548	
235		37+90.000								660.423	8.500	2.000	5.000	72.000	4.000	3.000								652.548

Project	SCI-TR234-0122 over CSXT Railroad		
Item	Superstructure		
Design	Deck Elevations		
Originator	DAT	Date	04-Feb-09
Checker		Date	

<-- PROFILE INPUT -->

PVI	STA	ELEV	VCL	G1	G2	START	A	B	C
	36+00.000	662.865		N.A.	-1.0000%	3600.000		-1.0000%	662.865
1	38+20.540	660.660		-1.0000%	-2.0017%	3820.540		-1.0000%	660.660
						3820.540		-2.0017%	660.660
2	38+66.500	659.740		-2.0017%	-2.0000%	3866.500		-2.0017%	659.740
						3866.500		-2.0000%	659.740
3	39+66.500	657.740	200	-2.0000%	0.4900%	3866.500	6.2250E-05	-2.0000%	659.740
						4066.500		0.4900%	658.230
4	40+66.500	658.230		0.4900%		4066.500		0.4900%	658.230
						4066.500			658.230

UP <-- WB SUPERELEVATION -->

PVI	STA	SUPER	VCL	G1	G2	START	A	B	C
				N.A.	0.0006%			0.0006%	
1	35+00.000	0.0200		0.0006%		3500.000		0.0006%	0.0200
						3500.000			0.0200
2	41+00.000	0.0200				4100.000			0.0200
						4100.000			0.0200

DOWN <-- EB SUPERELEVATION -->

PVI	STA	SUPER	VCL	G1	G2	START	A	B	C
				N.A.	-0.0006%			-0.0006%	
1	35+00.000	-0.0200		-0.0006%		3500.000		-0.0006%	-0.0200
						3500.000			-0.0200
2	41+00.000	-0.0200				4100.000			-0.0200
						4100.000			-0.0200

	AK	AL	AM	AN	AO	AP	AQ	AR
1								
2								
3								
4								
5								
6	Parameters for the general Vertical Curve (Axx+Bx+C) or Grade (Bx+C) equations							
7	Profile	Coefficients			Super	Coefficients		
8	ProX	ProA	ProB	ProC	SupX	SupA	SupB	SupC
9	69.50		-0.01	662.8654	169.50			0.02
10	69.50		-0.01	662.8654				
11	69.50		-0.01	662.8654	169.50			-0.02
12	70.50		-0.01	662.8654	170.50			0.02
13	70.50		-0.01	662.8654	170.50			0.02
14	70.50		-0.01	662.8654	170.50			0.02
15	70.50		-0.01	662.8654	170.50			0.02
16	70.50		-0.01	662.8654	170.50			0.02
17								
18	70.50		-0.01	662.8654				
19								
20	70.50		-0.01	662.8654	170.50			-0.02
21	70.50		-0.01	662.8654	170.50			-0.02
22	70.50		-0.01	662.8654	170.50			-0.02
23	70.50		-0.01	662.8654	170.50			-0.02
24								
25								
26	70.50		-0.01	662.8654	170.50			-0.02
27	85.44		-0.01	662.8654	185.44			0.02
28	85.44		-0.01	662.8654	185.44			0.02
29	85.44		-0.01	662.8654	185.44			0.02
30	85.44		-0.01	662.8654	185.44			0.02
31	85.44		-0.01	662.8654	185.44			0.02
32								
33	85.44		-0.01	662.8654				
34								
35	85.44		-0.01	662.8654	185.44			-0.02
36	85.44		-0.01	662.8654	185.44			-0.02
37	85.44		-0.01	662.8654	185.44			-0.02
38	85.44		-0.01	662.8654	185.44			-0.02
39								
40								
41	85.44		-0.01	662.8654	185.44			-0.02
42	100.38		-0.01	662.8654	200.38			0.02
43	100.38		-0.01	662.8654	200.38			0.02
44	100.38		-0.01	662.8654	200.38			0.02
45	100.38		-0.01	662.8654	200.38			0.02
46	100.38		-0.01	662.8654	200.38			0.02
47								
48	100.38		-0.01	662.8654				
49								
50	100.38		-0.01	662.8654	200.38			-0.02
51	100.38		-0.01	662.8654	200.38			-0.02
52	100.38		-0.01	662.8654	200.38			-0.02
53	100.38		-0.01	662.8654	200.38			-0.02
54								
55								
56	100.38		-0.01	662.8654	200.38			-0.02
57	115.31		-0.01	662.8654	215.31			0.02
58	115.31		-0.01	662.8654	215.31			0.02
59	115.31		-0.01	662.8654	215.31			0.02
60	115.31		-0.01	662.8654	215.31			0.02
61	115.31		-0.01	662.8654	215.31			0.02
62								
63	115.31		-0.01	662.8654				
64								
65	115.31		-0.01	662.8654	215.31			-0.02
66	115.31		-0.01	662.8654	215.31			-0.02
67	115.31		-0.01	662.8654	215.31			-0.02
68	115.31		-0.01	662.8654	215.31			-0.02
69								
70								
71	115.31		-0.01	662.8654	215.31			-0.02
72	130.25		-0.01	662.8654	230.25			0.02
73	130.25		-0.01	662.8654	230.25			0.02
74	130.25		-0.01	662.8654	230.25			0.02
75	130.25		-0.01	662.8654	230.25			0.02
76	130.25		-0.01	662.8654	230.25			0.02
77								
78	130.25		-0.01	662.8654				
79								
80	130.25		-0.01	662.8654	230.25			-0.02

	AK	AL	AM	AN	AO	AP	AQ	AR
1								
2								
3								
4								
5								
6	Parameters for the general Vertical Curve (Axx+Bx+C) or Grade (Bx+C) equations							
7	Profile	Coefficients			Super	Coefficients		
8	ProX	ProA	ProB	ProC	SupX	SupA	SupB	SupC
81	130.25		-0.01	662.8654	230.25			-0.02
82	130.25		-0.01	662.8654	230.25			-0.02
83	130.25		-0.01	662.8654	230.25			-0.02
84								
85								
86	130.25		-0.01	662.8654	230.25			-0.02
87	145.19		-0.01	662.8654	245.19			0.02
88	145.19		-0.01	662.8654	245.19			0.02
89	145.19		-0.01	662.8654	245.19			0.02
90	145.19		-0.01	662.8654	245.19			0.02
91	145.19		-0.01	662.8654	245.19			0.02
92								
93	145.19		-0.01	662.8654				
94								
95	145.19		-0.01	662.8654	245.19			-0.02
96	145.19		-0.01	662.8654	245.19			-0.02
97	145.19		-0.01	662.8654	245.19			-0.02
98	145.19		-0.01	662.8654	245.19			-0.02
99								
100								
101	145.19		-0.01	662.8654	245.19			-0.02
102	160.13		-0.01	662.8654	260.13			0.02
103	160.13		-0.01	662.8654	260.13			0.02
104	160.13		-0.01	662.8654	260.13			0.02
105	160.13		-0.01	662.8654	260.13			0.02
106	160.13		-0.01	662.8654	260.13			0.02
107								
108	160.13		-0.01	662.8654				
109								
110	160.13		-0.01	662.8654	260.13			-0.02
111	160.13		-0.01	662.8654	260.13			-0.02
112	160.13		-0.01	662.8654	260.13			-0.02
113	160.13		-0.01	662.8654	260.13			-0.02
114								
115								
116	160.13		-0.01	662.8654	260.13			-0.02
117	175.06		-0.01	662.8654	275.06			0.02
118	175.06		-0.01	662.8654	275.06			0.02
119	175.06		-0.01	662.8654	275.06			0.02
120	175.06		-0.01	662.8654	275.06			0.02
121	175.06		-0.01	662.8654	275.06			0.02
122								
123	175.06		-0.01	662.8654				
124								
125	175.06		-0.01	662.8654	275.06			-0.02
126	175.06		-0.01	662.8654	275.06			-0.02
127	175.06		-0.01	662.8654	275.06			-0.02
128	175.06		-0.01	662.8654	275.06			-0.02
129								
130								
131	175.06		-0.01	662.8654	275.06			-0.02
132	190.00		-0.01	662.8654	290.00			0.02
133	190.00		-0.01	662.8654	290.00			0.02
134	190.00		-0.01	662.8654	290.00			0.02
135	190.00		-0.01	662.8654	290.00			0.02
136	190.00		-0.01	662.8654	290.00			0.02
137								
138	190.00		-0.01	662.8654				
139								
140	190.00		-0.01	662.8654	290.00			-0.02
141	190.00		-0.01	662.8654	290.00			-0.02
142	190.00		-0.01	662.8654	290.00			-0.02
143	190.00		-0.01	662.8654	290.00			-0.02
144								
145								
146	190.00		-0.01	662.8654	290.00			-0.02
147	191.00		-0.01	662.8654	291.00			0.02
148	191.00		-0.01	662.8654				
149	191.00		-0.01	662.8654	291.00			-0.02

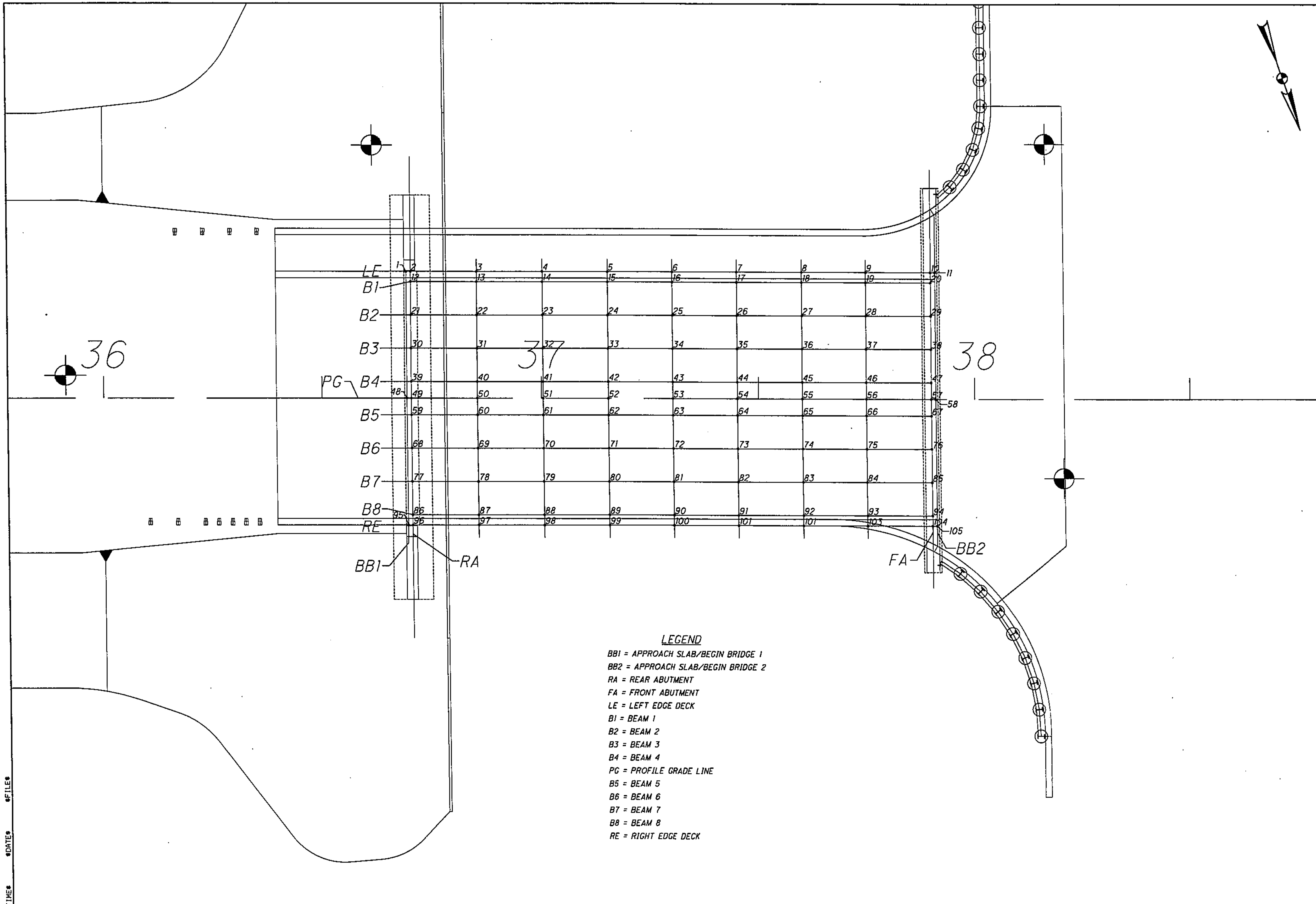
STRUCTURE NO. TR234-0122		SCREED TABLE										
KZF DESIGN DEC. 19, 2008		Approach Slab	CL Brg. Rear Abut.	1/8	2/8	3/8	4/8	5/8	6/8	7/8	CL Brg. Front Abut.	Approach Slab
LEFT EDGE DECK	Point #	1	2	3	4	5	6	7	8	9	10	11
	Station	36+69.499	36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	37+90.999
	Sta. Offset (CL)	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000
	Deck Elevation	661.58	661.57	661.42	661.27	661.12	660.97	660.82	660.67	660.52	660.37	660.36
	Deflection											
	Screed Elevation											
BEAM 1	Point #		12	13	14	15	16	17	18	19	20	
	Station		36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	
	Sta. Offset (CL)		27.1250	27.1250	27.1250	27.1250	27.1250	27.1250	27.1250	27.1250	27.1250	
	Deck Elevation		661.62	661.47	661.32	661.17	661.02	660.87	660.72	660.57	660.42	
	Deflection											
	Screed Elevation											
BEAM 2	Point #		21	22	23	24	25	26	27	28	29	
	Station		36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	
	Sta. Offset (CL)		19.3750	19.3750	19.3750	19.3750	19.3750	19.3750	19.3750	19.3750	19.3750	
	Deck Elevation		661.77	661.62	661.47	661.32	661.17	661.02	660.87	660.72	660.57	
	Deflection											
	Screed Elevation											
BEAM 3	Point #		30	31	32	33	34	35	36	37	38	
	Station		36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	
	Sta. Offset (CL)		11.6250	11.6250	11.6250	11.6250	11.6250	11.6250	11.6250	11.6250	11.6250	
	Deck Elevation		661.93	661.78	661.63	661.48	661.33	661.18	661.03	660.88	660.73	
	Deflection											
	Screed Elevation											
BEAM 4	Point #		39	40	41	42	43	44	45	46	47	
	Station		36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	
	Sta. Offset (CL)		3.8750	3.8750	3.8750	3.8750	3.8750	3.8750	3.8750	3.8750	3.8750	
	Deck Elevation		662.08	661.93	661.78	661.63	661.48	661.33	661.18	661.03	660.88	
	Deflection											
	Screed Elevation											
PROFILE GRADE LINE	Point #	48	49	50	51	52	53	54	55	56	57	58
	Station	36+69.499	36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	37+90.999
	Sta. Offset (CL)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	Deck Elevation	662.17	662.16	662.01	661.86	661.71	661.56	661.41	661.26	661.11	660.96	660.95
	Deflection											
	Screed Elevation											
BEAM 5	Point #		59	60	61	62	63	64	65	66	67	
	Station		36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	
	Sta. Offset (CL)		3.8750	3.8750	3.8750	3.8750	3.8750	3.8750	3.8750	3.8750	3.8750	
	Deck Elevation		662.08	661.93	661.78	661.63	661.48	661.33	661.18	661.03	660.88	
	Deflection											
	Screed Elevation											
BEAM 6	Point #		68	69	70	71	72	73	74	75	76	
	Station		36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	
	Sta. Offset (CL)		11.6250	11.6250	11.6250	11.6250	11.6250	11.6250	11.6250	11.6250	11.6250	
	Deck Elevation		661.93	661.78	661.63	661.48	661.33	661.18	661.03	660.88	660.73	
	Deflection											
	Screed Elevation											
BEAM 7	Point #		77	78	79	80	81	82	83	84	85	
	Station		36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	
	Sta. Offset (CL)		19.3750	19.3750	19.3750	19.3750	19.3750	19.3750	19.3750	19.3750	19.3750	
	Deck Elevation		661.77	661.62	661.47	661.32	661.17	661.02	660.87	660.72	660.57	
	Deflection											
	Screed Elevation											
BEAM 8	Point #		86	87	88	89	90	91	92	93	94	
	Station		36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	
	Sta. Offset (CL)		27.1250	27.1250	27.1250	27.1250	27.1250	27.1250	27.1250	27.1250	27.1250	
	Deck Elevation		661.62	661.47	661.32	661.17	661.02	660.87	660.72	660.57	660.42	
	Deflection											
	Screed Elevation											
RIGHT EDGE DECK	Point #	95	96	97	98	99	100	101	102	103	104	105
	Station	36+69.499	36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	37+90.999
	Sta. Offset (CL)	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000
	Deck Elevation	661.58	661.57	661.42	661.27	661.12	660.97	660.82	660.67	660.52	660.37	660.36
	Deflection											
	Screed Elevation											

STRUCTURE NO. TR234-0122

LEFT BRIDGE				
POINT	LOCATION		STATION (ALONG CL OF CONST.)	OFFSET FROM PROFILE GRADE LINE
	SPAN (VERTICAL)	(HORIZONTAL)		
1	BB1	LE1	36+69.499	29.5000
2	RA	LE1	36+70.499	29.5000
3	RA	LE1	36+85.436	29.5000
4	RA	LE1	37+00.374	29.5000
5	RA	LE1	37+15.311	29.5000
6	RA	LE1	37+30.249	29.5000
7	RA	LE1	37+45.186	29.5000
8	RA	LE1	37+60.124	29.5000
9	RA	LE1	37+75.061	29.5000
10	FA	LE1	37+89.999	29.5000
11	BB2	LE1	37+90.899	29.5000
12	RA	B1	36+70.499	27.1250
13	RA	B1	36+85.436	27.1250
14	RA	B1	37+00.374	27.1250
15	RA	B1	37+15.311	27.1250
16	RA	B1	37+30.249	27.1250
17	RA	B1	37+45.186	27.1250
18	RA	B1	37+60.124	27.1250
19	RA	B1	37+75.061	27.1250
20	FA	B1	37+89.999	27.1250
21	RA	B2	36+70.499	19.3750
22	RA	B2	36+85.436	19.3750
23	RA	B2	37+00.374	19.3750
24	RA	B2	37+15.311	19.3750
25	RA	B2	37+30.249	19.3750
26	RA	B2	37+45.186	19.3750
27	RA	B2	37+60.124	19.3750
28	RA	B2	37+75.061	19.3750
29	FA	B2	37+89.999	19.3750
30	RA	B3	36+70.499	11.6250
31	RA	B3	36+85.436	11.6250
32	RA	B3	37+00.374	11.6250
33	RA	B3	37+15.311	11.6250
34	RA	B3	37+30.249	11.6250
35	RA	B3	37+45.186	11.6250
36	RA	B3	37+60.124	11.6250
37	RA	B3	37+75.061	11.6250
38	FA	B3	37+89.999	11.6250
39	RA	B4	36+70.499	3.8750
40	RA	B4	36+85.436	3.8750
41	RA	B4	37+00.374	3.8750
42	RA	B4	37+15.311	3.8750
43	RA	B4	37+30.249	3.8750
44	RA	B4	37+45.186	3.8750
45	RA	B4	37+60.124	3.8750
46	RA	B4	37+75.061	3.8750

47	FA	B4	37+89.999	3.8750
48	BB1	PG	36+69.499	0.0000
49	RA	PG	36+70.499	0.0000
50	RA	PG	36+85.436	0.0000
51	RA	PG	37+00.374	0.0000
52	RA	PG	37+15.311	0.0000
53	RA	PG	37+30.249	0.0000
54	RA	PG	37+45.186	0.0000
55	RA	PG	37+60.124	0.0000
56	RA	PG	37+75.061	0.0000
57	FA	PG	37+89.999	0.0000
58	BB2	PG	37+90.999	0.0000
59	RA	B5	36+70.499	3.8750
60	RA	B5	36+85.436	3.8750
61	RA	B5	37+00.374	3.8750
62	RA	B5	37+15.311	3.8750
63	RA	B5	37+30.249	3.8750
64	RA	B5	37+45.186	3.8750
65	RA	B5	37+60.124	3.8750
66	RA	B5	37+75.061	3.8750
67	FA	B5	37+89.999	3.8750
68	RA	B6	36+70.499	11.6250
69	RA	B6	36+85.436	11.6250
70	RA	B6	37+00.374	11.6250
71	RA	B6	37+15.311	11.6250
72	RA	B6	37+30.249	11.6250
73	RA	B6	37+45.186	11.6250
74	RA	B6	37+60.124	11.6250
75	RA	B6	37+75.061	11.6250
76	RFA	B6	37+89.999	11.6250
77	RA	B7	36+70.499	19.3750
78	RA	B7	36+85.436	19.3750
79	RA	B7	37+00.374	19.3750
80	RA	B7	37+15.311	19.3750
81	RA	B7	37+30.249	19.3750
82	RA	B7	37+45.186	19.3750
83	RA	B7	37+60.124	19.3750
84	RA	B7	37+75.061	19.3750
85	FA	B7	37+89.999	19.3750
86	RA	B8	36+70.499	27.1250
87	RA	B8	36+85.436	27.1250
88	RA	B8	37+00.374	27.1250
89	RA	B8	37+15.311	27.1250
90	RA	B8	37+30.249	27.1250
91	RA	B8	37+45.186	27.1250
92	RA	B8	37+60.124	27.1250
93	RA	B8	37+75.061	27.1250
94	FA	B8	37+89.999	27.1250
95	BB1	RE1	36+69.499	29.5000
96	RA	RE1	36+70.499	29.5000
97	RA	RE1	36+85.436	29.5000
98	RA	RE1	37+00.374	29.5000

99	RA	RE1	37+15.311	29.5000
100	RA	RE1	37+30.249	29.5000
101	RA	RE1	37+45.186	29.5000
102	RA	RE1	37+60.124	29.5000
103	RA	RE1	37+75.061	29.5000
104	FA	RE1	37+89.999	29.5000
105	BB2	RE1	37+90.899	29.5000



LEGEND

BB1 = APPROACH SLAB/BEGIN BRIDGE 1
BB2 = APPROACH SLAB/BEGIN BRIDGE 2
RA = REAR ABUTMENT
FA = FRONT ABUTMENT
LE = LEFT EDGE DECK
B1 = BEAM 1
B2 = BEAM 2
B3 = BEAM 3
B4 = BEAM 4
PG = PROFILE GRADE LINE
B5 = BEAM 5
B6 = BEAM 6
B7 = BEAM 7
B8 = BEAM 8
RE = RIGHT EDGE DECK

DESIGN AGENCY: KZF DESIGN

DESIGNED: DEF | DRAWN: DMG | REVIEWED: DMG | DATE: 12/19/08
CHECKED: DAT | REVISED: | STRUCTURE FILE NUMBER: 7336934

SCIOTO COUNTY
STA.
STA.

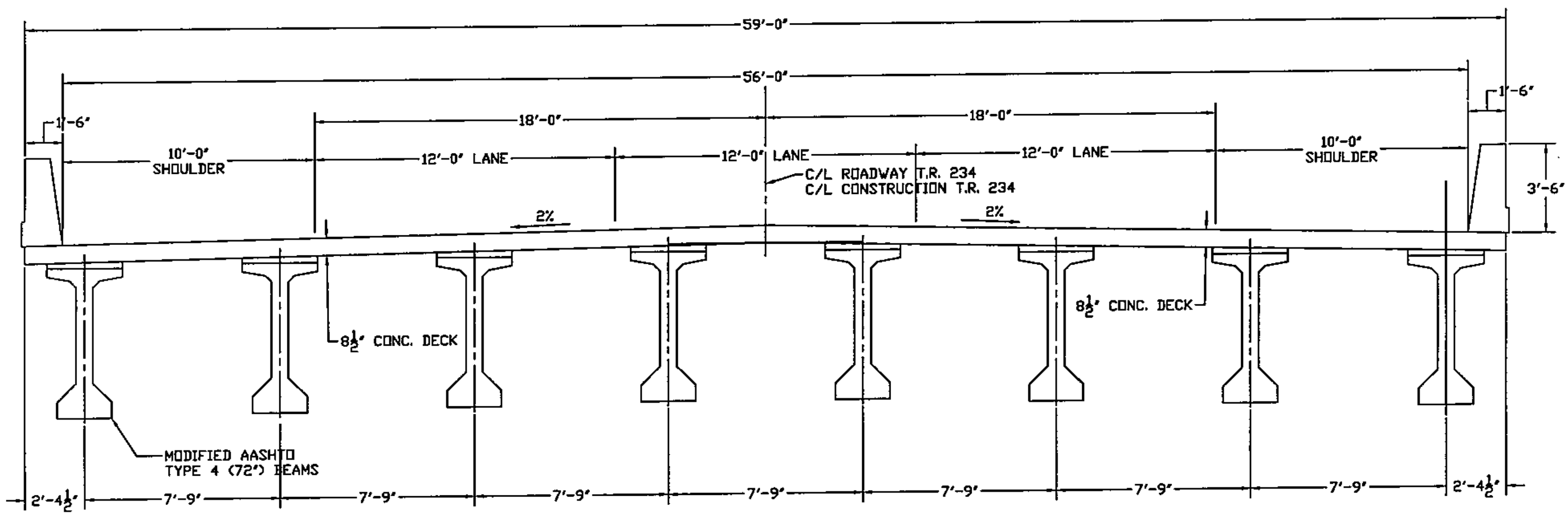
SCREED POINT LOCATIONS
BRIDGE NO. SCI-TR234-0122
SHUMWAY HOLLOW ROAD OVER CSXT RAILROAD

SCI-823-6.81
PID 19415

STRUCTURE NO. TR234-0122		SCREED TABLE										
KZF DESIGN DEC. 19, 2008		Approach Slab	CL Brg. Rear Abut.	1/8	2/8	3/8	4/8	5/8	6/8	7/8	CL Brg. Front Abut.	Approach Slab
LEFT EDGE DECK	Point #	1	2	3	4	5	6	7	8	9	10	11
	Station	36+69.499	36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	37+90.999
	Sta. Offset (CL)	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000
	Deck Elevation	661.58	661.57	661.42	661.27	661.12	660.97	660.82	660.67	660.52	660.37	660.36
	Deflection											
	Screed Elevation											
BEAM 1	Point #		12	13	14	15	16	17	18	19	20	
	Station		36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	
	Sta. Offset (CL)		27.1250	27.1250	27.1250	27.1250	27.1250	27.1250	27.1250	27.1250	27.1250	
	Deck Elevation		661.62	661.47	661.32	661.17	661.02	660.87	660.72	660.57	660.42	
	Deflection											
	Screed Elevation											
BEAM 2	Point #		21	22	23	24	25	26	27	28	29	
	Station		36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	
	Sta. Offset (CL)		19.3750	19.3750	19.3750	19.3750	19.3750	19.3750	19.3750	19.3750	19.3750	
	Deck Elevation		661.77	661.62	661.47	661.32	661.17	661.02	660.87	660.72	660.57	
	Deflection											
	Screed Elevation											
BEAM 3	Point #		30	31	32	33	34	35	36	37	38	
	Station		36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	
	Sta. Offset (CL)		11.6250	11.6250	11.6250	11.6250	11.6250	11.6250	11.6250	11.6250	11.6250	
	Deck Elevation		661.93	661.78	661.63	661.48	661.33	661.18	661.03	660.88	660.73	
	Deflection											
	Screed Elevation											
BEAM 4	Point #		39	40	41	42	43	44	45	46	47	
	Station		36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	
	Sta. Offset (CL)		3.8750	3.8750	3.8750	3.8750	3.8750	3.8750	3.8750	3.8750	3.8750	
	Deck Elevation		662.08	661.93	661.78	661.63	661.48	661.33	661.18	661.03	660.88	
	Deflection											
	Screed Elevation											
PROFILE GRADE LINE	Point #	48	49	50	51	52	53	54	55	56	57	58
	Station	36+69.499	36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	37+90.999
	Sta. Offset (CL)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
	Deck Elevation	662.17	662.16	662.01	661.86	661.71	661.56	661.41	661.26	661.11	660.96	660.95
	Deflection											
	Screed Elevation											
BEAM 5	Point #		59	60	61	62	63	64	65	66	67	
	Station		36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	
	Sta. Offset (CL)		3.8750	3.8750	3.8750	3.8750	3.8750	3.8750	3.8750	3.8750	3.8750	
	Deck Elevation		662.08	661.93	661.78	661.63	661.48	661.33	661.18	661.03	660.88	
	Deflection											
	Screed Elevation											
BEAM 6	Point #		68	69	70	71	72	73	74	75	76	
	Station		36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	
	Sta. Offset (CL)		11.6250	11.6250	11.6250	11.6250	11.6250	11.6250	11.6250	11.6250	11.6250	
	Deck Elevation		661.93	661.78	661.63	661.48	661.33	661.18	661.03	660.88	660.73	
	Deflection											
	Screed Elevation											
BEAM 7	Point #		77	78	79	80	81	82	83	84	85	
	Station		36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	
	Sta. Offset (CL)		19.3750	19.3750	19.3750	19.3750	19.3750	19.3750	19.3750	19.3750	19.3750	
	Deck Elevation		661.77	661.62	661.47	661.32	661.17	661.02	660.87	660.72	660.57	
	Deflection											
	Screed Elevation											
BEAM 8	Point #		86	87	88	89	90	91	92	93	94	
	Station		36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	
	Sta. Offset (CL)		27.1250	27.1250	27.1250	27.1250	27.1250	27.1250	27.1250	27.1250	27.1250	
	Deck Elevation		661.62	661.47	661.32	661.17	661.02	660.87	660.72	660.57	660.42	
	Deflection											
	Screed Elevation											
RIGHT EDGE DECK	Point #	95	96	97	98	99	100	101	102	103	104	105
	Station	36+69.499	36+70.499	36+85.436	37+00.374	37+15.311	37+30.249	37+45.186	37+60.124	37+75.061	37+89.999	37+90.999
	Sta. Offset (CL)	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000	29.5000
	Deck Elevation	661.58	661.57	661.42	661.27	661.12	660.97	660.82	660.67	660.52	660.37	660.36
	Deflection											
	Screed Elevation											

STRUCTURE NO. TR234-0122

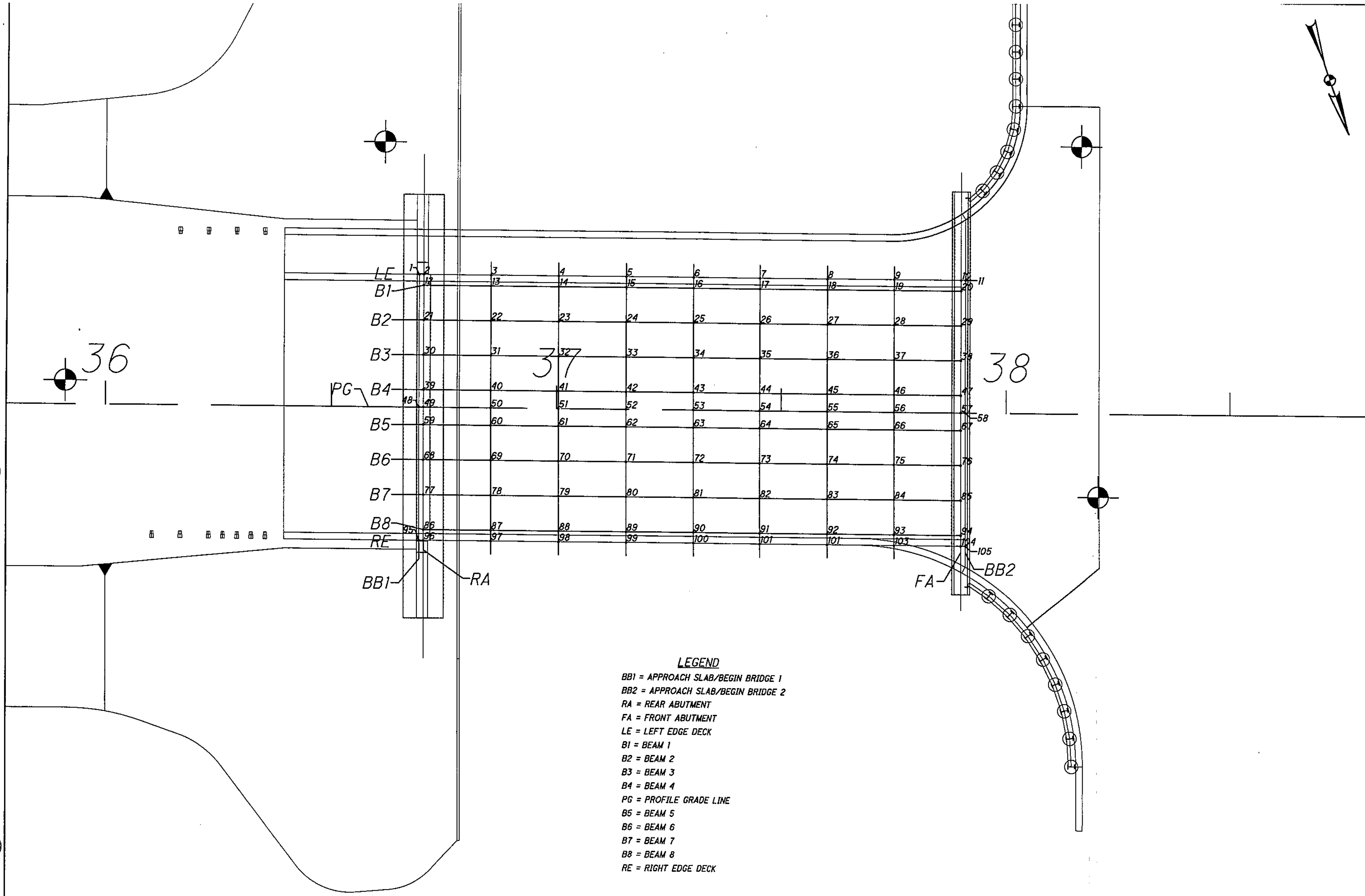
STRUCTURE NO. TR234-0122			BEAM SEAT ELEVATIONS	
KZF DESIGN			Rear Abutment	Forward Abutment
DEC. 19, 2008				
STRUCTURE NO. TR234-0122	BEAM 1	Point #	12	20
		Station	36+70.499	37+89.999
		Sta. Offset (CL)	27.1250	27.1250
		Deck Elevation	661.62	660.42
		Beam Seat Elev.	653.41	653.13
	BEAM 2	Point #	21	29
		Station	36+70.499	37+89.999
		Sta. Offset (CL)	19.3750	19.3750
		Deck Elevation	661.77	660.57
		Beam Seat Elev.	653.56	653.28
	BEAM 3	Point #	30	38
		Station	36+70.499	37+89.999
		Sta. Offset (CL)	11.6250	11.6250
		Deck Elevation	661.93	660.73
		Beam Seat Elev.	653.72	653.44
	BEAM 4	Point #	39	47
		Station	36+70.499	37+89.999
		Sta. Offset (CL)	3.8750	3.8750
		Deck Elevation	662.08	660.88
		Beam Seat Elev.	653.87	653.59
	BEAM 5	Point #	59	67
		Station	36+70.499	37+89.999
		Sta. Offset (CL)	3.8750	3.8750
		Deck Elevation	662.08	660.88
		Beam Seat Elev.	653.87	653.59
	BEAM 6	Point #	68	76
		Station	36+70.499	37+89.999
		Sta. Offset (CL)	11.6250	11.6250
		Deck Elevation	661.93	660.73
		Beam Seat Elev.	653.72	653.44
	BEAM 7	Point #	77	85
		Station	36+70.499	37+89.999
		Sta. Offset (CL)	19.3750	19.3750
		Deck Elevation	661.77	660.57
		Beam Seat Elev.	653.56	653.28
	BEAM 8	Point #	86	94
		Station	36+70.499	37+89.999
		Sta. Offset (CL)	27.1250	27.1250
		Deck Elevation	661.62	660.42
		Beam Seat Elev.	653.41	653.13



PROPOSED BRIDGE SECTION TR 234

SUBMITTED DATE: 11-25-08, BY: DEF

1.) Q..?
CONFIRM WITH THE ROADWAY DESIGN..??



LEGEND

BB1 = APPROACH SLAB/BEGIN BRIDGE 1
BB2 = APPROACH SLAB/BEGIN BRIDGE 2
RA = REAR ABUTMENT
FA = FRONT ABUTMENT
LE = LEFT EDGE DECK
B1 = BEAM 1
B2 = BEAM 2
B3 = BEAM 3
B4 = BEAM 4
PG = PROFILE GRADE LINE
B5 = BEAM 5
B6 = BEAM 6
B7 = BEAM 7
B8 = BEAM 8
RE = RIGHT EDGE DECK

DESIGN AGENCY KZF DESIGN

DRAWN DMG | REVIEWED DMG | DATE 12/19/08
DESIGNED DEF | CHECKED DAT | REVISED
STRUCTURE FILE NUMBER 7336934

SCIOTO COUNTY
STA.
STA.

SCREED POINT LOCATIONS
BRIDGE NO. SCI-TR234-0122
SHUMWAY HOLLOW ROAD OVER CSXT RAILROAD

SCI-823-6.81
PID 19415

project name: SCI-TR234-0122 sheet # 1 of 2

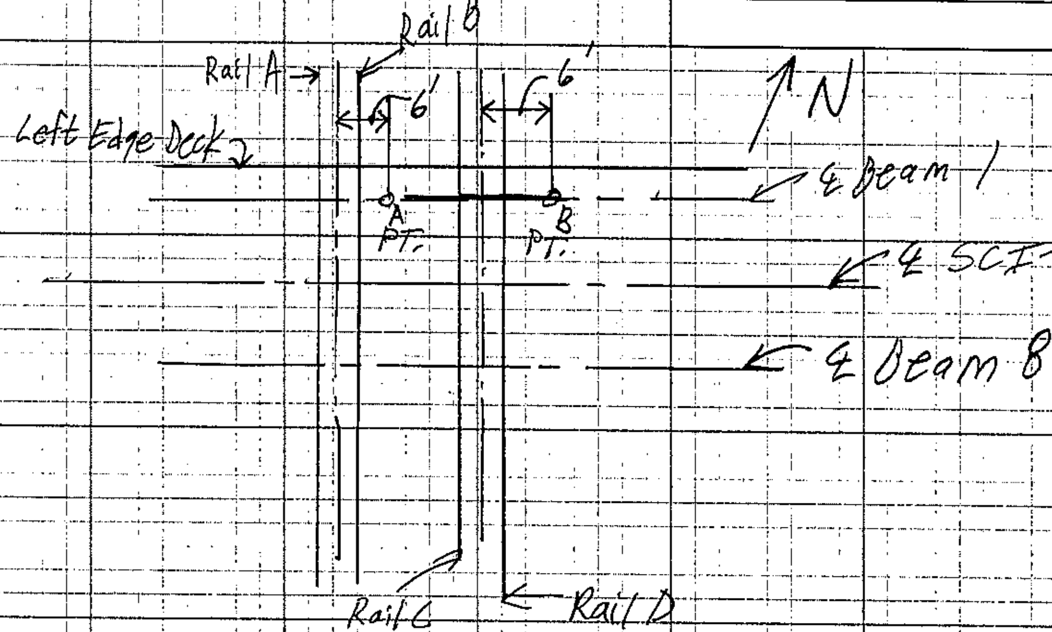
project # 5355.02

subject: RR VERTICAL CLEARANCE

date: 7-13-09

designed by: DAT

checked by:



℄ STA. AT PT. A = 37+41.77
PGL EL. = 662.8654 − 0.01(141.77)
= 661.4477

DECK EL. AT PT. A (BM. 1) = 661.4477 − 0.02(27.125)
= 660.9052

HAUNCH THICK. @ 11.52' FR. MIDSPAN = 2" + 0.11" = $\underline{\underline{2.11"}}$

$x^2 = -2py$

$p = \dfrac{(59.75')^2}{2\left(\frac{2.98"}{12}\right)} = 7188$

$y_{@11.52'} = \dfrac{(11.52')^2}{2(7188)} = 0.009' = 0.11"$

BOTTOM BEAM ELEV. @ PT. A = $660.9052 - \left(\dfrac{8.5" + 2.11" + 72"}{12}\right)$

= $\underline{\underline{654.0210}}$

℄ STA. AT PT. B = 37+54.66
PGL EL. = 662.8654 − 0.01(154.66)
= 661.3188

DECK EL. AT PT. B (BM. 1) = 661.3188 − 0.02(27.125)
= 660.7763

HAUNCH THICK. @ 24.41' FR. MIDSPAN = 2" + 0.50" = $\underline{\underline{2.50"}}$

$y_{@24.41'} = \dfrac{(24.41')^2}{2(7188)} = 0.041'$
= 0.50"

BOTTOM BEAM ELEV. @ PT. B = $660.7763 - \left(\dfrac{8.5" + 2.5" + 72"}{12}\right)$

= $\underline{\underline{653.8596}}$

℄ RR STA. AT PT. A = 452+28.76
RAIL B CONTROLS

℄ RR STA. AT PT. B = 452+28.70
RAIL D CONTROLS

FROM SUPPLEMENTAL
SITE PLAN

THROUGH INTERPOLATION

RR STA. 452+00, RAIL B EL. = 630.79
RR STA. 453+00, RAIL B EL. = 631.06

RR STA. 452+00, RAIL D EL. = 630.60
RR STA. 453+00, RAIL D EL. = 630.86

RAIL B EL. AT PT. A $= 630.79 + 28.76\left(\frac{31.06-30.79}{100'}\right)$

$= \underline{\underline{630.8677}}$

RAIL D EL. AT PT. B $= 630.60 + 28.70\left(\frac{30.86-30.60}{100'}\right)$

$= \underline{\underline{630.6746}}$

VERTICAL CLEARANCE
AT PT. A $= 654.0210 - 630.8677$

$= \underline{23.1533} > 23.0'$ REQ'D

VERTICAL CLEARANCE
AT PT. B $= 653.8596 - 630.6746$

$= \underline{23.1850} > 23.0'$ REQ'D.

project name ___ of ___ sheet #

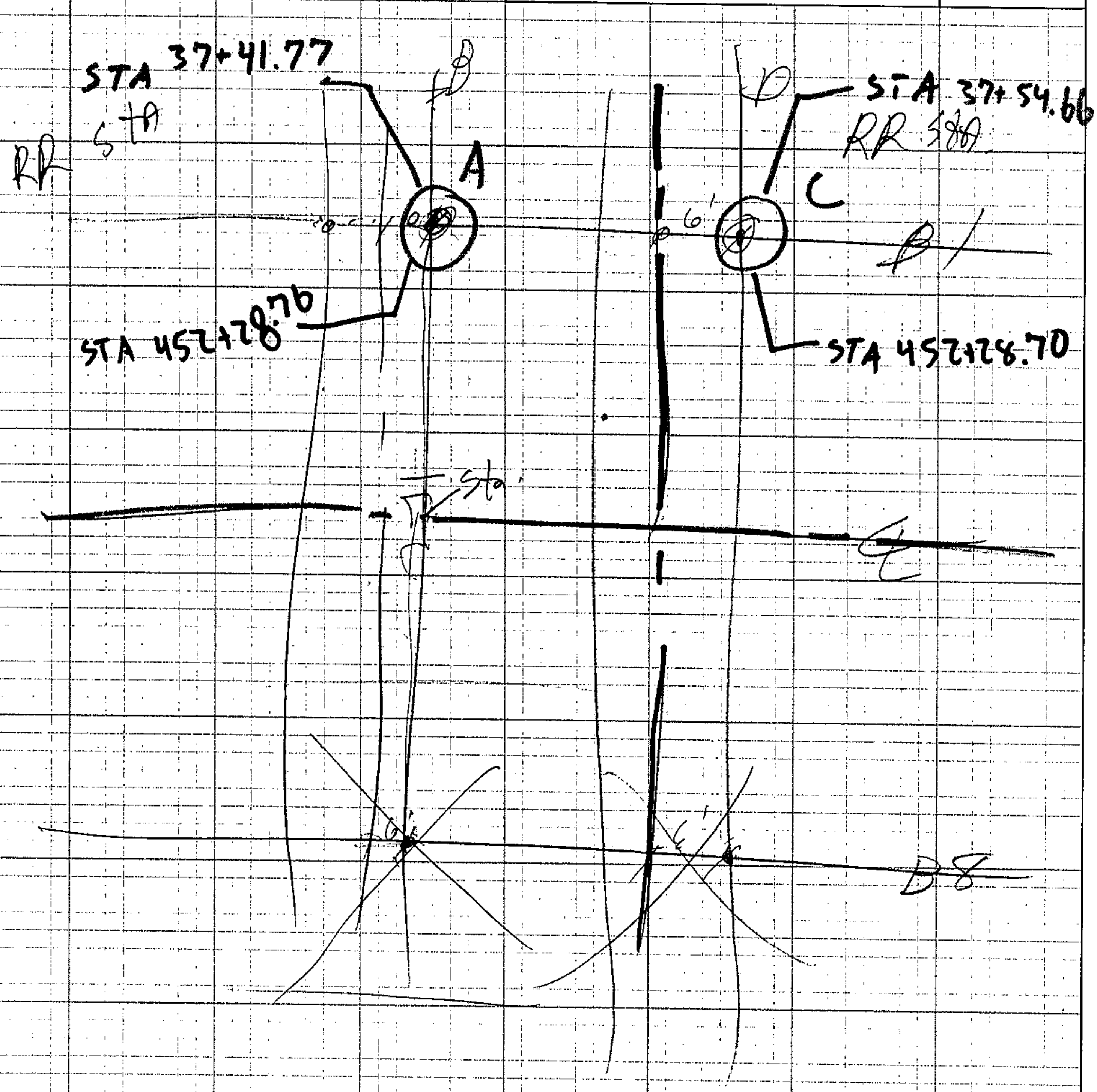
project #

subject

date

designed by

checked by



B. DECK DESIGN

BRIDGE: SINGLE SPAN 119'-6" ℄ BRG. (≈ OUT-TO-OUT BM = 121.5')
w/ REAR ABUTMENT - SPREAD FOOTING/MSE WALL
w/ FOR. Abutment - INTEGRAL WALL (14') & DRILLED
SHAFTS INTO BEDROCK.

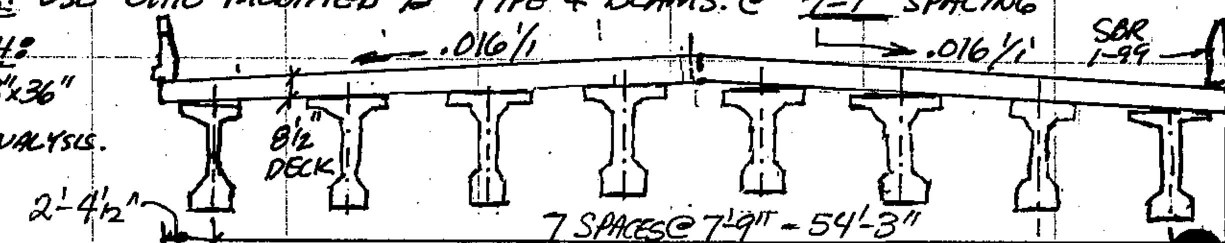
GRADE: -1%

DECK: 8½" CONC. DECK $f'_c = 4.5$ ksi, (1" INTEGRAL WEARING COURSE)

FWS: USE 60 PSF FWS.

BEAMS: USE OHIO MODIFIED 72" TYPE 4 BEAMS. @ 7'-9" SPACING

HAUNCH: USE 3"x36" IN ANALYSIS.



CODE: AASHTO STD SPEC & 2004 OHIO SPEC w/ INTERIMS.

LOADING: HS25 LOADING TRUCK/LANE w/ MILITARY LOAD CHECK.

BREAKING FORCE LONG = Use [121.5'(.64) + 18 = 95.76 K](1.25) x 4 LANES x .05 = 24 KIPS

BF/BM = (.67)(24 K/ROAD)/8 BMS = 2.01 K/BM {CAN REDUCE 75% FOR 4 LANES}

DEAD LOADS:

BARRIERS: USE SBR-1-99, Use wt = 530 plf

INTERMEDIATE BM BRACING: USE TYPE-4 STL ANGLE BRACING @ 1/4 PTS OR 25' MAX.

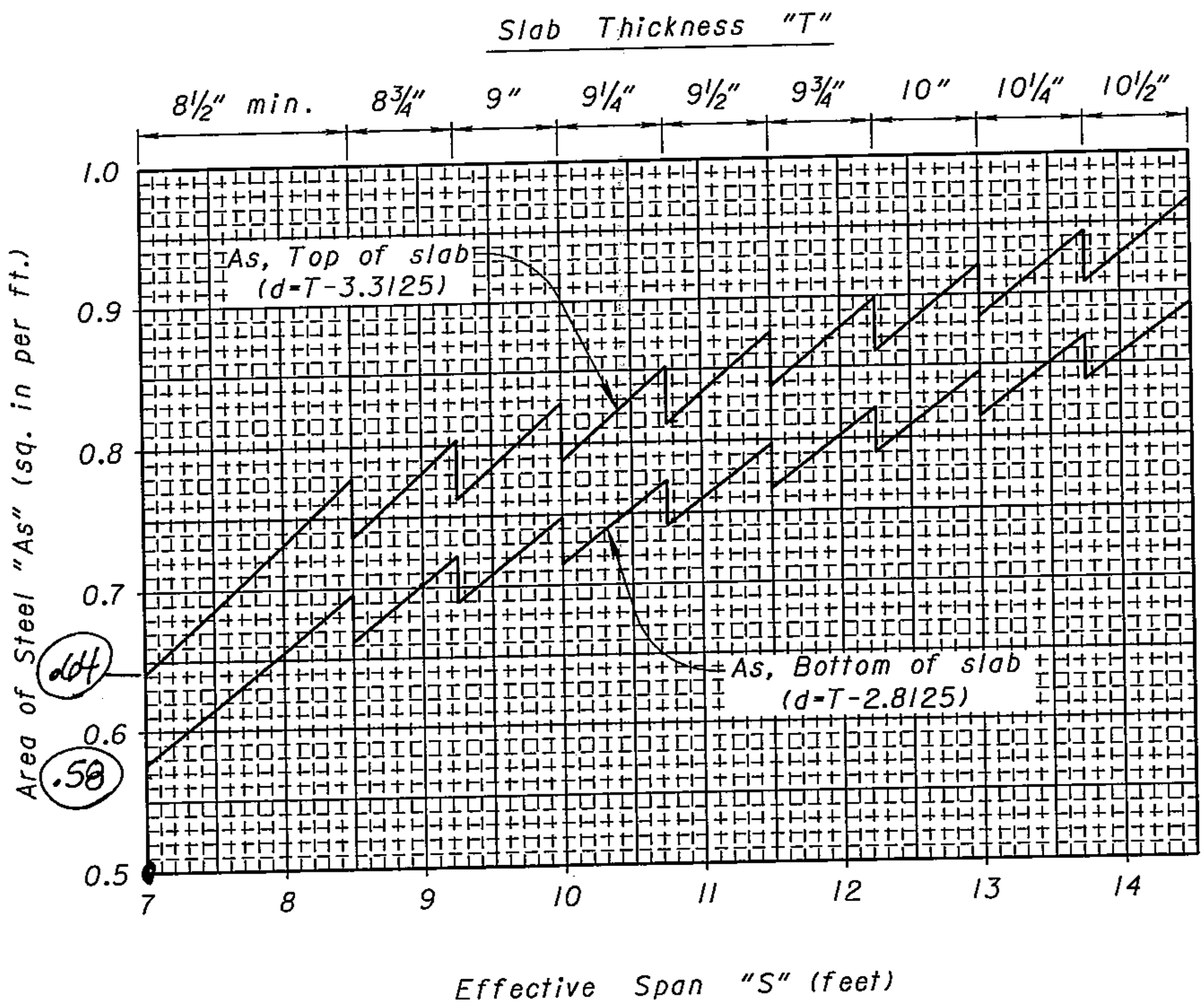
LIVE LOADS: HS25, HS25 LANE, OR MILITARY LOAD w/ 60 psf FWS.

IMPACT: $I = \frac{50}{L+125}$ ✓

WIND: AASHTO STANDARD WIND ANALYSIS.

LL DIST: DF = S/5.5 BRIDGE w/ 2+ LANES.

Prepared	Checked	Traced	Date
RZ	SAM	RZ	12-08-99



Note: This Figure is for the design of a reinforced concrete deck on new steel beams/girders using HS25.

Figure 314A

Deck Design SpreadSheet:: SCI-823: TR234@CSXT-RR

Design Code:	AASHTO LFD	
Loading:	HS25 Loading	
Wheel P Load=1.25x16=20 K=	20	kips
Concrete, F'c=	4500	psi
Reinforcement=	60000	psi
phi =	0.9	
Z (top slab) =	130	k/in
Z (bot slab) =	170	k/in
n =	8	
Impact =	1.3	
Fut. Wearing Sur=	60	psf
Beam Spacing=	7.75	feet
Top Flange Width=	36	in
S minimum Use =	6.25	ft
S Minimun Per Chart =	7	ft

""Summary""

Top Bars::
Use # 5 Bars at 5.50" o.c. =
Bottom Bars::
Use # 5 Bars at 5.50" o.c. =

Top Dist. #4 Long. Bars::
Use #4 Bars Spaced at = 8" o.c.
Bottom Dist. #5 Long. Bars::
Use #5 Bars at 8" o.c. Spacing

2.50 ft Overhang dist.
Top Bars::
Use # 5 Bars at 5.50" o.c. =

4.0 ft Overhang dist.
Top Bars::
Add #5 Bar between T/Bars
Spacing = 2.750 inches

S minimum Use = 7 feet

Tmin. =	S+17/36=	0.667	feet
	=	8.000	inches-Minimum
T Slab Selected=	8.5 inches		

Dead Loads:

Slab DL =	0.10625	k/ft ✓
FWS DL =	0.06	k/ft ✓
Total DL =	0.16625	k/ft ✓

Service Design Moments:

DL=0.125xS^2x0.8W=	0.814625	k-ft ✓
LL+I=1.3x0.8xPx(S+2)/32=	5.85	k-ft ✓
Total Service Mo:DL+(LL+I)=	6.664625	k-ft

Conc. Slab LFD Moment:

Mu=1.3(DL + 1.67(LL+I)) =	13.7593625	k-ft ✓

Top Reinforcement Design:::

d = T-2.5-0.5-0.375=	5.125	inches	=(T- 2.5"cover-#4 Bar-(1/2)#5)
Ru=Mu/(phi x b x d^2)=	582.0604138	psi ✓	
Reinf., rho =	0.010578728		
Area Steel, As = pbd =	0.650591798	in2/ft ✓	

Required:::::

Spacing# 5 Bar= 0.31x12/As=	5.717871037	in O.C.	= Use 5.5" ✓
Use # 5 Bars at 5.50" o.c. =	5.5	in O.C.	
Recalculated #5 Bar As/ft =	0.676363636	in2/ft	
Revised rho =As/dxb =	0.010997783		

Reinforcement Cracking and Spacing Control:

dc=2+(0.625/2) = 2.3125 inch
Ac=2x(dc x bar spacing) = 25.4375 in2

fs (All.)=z/(dcAc)^0.333= 33.47196035 Ksi
Maximun fs = 0.60xFy= 36 Ksi

Reinforce Stress Actual: fs::
K=(2pn+(pn)^2)^0.5= 0.428608681
j = 1-k/3 =......J= 0.85713044

fs(Act) =Mo/Asjd = 26.91757459 Ksi..... "OK"< 33.47 KSI

Bottom Reinforcement Design:::
d = T-1-1.5-0.375= 5.625 inches =(T-1-1.5"cover-(1/2)#5 Bar)

Ru=Mu/(phi x b x d^2)= 483.1820027 psi

Reinf., rho = 0.008638289
Area Steel, As = pbd = 0.583084474 in2/ft

Required:::::
Spacing# 5 Bar= 0.31x12/As= 6.37986461 in O.C. Required Minimum As

Use # 5 Bars at 5.50" o.c. = 5.5 in O.C. BDM 302.2.4.2 T&B Match

Recalculated #5 Bar As/ft = 0.676363636 in2/ft
Revised rho =As/dxb = 0.010020202

Reinforcement Cracking and Spacing Control:

dc=1.5+(0.625/2) = 1.8125 inch
Ac=2x(dc x bar spacing) = 19.9375 in2

fs (All.)=z/(dcAc)^0.333= 51.48159563 Ksi
Maximun fs = 0.60xFy= 36 Ksi....Controls

Reinforce Stress Actual: fs::
K=(2pn+(pn)^2)^0.5= 0.408349259
j = 1-k/3 =......J= 0.86388358

fs(Act) =Mo/Asjd = 24.33318551 Ksi..... "OK"< 36.00 KSI

Distribution Reinforcement:::
Top Dist. #4 Bars::
As (top)=0.33As = 0.2232 in2/ft
Use #4 Bars Spaced at = 10.75268817 in O.C. = Match B/Bars=Use 8.00"
Use #4 Bars Spaced at = 8" o.c. or Maximum 10.75" O.C.

Bottom Dist. #5 Bars::
As (bottom) % =220/(s)^0.5 = 83.15218406 Precent

Maximun Precent = 67 Precent

Use Precent = 67 Precent
Middle Half Design Only = 0.453163636 in2/ft

Spacing #5 Bars = 8.208955224 in O.C. = Use 8.00"

Use #5 Bars at 8" o.c. Spacing

Overhang Deck Design SpreadSheet:: OH Dist=2.5 ft.

Design Code:	AASHTO LFD	
Loading:	HS25 Loading	
Wheel Loading, P1 =	**20**	**kips**
Barrier Fx Loading, P2=	**10**	**Kips**
Overhang dist =	**2.5**	**feet**
Barrier width =	**1.5**	**feet**
Barrier height =	**3.5**	**feet**
Dist. C.G. Bar to Slab Edge=	**0.5**	**feet**
Barrier Weight; Use =	**530**	**PLF**
X1=O.D.-Bw-1 ft =	**0**	**ft**
X2=O.D.-Barrier Edge dist=	**2**	

Truck Load Distribution Factor:

E1=0.8(X1)+3.75 =	**3.75**	**feet**

Railing Distribution Factor:

E2=0.8(X2)+5 =	**6.6**	**feet**
Slab T overhang = T+2" =	**10.5**	**inches**

Uniform Dead Loads::

Slab =	**0.131**	**ksf**
FWS =	**0.06**	**ksf**

Dead Load Moments: (per ft of Length):

DLM=MoSlab+MoFWS+Barrier

DLM Slab=0.5w1L^2 =	0.410	K-ft
DLM FWS =0.5w2(L-1.5)^2 =	0.03	K-ft
DLM Parapet =P(L-0.5)=	1.06	K-Ft
......Total DLM =	**1.500**	**K-Ft**

Service LL+I Design Moments:

LL+I=1.3xPxd / E1 =	0.000	k-ft
Rail LL M= P2xh / E2 =	5.303	k-Ft
CONTROLING LL Moment =	5.303	K-FT
Total Service Mo:DL+(LL+I)=	6.803	k-ft

Conc. Slab LFD Moment:

Mu=1.3(DL + 1.67(LL+I)) =	13.463	k-ft

Conc. Slab Service Moment:

Mu=(DL + (LL+I)) =	6.803	k-ft

Top Reinforcement Design:::

d = T2-2.5-0.5-0.375=	7.125	inches	=(T2- 2.5"cover-#4 Bar-(1/2)#5)

Ru=Mu/(phi x b x d^2)=	294.6675019 psi		
Reinf., rho =	0.005116443		
Area Steel, As = pbd =	0.437455839 in2/ft		
Required:::::			
Spacing# 5 Bar= 0.31x12/As=	8.50371551 in O.C.		= Use 5.50"
Use # 5 Bars at 5.50" o.c. =	5.5	in O.C.	Match Top Bar Spacing
Recalculated #5 Bar As/ft =	0.676363636 in2/ft		
Revised rho =As/dxb =	0.007910686		

Reinforcement Cracking and Spacing Control:

dc=2+(0.625/2) =	2.3125 inch
Ac=2x(dc x bar spacing) =	25.4375 in2
fs (All.)=z/(dcAc)^0.333=	33.47196035 Ksi
Maximun fs = 0.60xFy=	36 Ksi

Reinforce Stress Actual: fs::

K=(2pn+(pn)^2)^0.5=	0.361353049			
j = 1-k/3 =......J=	0.879548984			
fs(Act) =Mo/Asjd =	19.26054074 Ksi.....	"OK"<	33.47	KSI

Overhang Deck Design SpreadSheet:: OH DIST. = 4.0 Ft

Design Code:	AASHTO LFD
Loading:	HS25 Loading
Wheel Loading, P1 =	20 kips
Barrier Fx Loading, P2=	10 Kips
Overhang dist =	4 feet
Barrier width =	1.5 feet
Barrier height =	3.5 feet
Dist. C.G. Bar to Slab Edge=	0.5 feet
Barrier Weight; Use =	530 PLF
X1=O.D.-Bw-1 ft =	1.5 ft
X2=O.D.-Barrier Edge dist=	3.5

Truck Load Distribution Factor:

E1=0.8(X1)+3.75 = 4.95 feet

Railing Distribution Factor:

E2=0.8(X2)+5 = 7.8 feet

Slab T overhang = T+2" = 10.5 inches

Uniform Dead Loads::

Slab =	0.131 ksf
FWS =	0.06 ksf

Dead Load Moments: (per ft of Length):

DLM=MoSlab+MoFWS+Barrier

DLM Slab=0.5w1L^2 =	1.050	K-ft
DLM FWS =0.5w2(L-1.5)^2 =	0.1875	K-ft
DLM Parapet =P(L-0.5)=	1.855	K-Ft
......Total DLM =	3.093	K-Ft

Service LL+I Design Moments:

LL+I=1.3xPxd / E1 =	7.879	k-ft
Rail LL M= P2xh / E2 =	4.487	k-Ft
CONTROLING LL Moment =	7.879	K-FT
Total Service Mo:DL+(LL+I)=	10.971	k-ft

Conc. Slab LFD Moment:

Mu=1.3(DL + 1.67(LL+I)) = 21.125 k-ft

Conc. Slab Service Moment:

Mu=(DL + (LL+I)) = 10.971 k-ft

Top Reinforcement Design:::

d = T2-2.5-0.5-0.375=	7.125	inches	=(T2- 2.5"cover-#4 Bar-(1/2)#5)
Ru=Mu/(phi x b x d^2)=	462.3666438	psi	
Reinf., rho =	0.008238439		
Area Steel, As = pbd =	0.704386552	in2/ft	

Required:::::

Spacing# 5 Bar= 0.31x12/As= 5.281191115 in O.C.

Add #5 Bar between T/Bars	2.75	in O.C.	Min. Match Top Bar Spacing

Recalculated #5 Bar As/ft =	1.352727273	in2/ft
Revised rho =As/dxb =	0.015821372	

Reinforcement Cracking and Spacing Control:

dc=2+(0.625/2) =	2.3125	inch
Ac=2x(dc x bar spacing) =	12.71875	in2
fs (All.)=z/(dcAc)^0.333=	42.16228474	Ksi
Maximun fs = 0.60xFy=	36	Ksi

Reinforce Stress Actual: fs::

K=(2pn+(pn)^2)^0.5=	0.518808401
j = 1-k/3 =......J=	0.827063866

fs(Act) =Mo/Asjd = 10.24140319 Ksi..... "OK"< 36.00 KSI

- FOR BRIDGE NO. SCI-TR234-0122 (SKEW = 0°)

RAIL-TO-RAIL L = 1.00 W = 1.00(59.00') = 59.0'

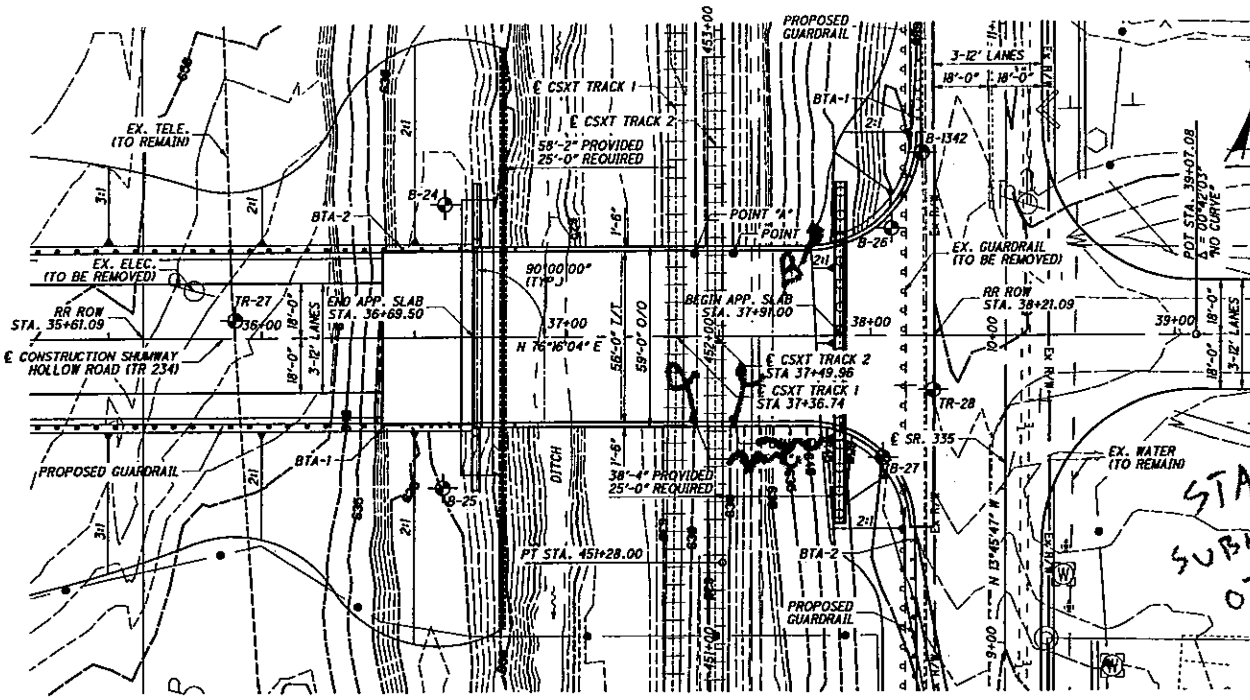
EXTRA END L = 0 ft.

TOTAL FINISHING MACHINE L = 2(0.0') + 59.0' = $\underline{59.0'}$

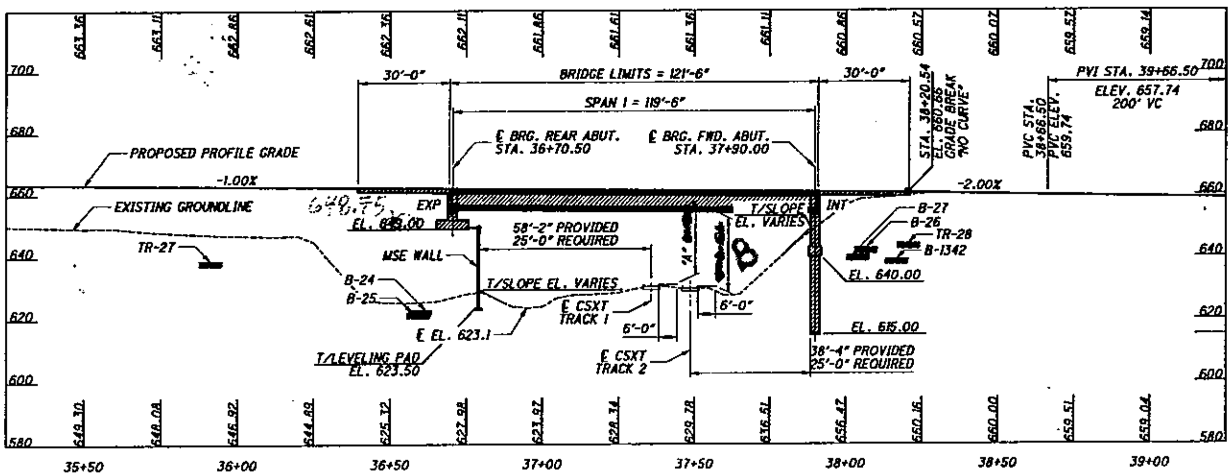
TOTAL MACHINE WEIGHT = 7.6 K + 0.09(59.0' - 36')

= $\underline{\underline{9.67\ K}}$

MAXIMUM WHEEL LOAD = 9.67/8 = $\underline{\underline{1.21\ K}}$



PLAN



PROFILE ALONG ℄ CONSTRUCTION SHUMWAY HOLLOW ROAD

BENCHMARK DATA

BM #7	STA. 381+08,	ELEV. 646.20',	OFFSET 557', RT.
BM #8	STA. 386+68,	ELEV. 659.05',	OFFSET 558', RT.

FOR ADDITIONAL BENCHMARK INFORMATION. SEE ROADWAY PLAN SHEET

NOTES

EARTHWORK LIMITS SHOWN ARE APPROXIMATE. ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.

DESIGN TRAFFIC:
2010 ADT = 3800 2010 ADTT = 228
2030 ADT = 7800 2030 ADTT = 468
DIRECTIONAL DISTRIBUTION =

FOUNDATION DATA:
REAR ABUTMENT SHALL BE ON SPREAD FOOTING WITH AN ALLOWABLE BEARING CAPACITY OF 4 KSF.
FORWARD ABUTMENT SHALL BE ON DRILLED SHAFTS WITH AN ALLOWABLE BEARING CAPACITY OF 40 KSF.

LEGEND

BTA-1 = BRIDGE TERMINAL ASSEMBLY TYPE 1
BTA-2 = BRIDGE TERMINAL ASSEMBLY TYPE 2
⌖ = SOIL BORING LOCATION

23.15'

STAGE 2
SUBMITTED
02/06/09

TABLE OF VERTICAL CLEARANCES				
LOCATION	"A" *	"B" *	"C" *	"D" *
PROPOSED	[illegible]	[illegible]	24.35'	24.51'
REQUIRED	23.0'	23.0'	23.0'	23.0'

* 6'-0" FROM ℄ CSXT TRACK

23.19

BORING INFORMATION	
BORING ID	T/ROCK EL.
B-24	622.9
B-25	622.0
B-26	642.7
B-27	640.3
TR-27	638.8
TR-28	644.2
B-1342	639.1

GUARDRAIL POST		
LOCATION	STATION	OFFSET
REAR ABUT	36+64.75	29.08' LT.
REAR ABUT	36+64.85	29.08' RT.
FWD ABUT	9+33.36	30.75' LT.
FWD ABUT	10+69.69	29.08' LT.

PROPOSED STRUCTURE

TYPE: SINGLE SPAN 72" MODIFIED AASHTO TYPE 4 PRESTRESSED CONCRETE I-BEAMS WITH COMPOSITE REINFORCED CONCRETE DECK ON SEMI-INTEGRAL REAR ABUTMENT AND INTEGRAL FORWARD ABUTMENT
SPANS: 119'-6" C/C BEARINGS
ROADWAY: 56'-0" T/T BARRIER
LOADING: HS25 CASE II AND ALTERNATE MILITARY FWS = 60 PSF
SKEW: NONE
WEARING SURFACE: MONOLITHIC CONCRETE
APPROACH SLABS: AS-1-81 (30'-0" LONG)
ALIGNMENT: TANGENT
CROWN: NORMAL
COORDINATES: LATITUDE 38°50'30" N
LONGITUDE 82°51'00" W

KZF DESIGN

DESIGNED	CHECKED	DRAWN	REVIEWED	REVIEWED	DATE
DEF	DAT	RBK	—	BMB	02/13/09

STRUCTURE FILE NUMBER 7336934

SCIOTO COUNTY
STA. 36+39.50
STA. 37+91.00

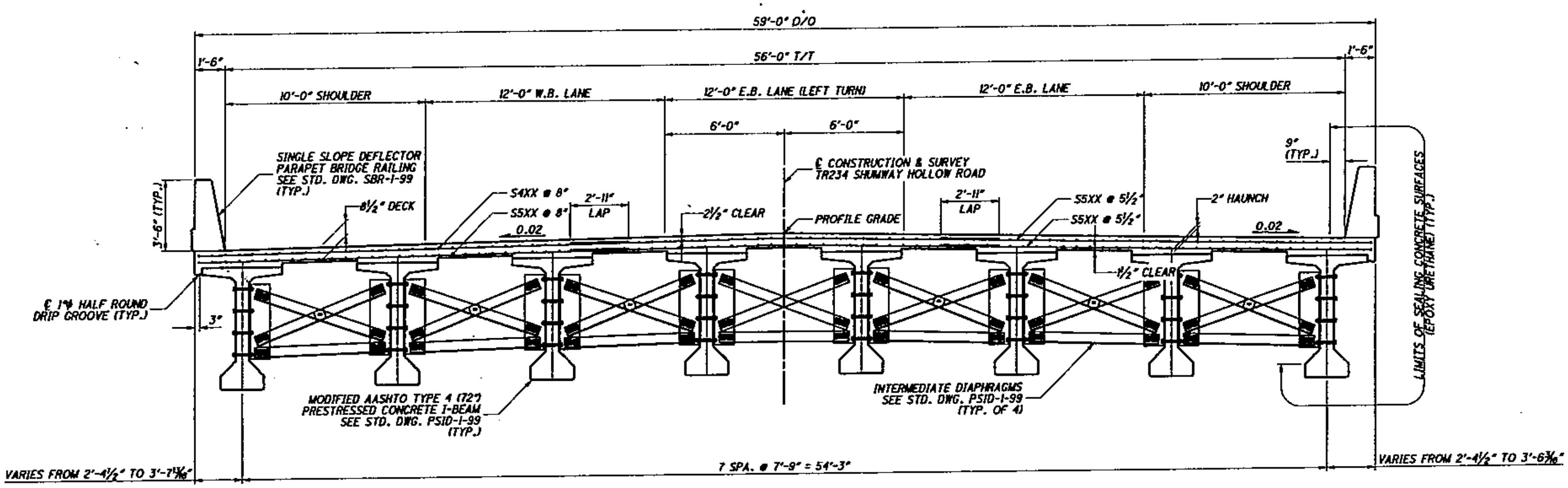
SITE PLAN
BRIDGE NO. SCI-TR234-0122
SHUMWAY HOLLOW ROAD OVER CSXT RAILROAD

SCI-823-6.81
PID No. 19415

1 / 14

XX / XXX

_0122CSP001.dgn 2/9/2009 7:22:24 AM



BRIDGE TYPICAL SECTION
BRIDGE NO. SCI-TR234-0122

NOTES:
1. MIN. LAP LENGTH #4 BAR = 27"
2. MIN. LAP LENGTH #5 BAR = 35"

DESIGNED	DRAWN	REVIEWED	DATE
DEF	RBK	BMB	02/13/09
CHECKED	REVISED	STRUCTURE FILE NUMBER	
DAT	—	7336934	

DESIGN AGENCY: KZ DESIGN

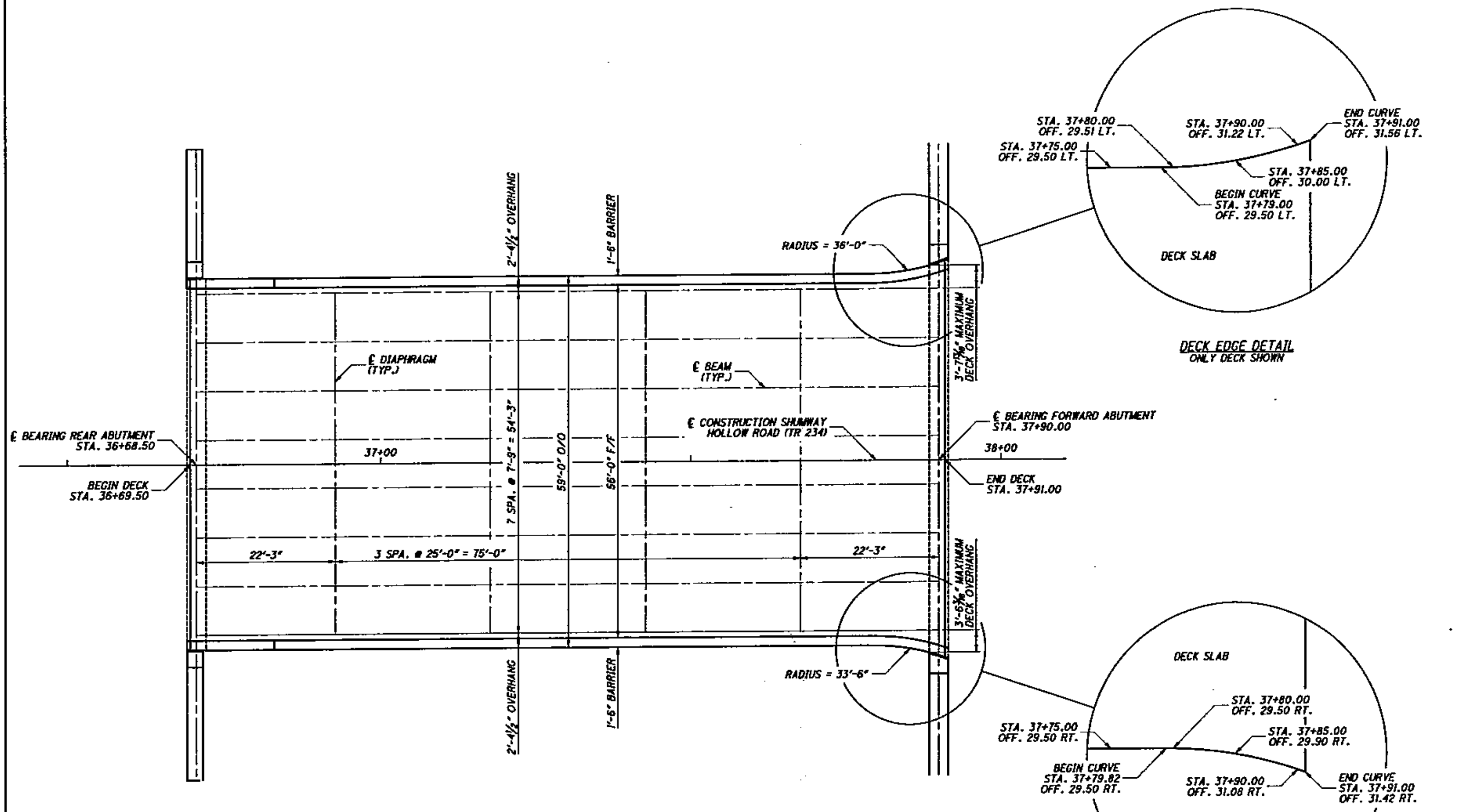
TRANSVERSE SECTION
BRIDGE NO. SCI-TR234-0122
SHUMWAY HOLLOW ROAD OVER CSXT RAILROAD

SCI-823-6.81
PID No. 19415

12 / 14

XX / XXX

_0122CTS001.dgn 2/9/2009 7:24:28 AM



PLAN
BRIDGE NO. SCI-TR324-0122

DECK EDGE DETAIL
ONLY DECK SHOWN

DECK EDGE DETAIL
ONLY DECK SHOWN

KZF DESIGN

DESIGNED	DRAWN	REVIEWED	DATE
DEF	RBK	BMB	02/13/09
CHECKED	REVISED	STRUCTURE FILE NUMBER	
DAT	—	7336934	

BRIDGE DECK LAYOUT
BRIDGE NO. SCI-TR234-0122
SHUMWAY HOLLOW ROAD OVER CSXT RAILROAD

SCI-823-6.81
PID No. 19415

13 / 14

XX / XXX

I_0122CDP001.dgn 2/9/2009 7:24:19 AM

This reinforcement should be placed approximately symmetrical to the centerline of pier bearings but with every other reinforcing bar staggered 3 feet [1000 mm] longitudinally.

For composite designs, the total longitudinal reinforcement over a pier shall meet the requirements of AASHTO.

302.2.4.2 TRANSVERSE

To facilitate the placement of reinforcing steel and concrete in transversely reinforced deck slabs top and bottom main reinforcement shall be equally spaced and placed to coincide in a vertical plane.

For steel beam or girder bridges with a skew of less than 15 degrees the transverse reinforcing may be shown placed parallel to the abutments. Bridges with a skew greater than 15 degrees or where the transverse reinforcing will interfere with the shear studs should have the transverse reinforcement placed perpendicular to the centerline of the bridge. Refer to the appropriate Standard Bridge Drawing for the requirements on slab bridges.

For prestressed I-beams, transverse reinforcing shall be placed perpendicular to the centerline of the bridge.

For composite box beam decks, the transverse reinforcing steel may be placed parallel to the abutment.

For steel beam or girder bridges, the clearance of the bottom transverse bars over the top of any bolted beam splice plates or moment plates should be checked as reinforcing bars at a skew generally cannot be placed between bolt heads.

302.2.5 HAUNCHED DECK REQUIREMENTS

Concrete decks on steel beam, girder or prestressed I-beam structures shall have a concrete haunch to prevent a thinning of the deck slab as a result of unforeseen variations in beam camber. At a minimum, the design haunch shall allow for 2 inches [50 mm] of excessive camber. For steel beam and girder structures, the haunch shall be tapered back to the original concrete deck thickness in a 9 inch [225 mm] length and the concrete haunch shall encase the edges of the top flange. See Figures 317 & 318.

302.2.6 STAY IN PLACE FORMS

Galvanized steel or any other material type, stay in place forms, shall not be used.

302.2.7 CONCRETE DECK PLACEMENT CONSIDERATIONS

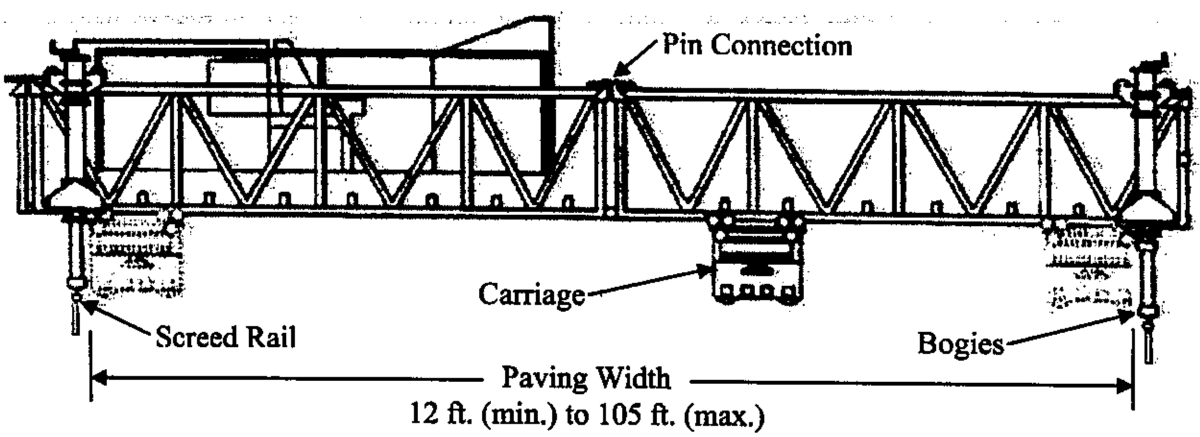
SCI-823

Mechanized finishing machines are preferred to hand finishing methods for both consistency of surface finish and economics. Designers should be aware of finishing machine limitations in order to avoid deck designs that require hand finishing methods.

The placement of deck concrete using mechanized finishing machines alone does not ensure a smooth riding surface. Achieving a smooth riding surface as well as ensuring the proper geometry of the concrete deck is further complicated by deflections of the concrete falsework and of the main structural support members during the placement operation. The Contractor is responsible for designing falsework and finishing machine support to minimize deflection during placement, but the Designer is responsible for deflections induced by deck placement on the superstructure. Many complications due to deflection during placement can be avoided with proper design considerations.

302.2.7.1 FINISHING MACHINES

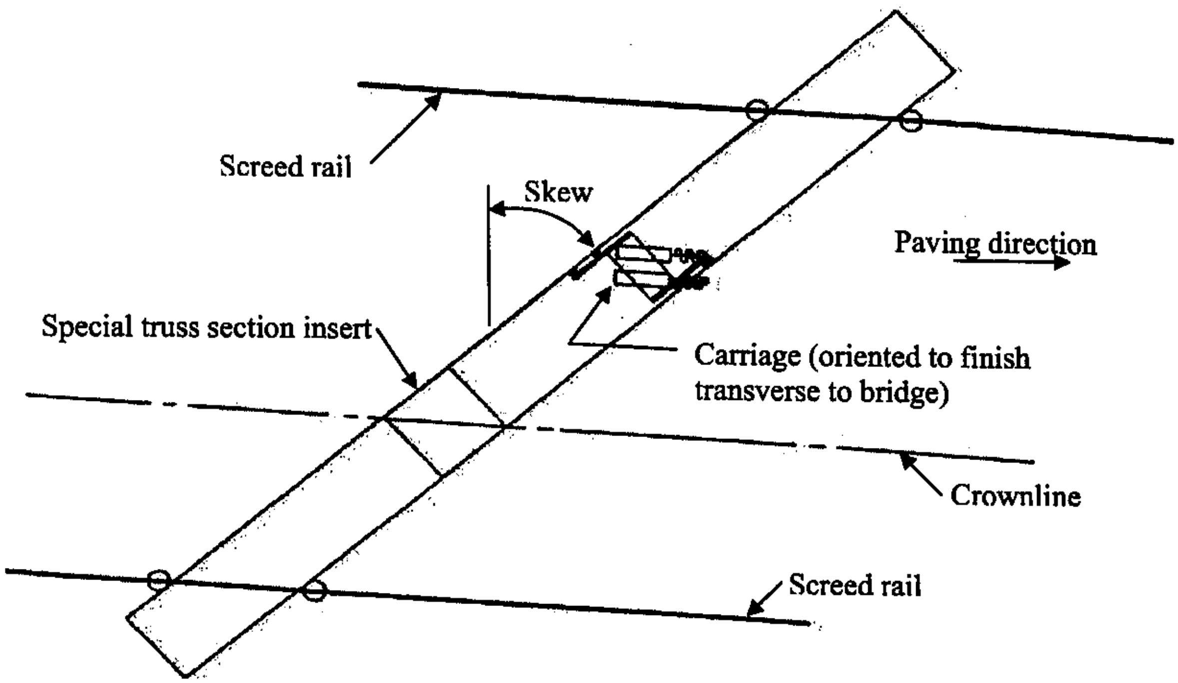
Mechanized finishing machines are comprised of fabricated truss sections pinned together to span the bridge deck width to be paved. The truss spans are supported at each end on a set of wheels, called "bogies," which ride along the length of the bridge on screed rails. Suspended below the truss is a finishing head, called a "carriage," which levels, compacts, vibrates and finishes the concrete.



Finishing machines can be placed such that the truss sections are skewed with respect to the screed rails. This orientation allows for concrete placement parallel to the substructure skew as required by the C&MS 511. For skew angles of 15° and greater, the finishing machine can be skewed to within 5 degrees of the plan specified skew angle.

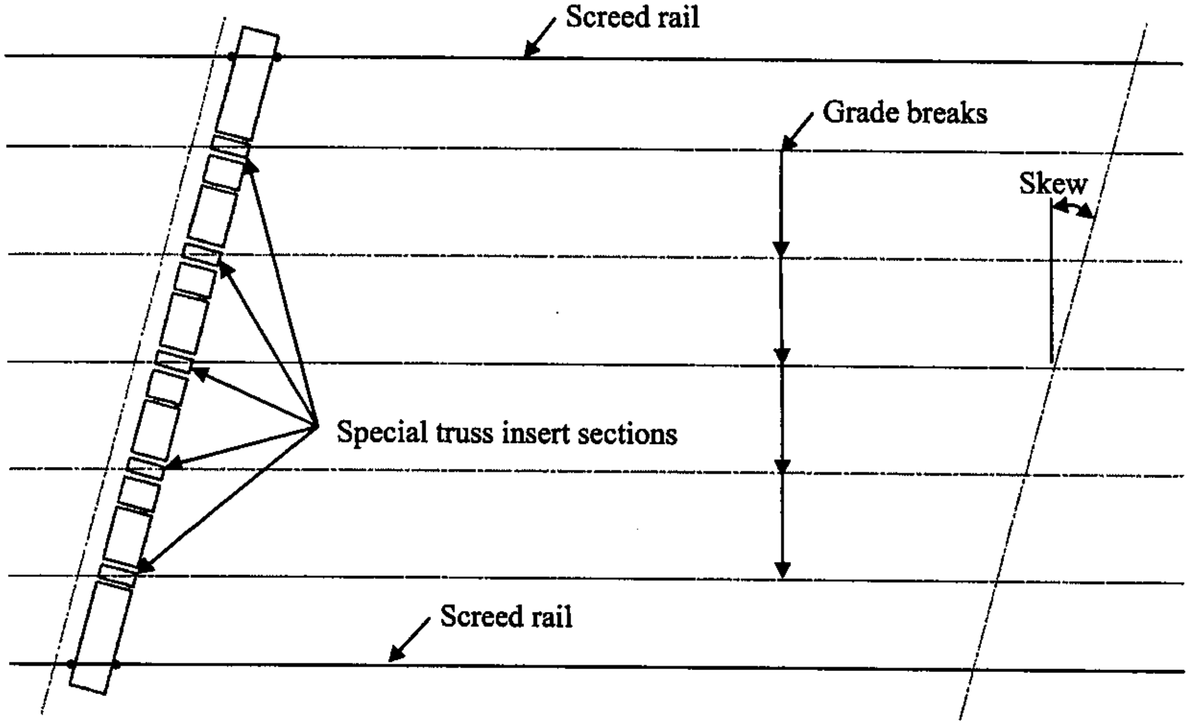
The carriage can also be skewed with respect to the truss sections. This feature allows the carriage to finish the concrete transverse to the bridge when the truss sections are placed at some other orientation (e.g. parallel to the substructure skew). In order to ensure a proper finish at transverse grade breaks (e.g. crown points), the carriage should always be oriented to finish the concrete transverse to the bridge. A special length truss section insert is required above the grade break locations such that the grade break line lies directly below opposite corners of the section.

For skewed bridges without transverse grade breaks, skewing the carriage with respect to the truss sections is not required.

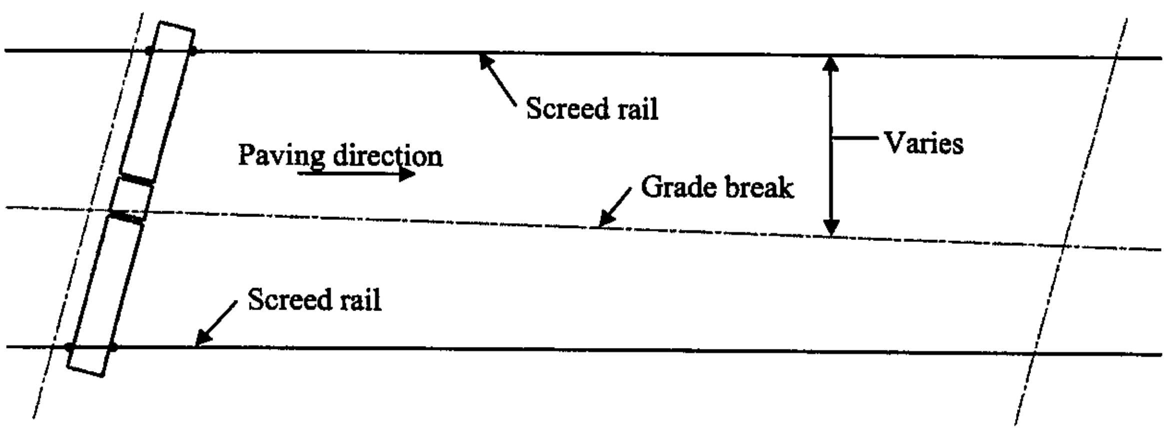


Most finishing machines do not easily accommodate non-parallel rails. The distance between the screed rails should be a fixed width. Designs that require tapered paving widths should be avoided.

The finishing machines can be hinged at the pin connections between truss sections in order to provide transverse grade breaks (e.g. crown points). In theory, multiple transverse grade breaks can be accommodated, but the grade breaks must remain at a fixed spacing in order to line up with a pin connection. The figure below illustrates the complexity of the machine set-up to accommodate multiple grade breaks in a transverse section placed on a skew. Note that the length of truss sections required between grade breaks must fit the standard truss section lengths.



Grade break locations that move laterally along the length of the bridge cannot be paved in a single operation using a mechanized finishing machine and should therefore be avoided. Note that as the machine progresses forward, the truss hinge locations and the grade break locations no longer coincide. See the figure below.



302.2.7.2 SOURCES OF GIRDER TWIST

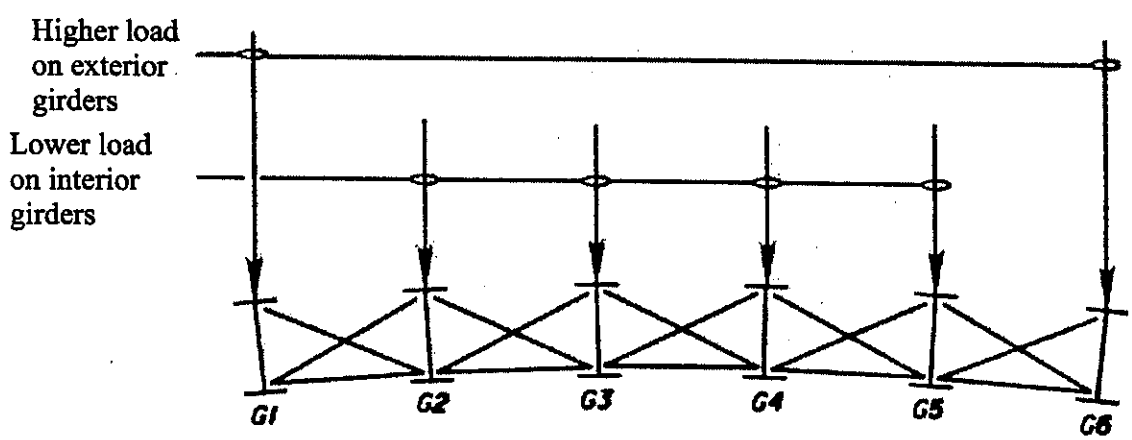
The interconnectivity between girders, intermediate crossframes/diaphragms and end crossframes/diaphragms is essential to a structure's stability throughout the construction process. Therefore, it is of utmost importance to ensure that all crossframes/diaphragms are fully installed prior to deck placement. Failure to do so may lead to construction disputes, expensive repairs and lengthy construction delays or even impact project safety. One major drawback to this interconnectivity is that the deflection caused by the placement of the concrete deck will result in girder twisting.

There are primarily three independent sources of girder twist resulting from deck placement. This manual will refer to these sources as: global superstructure distortion, oil-canning and girder warping.

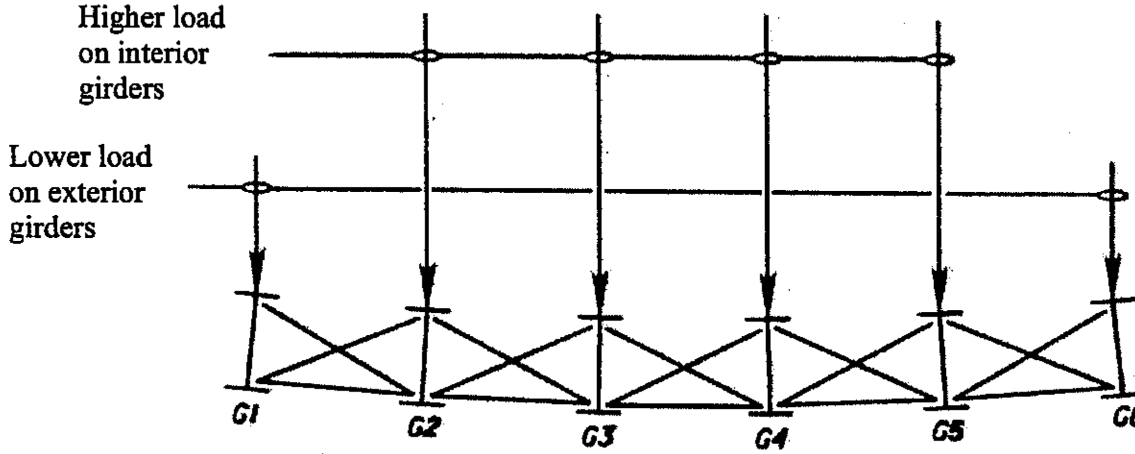
302.2.7.2.a GLOBAL SUPERSTRUCTURE DISTORTION

Global superstructure distortion is distortion of the bridge transverse section primarily caused by differential deflections between adjacent girders. As a girder deflects downward with respect to an adjacent girder, the rigidity of the cross framing between the two girders causes the deflecting girder to rotate as it deflects. This distortion may occur with both steel and prestressed concrete superstructures. The most common differential deflections occur between the exterior girders and adjacent interior girders for a given construction phase when the loaded tributary areas over the girders differ.

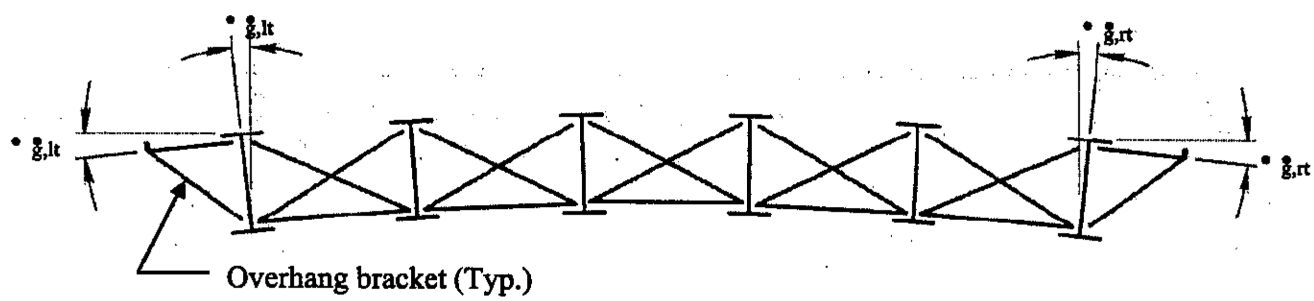
Transverse sections with more heavily loaded exterior girders distort in a convex shape.



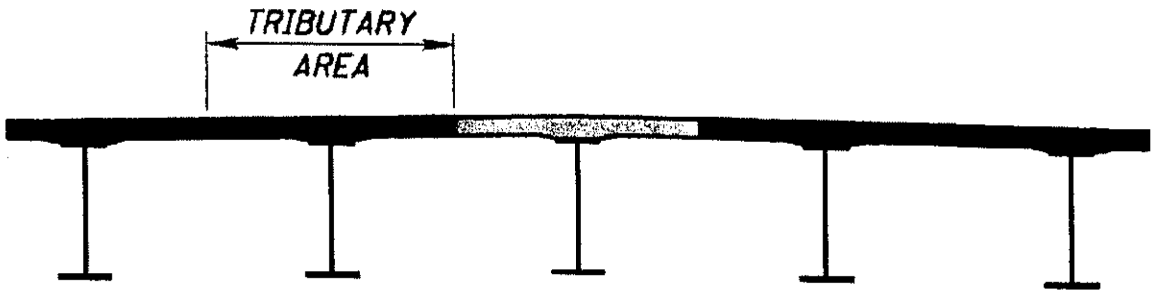
Transverse sections with more heavily loaded interior girders distort in a concave shape.



Twisting of the exterior girders can result in deck thickness and cover loss if the screed rails are supported on cantilevered falsework. The magnitude of girder twist (measured as • g) will vary over the length of the bridge and will be different for the left and right sides if loading or geometry is not symmetrical.



For bridges with tangent alignments and adjacent substructure skews that vary by no more than 15°, the magnitude of the girder twist can be reduced by utilizing transverse sections with balanced tributary deck loadings. For a new superstructure, the amount of girder twist due to global superstructure deformation can be neglected when the tributary deck load carried by the fascia girder does not exceed 110% of the average of the tributary deck load carried by the interior members for a given construction phase. For an existing superstructure, the amount of global deformation may be neglected when the tributary deck load carried by the fascia girder does not exceed 115% of the average of the tributary deck load carried by the interior members for a given construction phase.

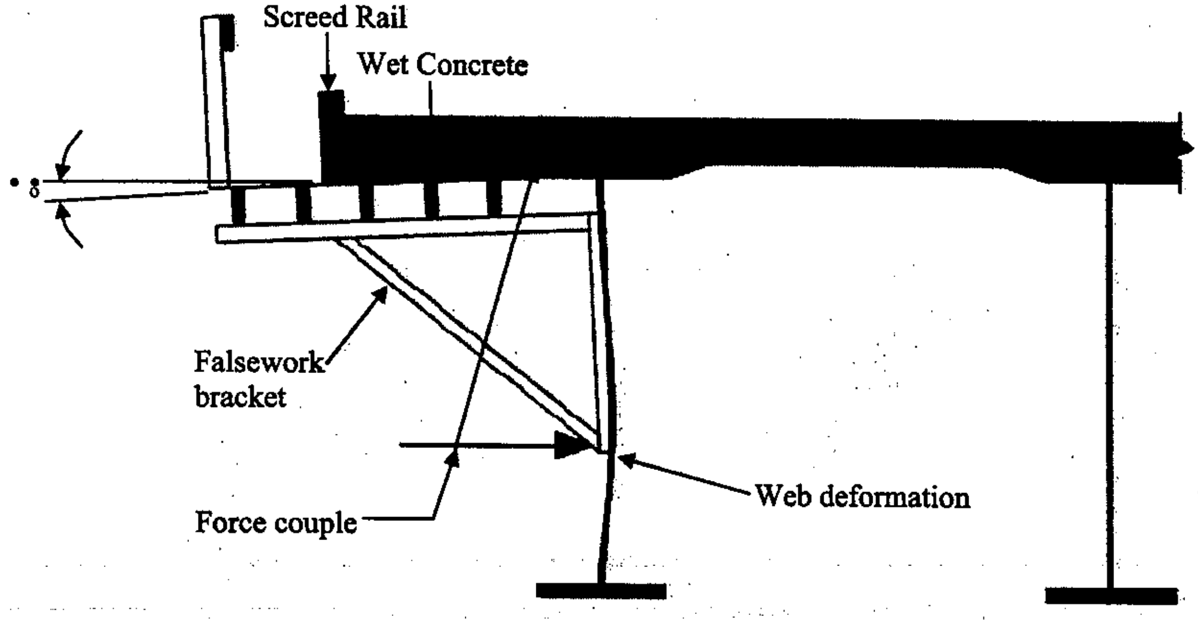


When the aforementioned tributary deck loading requirements of the fascia members cannot be met or, because of geometry, do not apply, the Designer shall perform a refined analysis of the superstructure system to determine the magnitude of fascia girder twist (•$_g$) due to deck concrete placement. To properly calculate the effect of the twist angle on deck thickness, the analysis should be based on the deflection occurring due to the concrete present at the time that the finishing machine passes over the point under consideration. This degree of precision requires a separate refined analysis for each point of consideration. It is generally sufficient to calculate •$_g$ based on the full wet concrete load placed over the entire structure. However, on complex structures with variable skews and/or curved girders, a higher degree of precision may be warranted to ensure proper deck thickness.

Additional measures to reduce global deformation include: adding or stiffening the crossframes/diaphragms; and increasing the stiffness of the girders. An increase in the crossframe stiffness results in better load distribution across the width of the structure and less distortion. An increase in the stiffness of the girders reduces the magnitude of vertical deflection resulting in less distortion of the transverse section.

302.2.7.2.b OIL-CANNING

Distortion due to oil-canning occurs when large lateral loads from the cantilevered deck slab falsework bracket deform the girder web.



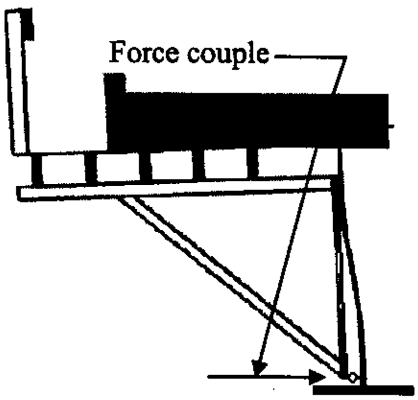
Locating the falsework bracket near the bottom flange will reduce the amount of web deformation. C&MS Item 508 requires the lower point of contact to be within 8" of the top of the bottom flange. Given this requirement and the geometric capabilities of the falsework brackets, the magnitude of girder twist (• ɵ) resulting from oil-canning may be neglected for girder webs 84" deep or less.

For web depths greater than 84", designers shall provide the location of the falsework bracket in the plans. Provide a General Note that removes the lower point of contact requirement of C&MS Item 508 (see BDM Section 600 for an example). The pay item for deck concrete shall be "as per plan". Using the plan bracket location, designers shall determine • ɵ. Designers may assume the lowest location of the falsework bracket to be 76" measured below the bottom of the top flange. The magnitude of twist can be predicted using finite element analysis of the web or by various approximate methods. If the magnitude results in excessive deck thickness loss, reducing the transverse stiffener spacing or adding temporary bracing on the inside of the web may be necessary. Any temporary bracing should be detailed in the plans.

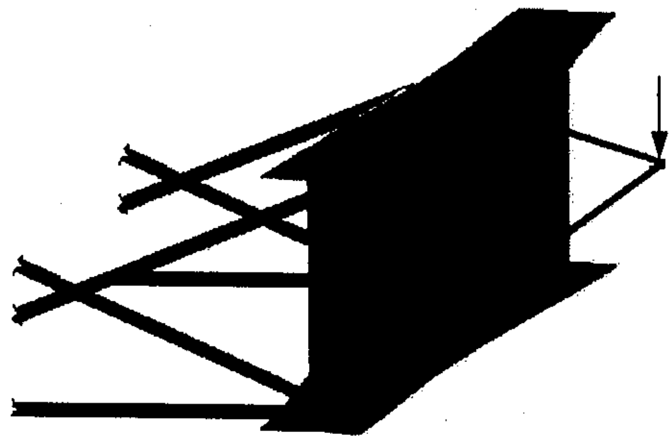
The magnitude of girder twist resulting from oil-canning may be neglected for prestressed I-beam superstructures.

302.2.7.2.c GIRDER WARPING

Distortion due to girder warping occurs as a result of deck slab overhang falsework loading on the fascia girder between points of lateral bracing (e.g. crossframes). The bracket loads produce twist between the crossframes due to a combination of girder warping and pure torsional distortion. The girder is restrained from warping at the crossframe locations. Due to the inherent torsional stiffness of prestressed I-beams, the distortion due to girder warping may be neglected. Other design considerations for I-beams due to the overhang bracket loadings are presented at the end of this section.



Girder warping



Torsional distortion

For steel superstructures, Designers should calculate the magnitude of twist (• ᵴ) due girder warping using the TAEG software developed by the Kansas Department of Transportation. TAEG ("Torsional Analysis of Exterior Girders") is available at no cost and can be downloaded at: http://www.ksdot.org/kart/.

Since most of the data input in TAEG is dependent upon the contractor's equipment and falsework design, designers should use conservative assumptions to accommodate most contractor resources. For design-build projects and value engineering change proposals (VECP's), data input for TAEG shall represent the actual falsework and equipment to be used by the contractor. Designers may use the following assumptions in lieu of actual contractor supplied information:

A. Girder Data:

   For bridges with constant web depths, designers may select the cross section with the least torsional resistance to represent the entire structure. For bridges with variable depth webs, designers may disregard the effect of girder warping in the web depth transition sections.

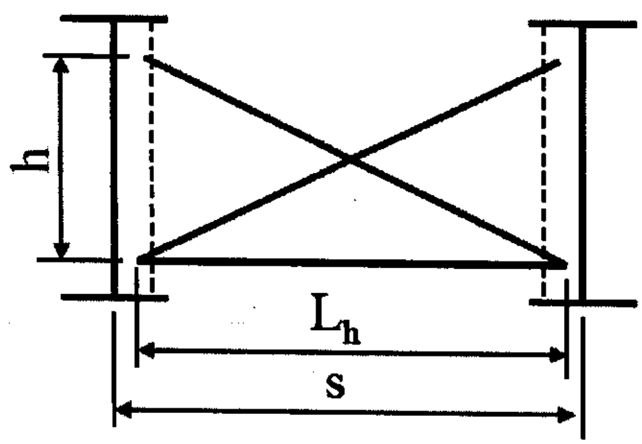
B. Bridge Lateral Data:

   Designers may select the largest crossframe spacing to represent the entire structure. For structures with variable beam spacings (i.e. flared girders) designers may select the largest spacing dimension to represent the entire structure. Designers should generally avoid utilizing temporary tie rods and timber blocks; however, if required these should be detailed in the contract plans.

C. Permanent Lateral Support Data:

The default crossframe type assumed by the TAEG software consists of a stiffener and diagonal x-bracing with top and bottom horizontal chords. In order to analyze the structure with a standard ODOT crossframe, designers should input stiffener dimensions and select the "Diaphragms (Inputted Ix)" option. For ODOT Type 1 crossframes, designers should assume a fictitious stiffener of dimensions: 5" x 3/8". Determine the diaphragm moment of inertia for all standard ODOT crossframes as follows:

$$I_x = \frac{h^2 s}{4L_d^3\left(\dfrac{1}{A_d L_h^2} + \dfrac{L_h}{A_h L_d^3 + A_d L_h^3}\right)}$$



Where:

A_d = Area of the diagonal member (in^2)

A_h = Area of the horizontal member (in^2)

$L_d = \sqrt{L_h^2 + h^2}$

D. Temporary Lateral Support Data:

Designers should generally avoid utilizing temporary tie rods and timber blocks; however, if required these should be detailed in the contract plans.

E. Load Data:

1. Live Load on Walkway........50 lb/ft^2
2. Live Load on Slab........50 lb/ft^2
3. Dead Load of Formwork........10 lb/ft^2
4. Dead Load of Concrete........150(t_{avg}) lb/ft^2

   (t_{avg} = Average thickness [ft.] of deck slab overhang)
5. Wheel Spacing [1-2-3]........36" – 31" – 36"
6. Maximum Wheel Load:

   To estimate the total finishing machine length required for placement along the skew, add the rail-to-rail length and the extra end length from the following table using the plan specified skew rounded to the nearest 5 degrees. W is the rail-to-rail length as measured perpendicular to the centerline of the bridge.

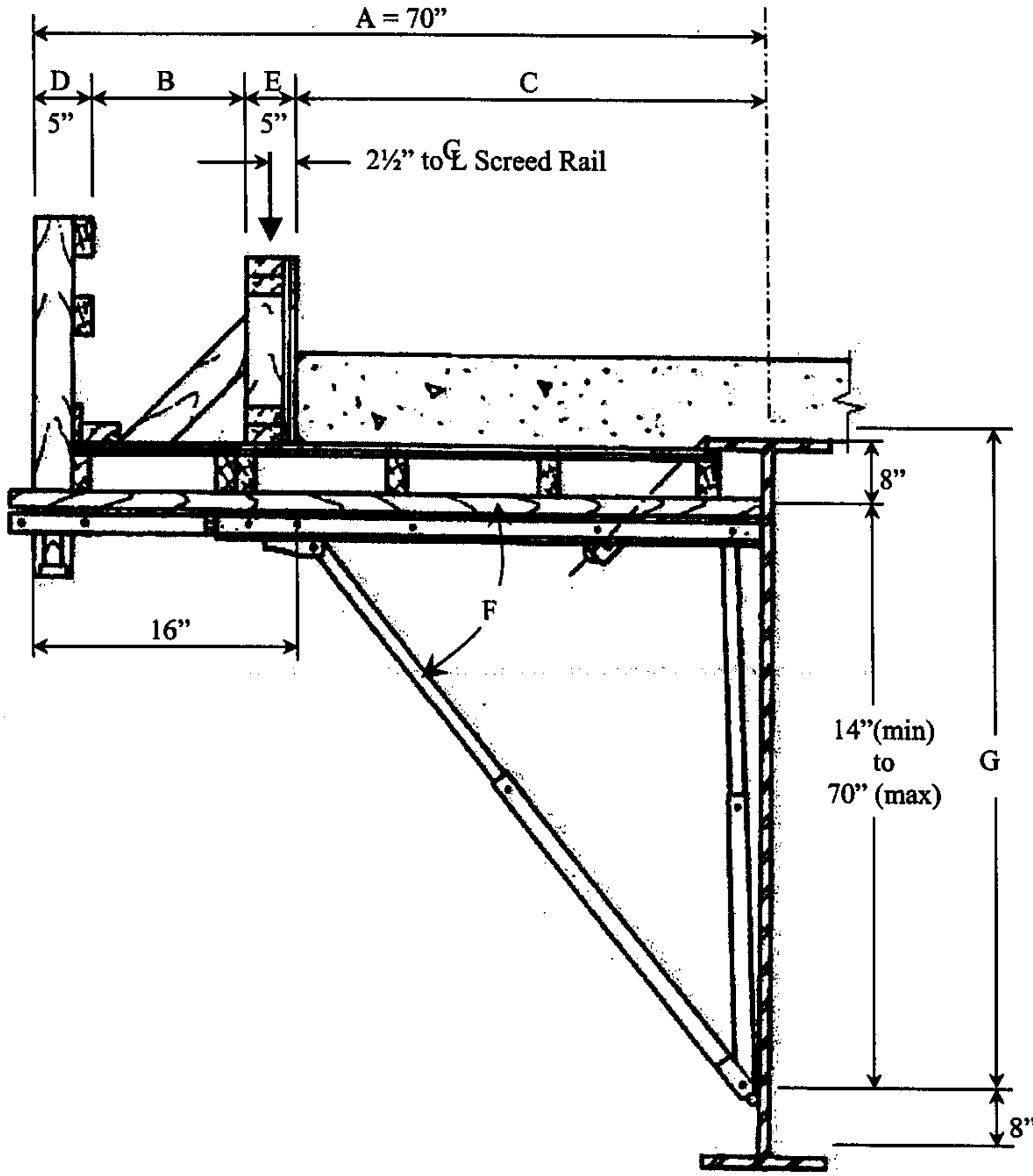
Skew Angle	Rail-to-Rail Length, ft.	Extra End Length, ft.
0	1.00 W	0.0
15	1.04 W	5.0
20	1.06 W	5.5
25	1.10 W	6.5
30	1.15 W	7.0
35	1.22 W	8.0
40	1.31 W	9.0
45	1.41 W	10.5
50	1.56 W	11.5
55	1.74 W	13.5

For total machine lengths of 36 ft. and less, assume a total machine weight of 7.6 kip. Add 0.09 kip for each additional foot of machine length required above 36 ft. The maximum total machine length shall not exceed 120 ft. If greater lengths are required, consult the Office of Structural Engineering for recommendations.

To determine the maximum wheel load, divide the total machine weight by 8.0.

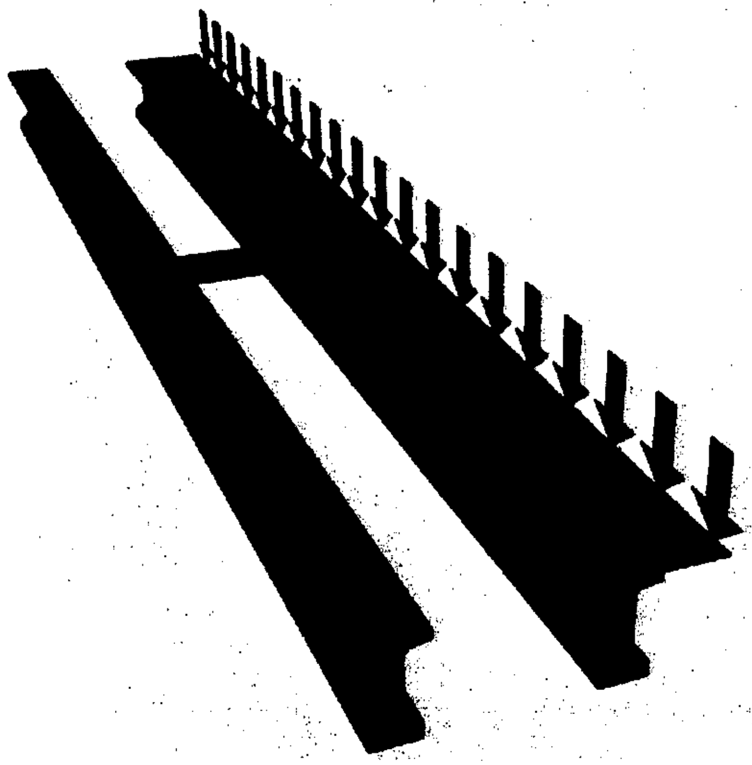
F. Bracket Data:

1. Refer to the following figure to determine TAEG dimensions A, B, C, D, E, F and G.
2. Designers may assume a center-to-center bracket spacing of 48.0 in.
3. Designers may assume a bracket weight of 50 lbs.



Assumptions for TAEG Bracket Data Input

For prestressed I-beam superstructures, Designers should verify that the intermediate crossframes/diaphragms in the exterior bay are capable of resisting the torsion caused by the cantilevered falsework.



302.2.7.3 DETERMINING EFFECT OF GIRDER TWIST

Once all sources of girder twist are quantified, Designers should determine the total effect that girder twist has on the finished deck surface. The primary effect of greatest concern is the loss of concrete cover over the top mat of deck reinforcing steel and the subsequent loss of deck thickness. The maximum loss due to twisting shall not exceed 0.5 in.

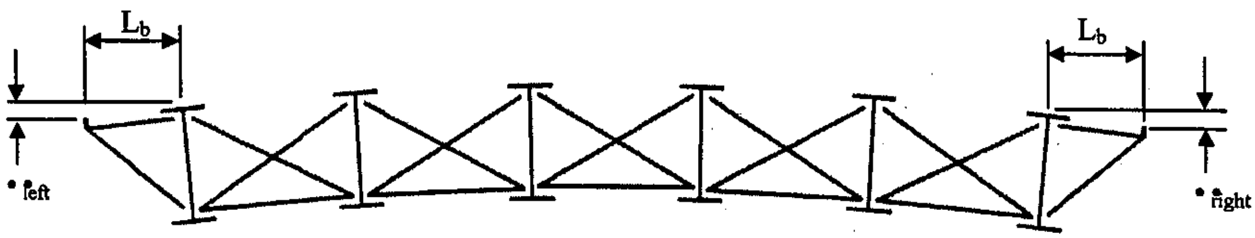
The total amount of girder twist at both the left and right screed rail should be determined as follows:

$\phi_{left} = \left(\phi_g + \phi_o + \phi_w\right)_{left}$ and $\phi_{right} = \left(\phi_g + \phi_o + \phi_w\right)_{right}$

where:

- $_g$ = Girder twist due to global superstructure distortion (See BDM Section 302.2.7.2.a)
- $_o$ = Girder twist due to "oil-canning" (See BDM Section 302.2.7.2.b)
- $_w$ = Girder twist due to girder warping (See BDM Section 302.2.7.2.c)

The total amount of screed rail deflection at both the left and right screed rail should be determined as follows:



$\delta_{left} = \tan(\phi_{left}) \times L_b$ and $\delta_{right} = \tan(\phi_{right}) \times L_b$

where:

δ_{left} δ_{right} = Deflection of the screed rail due to total girder twist (in.). Upward deflection is positive and downward deflection is negative.

L_b = Lateral distance from center of screed rail to centerline of fascia girder (in.)

The total loss of deck thickness should be determined as follows:

$\delta_{total} = (\delta_{left} + \delta_{right})/2$

302.2.8 SLAB DEPTH OF CURVED BRIDGES

For a curved deck on straight steel beams, steel girders or prestressed I-beams, the distance from the top of the slab to the top of the beams or girders will vary from end to end. The slab depth dimension shall show this variation by giving the maximum and minimum depth dimensions with their respective location, over the piers, center of span, etc.

An alternate is to accommodate the differential depth by including it in the Camber Table as geometric camber.

302.2.9 STAGED CONSTRUCTION

For all bridge types, except non-composite concrete box beams, where the differential dead load deflection between adjacent beams, girders or structural slabs is greater than ¼ inch [6 mm], a deck closure is required if the bridge is constructed in stages.

For requirements regarding closure pours on bridge widenings or on existing structures with new concrete decks see Section 400 of this Manual.

The closure pour between the stages shall be a minimum width of 30 inches [800 mm] but should be wide enough to accommodate the required reinforcing steel lap splices. In special cases, this distance may be reduced when mechanical reinforcing steel connectors are used (see

C. BEAM DESIGN

Company:	KZFDESIGN				
Structure :	SCI-TR234-0122	Design :	DAT	Date :	7/17/2009
Subject:	camber	Checked :		Date :	

compute beam camber (per ODOT BDM 302.5.1.5)
vertical curve data:
structure is not located within a vertical curver
therefore the vertical curve ordinate is 0

grade = -0.01
PVI Elev. = 662.8654
PVI Station = 36+00.000

sample calculation:
compute PGL elev. At the centerline bearing rear abutment
x = distance from PVI station to point
x = 70.50000
PGL Elev. = (BVC Elev.) + grade*(x)
PGL Elev. = 662.8654 + (-0.01*70.50)
PGL Elev. = 662.160

span no.	centerline bearing substructure location	station	X (ft.)	PGL Elev.
1	Rear Abut.	36+70.500	70.50000	662.160
1	Fwd. Abut.	37+90.000	190.00000	660.966

compute beam camber (per ODOT BDM 302.5.1.5), units in inches

[SPAN 1, BEAM 1]		
A =	8.5	design slab thickness
B =	4.021	camber due to prestress at release at mid-span
C =	1.761	mid-span deflection due to beam self weight
Dslab =	0.993	mid-span deflection due to deck slab + haunch
Ddiaphragm =	0.019	mid-span deflection due to diaphragms
Dsacrificial =	0.115	mid-span deflection due to sacrificial wearing surface (0 if included in slab deflection)
Dtotal =	1.127	total mid-span deflection
E =	0.009	mid-span deflection due to parapets
F =	0.000	vertical curve ordinate (negative for sag vertical curve)
G =	2	haunch thickness
		H = A+1.8B-1.85C-D-E-F+G
		H = A+G (if F>1.8B-1.85C-(D+E)
		check F<>1.8B-1.85C-(D+E)
		1.8B-1.85C-(D+E) = 2.844339
H =	13.34	total topping thickness at beam bearings
		total topping thickness at mid-span
		I = A + G
		I = A-(1.8B-1.85C)+D+E+F+G
		check F<>1.8B-1.85C-(D+E)
		1.8B-1.85C-(D+E) = 2.844339
I =	10.50	total topping thickness at mid-span
(B-C) =	2.260	camber at release
1.8B-1.85C =	3.980	camber at erection
2.45B-2.4C =	5.625	long term camber
[SPAN 1, BEAM 2]		
A =	8.5	design slab thickness
B =	4.021	camber due to prestress at release at mid-span
C =	1.761	mid-span deflection due to beam self weight
Dslab =	1.231	mid-span deflection due to deck slab + haunch
Ddiaphragm =	0.024	mid-span deflection due to diaphragms
Dsacrificial =	0.142	mid-span deflection due to sacrificial wearing surface (0 if included in slab deflection)
Dtotal =	1.397	total mid-span deflection
E =	0.09	mid-span deflection due to parapets
F =	0.000	vertical curve ordinate (negative for sag vertical curve)
G =	2	haunch thickness
		H = A+1.8B-1.85C-D-E-F+G
		H = A+G (if F>1.8B-1.85C-(D+E)
		check F<>1.8B-1.85C-(D+E)
		1.8B-1.85C-(D+E) = 2.844339
H =	12.99	total topping thickness at beam bearings
		total topping thickness at mid-span
		I = A + G
		I = A-(1.8B-1.85C)+D+E+F+G
		check F<>1.8B-1.85C-(D+E)
		1.8B-1.85C-(D+E) = 2.844339
I =	10.50	total topping thickness at mid-span
(B-C) =	2.260	camber at release
1.8B-1.85C =	3.980	camber at erection
2.45B-2.4C =	5.625	long term camber

Company:	KZFDESIGN				
Structure :	SCI-TR234-0122	Design :	DAT	Date :	7/13/2009
Subject:	parabolic dead load deflections	Checked :		Date :	

Beam Length (brg. to brg.) = 119.5 ft
Total Deflection (in.) = 1.127 in. = 0.093917 ft

parabolic equation (x)(x)=-2(p)(y)
p = (x)(x)/-2(y) = 19006.54

Screed Point Locations
Format A
End Spaces = 14.9375 ft
No. of Equal Spaces = 8
Equal Space Distance = 10.0 ft
Total Spa. 10

SPAN 1, BEAM 1

Point No.	Location on Beam	x	y	Deflection (ft)	Deflection (in)
1	0	-59.75	0.0939	0.0000	0.0000
2	14.9375	-44.81	0.0528	0.0411	0.4931
3	29.875	-29.88	0.0235	0.0704	0.8453
4	44.8125	-14.94	0.0059	0.0880	1.0566
5	59.75	0.00	0.0000	0.0939	1.1270
6	74.6875	14.94	0.0059	0.0880	1.0566
7	89.625	29.88	0.0235	0.0704	0.8453
8	104.5625	44.81	0.0528	0.0411	0.4931
9	119.5	59.75	0.0939	0.0000	0.0000

Company:	KZFDESIGN				
Structure :	SCI-823-0837 (left/right bridge)	Design :	DAT	Date :	7/13/2009
Subject:	parabolic dead load deflections	Checked :		Date :	

Beam Length (brg. to brg.) = 119.5 ft
Total Deflection (in.) = 1.397 in. = 0.116417 ft

parabolic equation (x)(x)=-2(p)(y)
p = (x)(x)/-2(y) = 15333.12

Screed Point Locations
Format A
End Spaces = 14.9375 ft
No. of Equal Spaces = 8
Equal Space Distance = 10.0 ft
Total Spa. 10

SPAN 1, BEAM 2

Point No.	Location on Beam	x	y	Deflection (ft)	Deflection (in)
1	0	-59.75	0.1164	0.0000	0.0000
2	14.9375	-44.81	0.0655	0.0509	0.6112
3	29.875	-29.88	0.0291	0.0873	1.0478
4	44.8125	-14.94	0.0073	0.1091	1.3097
5	59.75	0.00	0.0000	0.1164	1.3970
6	74.6875	14.94	0.0073	0.1091	1.3097
7	89.625	29.88	0.0291	0.0873	1.0478
8	104.5625	44.81	0.0655	0.0509	0.6112
9	119.5	59.75	0.1164	0.0000	0.0000

PROJECT DATA

Project:	SCI-823: Shumway Hollow Rd over CSXT RR
Designer:	def
Date:	Oct/27/2008
User job number:	5355
State:	OH, State Job #:
State Specification:	None
Design Mode:	AASHTO Standard (LFD)- US Units [17th Edition, 2003]
Flared Girder:	No
Comments:	121' Single Span 72" Mod. Type-4 Bm Span/ 7'-9" Centers - Standard Spec. HS25 Loading
File Name:	C:\PROJECTS-KZF-DESIGN\5355-SCI-823-Portsmouth-ByPass\SCI-823-Shumway-CSX-7.75-ft-Spacing.csl

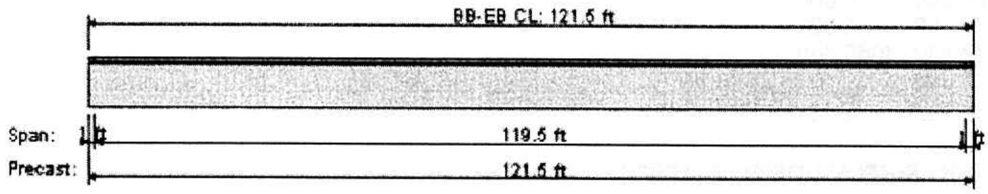
GEOMETRY DATA

BRIDGE LAYOUT

Overall Width (ft)	59.000
Left curb (ft)	1.500
Right curb (ft)	1.500
curb-to-curb width (ft)	56.000
Number of spans	1
Number of lanes	4
Lane width (ft)	12.000
Topping thickness (in)	7.500
Haunch thickness (in)	3.000
Haunch width (in)	36.000
Bridge c/s,MI(Ixx) (in4)	10677177.00

SPAN DATA

Precast length,	ft =	121.500
Bearing-to-bearing,	ft =	119.500
Release span,	ft =	121.500

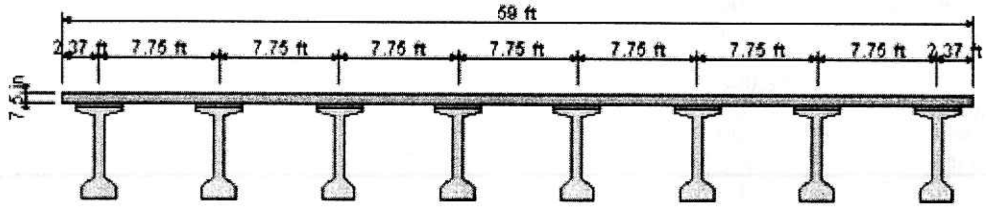


Bridge elevation section for all spans

BEAM DATA

No	ID	Loc-prev ft	Area in2	MI(Ixx) in4	Height in	Yb in	B-topg in	B-trib ft
1	OHIO-MOD-72-TYPE-IV	2.375	956.1	616736.0	72.00	34.44	36.00	6.250
2	OHIO-MOD-72-TYPE-IV	7.750	956.1	616736.0	72.00	34.44	36.00	7.750
3	OHIO-MOD-72-TYPE-IV	7.750	956.1	616736.0	72.00	34.44	36.00	7.750
4	OHIO-MOD-72-TYPE-IV	7.750	956.1	616736.0	72.00	34.44	36.00	7.750
5	OHIO-MOD-72-TYPE-IV	7.750	956.1	616736.0	72.00	34.44	36.00	7.750
6	OHIO-MOD-72-TYPE-IV	7.750	956.1	616736.0	72.00	34.44	36.00	7.750

No	ID	Loc-prev ft	Area in2	MI(Ixx) in4	Height in	Yb in	B-topg in	B-trib ft
7	OHIO-MOD-72-TYPE-IV	7.750	956.1	616736.0	72.00	34.44	36.00	7.750
8	OHIO-MOD-72-TYPE-IV	7.750	956.1	616736.0	72.00	34.44	36.00	6.250



Bridge cross section

MATERIAL DATA

CONCRETE PROPERTIES

	Precast	C.I.P
f'c (ksi)	7000.000	4500.000
Wc (pcf)	150.000	150.000
Ec (ksi)	5072.240	4066.840
f'ci (psi)	5500.000	
Eci (ksi)	4496.060	

STRAND AND REBAR PROPERTIES

PRESTRESSED STEEL:

6/10-270K-LL, Low relaxation strands
Depressed at 0.35L (42.52 ft from member end)
Strand Diameter = 0.600
Ult. Strength(f's) = 270.0 ksi
Strand Area = 0.217 in2
Use transformed strand and rebar: No

REINFORCING STEEL:

Tension steel: fy = 60.0 ksi Es = 29000 ksi fs = 24.0 ksi

LOADS DATA

LOADS ON PRECAST

UNITS: (Point: kips, Location: ft, Line: klf)

Span	Beam	DL/ADL	Type	Mag.	Loc.	Description
1	1	DL	Line	0.078	-	Sacrificial Wearing Surface
1	2	DL	Line	0.097 √	-	Sacrificial Wearing Surface
1	3	DL	Line	0.097	-	Sacrificial Wearing Surface
1	4	DL	Line	0.097	-	Sacrificial Wearing Surface
1	5	DL	Line	0.097	-	Sacrificial Wearing Surface
1	6	DL	Line	0.097	-	Sacrificial Wearing Surface
1	7	DL	Line	0.097	-	Sacrificial Wearing Surface
1	8	DL	Line	0.078	-	Sacrificial Wearing Surface

DIAPHRAGM LOADS

(kips, ft)

Span	Beam	Mag.	Loc.
1	1	0.210	22.500
1	1	0.210	47.500
1	1	0.210	72.500
1	1	0.210	97.500
1	2	0.420	22.500
1	2	0.420	47.500
1	2	0.420	72.500
1	2	0.420	97.500
1	3	0.420	22.500
1	3	0.420	47.500
1	3	0.420	72.500
1	3	0.420	97.500
1	4	0.420	22.500
1	4	0.420	47.500
1	4	0.420	72.500
1	4	0.420	97.500
1	5	0.420	22.500
1	5	0.420	47.500
1	5	0.420	72.500
1	5	0.420	97.500
1	6	0.420	22.500
1	6	0.420	47.500
1	6	0.420	72.500
1	6	0.420	97.500
1	7	0.420	22.500
1	7	0.420	47.500
1	7	0.420	72.500
1	7	0.420	97.500
1	8	0.210	22.500
1	8	0.210	47.500
1	8	0.210	72.500
1	8	0.210	97.500

LOADS ON COMPOSITE

UNITS: (Point: kips, Location: ft, Line: klf, Area: ksf, Width: ft)

Span	DL/ADL	Type	Mag.	Loc.	Description
1	DL	Line	0.530	-	Left Barrier Weight
1	DL	Line	0.530	-	Right Barrier Weight
1	ADL	Area	0.060	56.000	Future Wearing Surface

LIVE LOADS

Live load deflection: not included.

ID: H/HS25 Lane	(Type: Lane Load)
ID: HS25 Truck	(Type: Truck Load)
ID: Military Truck	(Type: Truck Load)

LIVE LOADS USED

LIVE LOAD LIBRARY: Default.cs4

1 ID: H/HS25 Lane

Description: H25/HS25 as in AASHTO-STANDARD
Type: Lane Load

Lane Load: Intensity = 0.80 klf, Width = 10.00 ft
Conc. Loads: Moment = 22.50 k, Shear = 32.50 k

2 ID: HS25 Truck

Description: HS25 Truck as in AASHTO-STANDARD
Type: Truck Load

Uniform Load	Intensity, klf	Location, ft	Length, ft
Preceding Load	0.00	0.00	0.00
Trailing Load	0.00	0.00	0.00

First Axle Magnitude = 10.00 k, Wheel Spacing = 6.00 ft, Truck Width = 10.00 ft

#	Magnitude, k	Max Spacing, ft	Min Spacing, ft	Increment, ft
1	40.00	14.00	14.00	0.00
2	40.00	30.00	14.00	2.00

3 ID: Military Truck

Description: Military Truck as in AASHTO-STANDARD
Type: Truck Load

Uniform Load	Intensity, klf	Location, ft	Length, ft
Preceding Load	0.00	0.00	0.00
Trailing Load	0.00	0.00	0.00

First Axle Magnitude = 24.00 k, Wheel Spacing = 6.00 ft, Truck Width = 10.00 ft

#	Magnitude, k	Max Spacing, ft	Min Spacing, ft	Increment, ft
1	24.00	4.00	4.00	0.00

ANALYSIS DATA

ANALYSIS PARAMETERS DATA

Beam#	Moment impact	Shear impact
1	1.204	Calculated (AASHTO 3.8.2.2)
2	1.204	Calculated (AASHTO 3.8.2.2)
3	1.204	Calculated (AASHTO 3.8.2.2)
4	1.204	Calculated (AASHTO 3.8.2.2)
5	1.204	Calculated (AASHTO 3.8.2.2)
6	1.204	Calculated (AASHTO 3.8.2.2)
7	1.204	Calculated (AASHTO 3.8.2.2)
8	1.204	Calculated (AASHTO 3.8.2.2)

NOTE: Beam specific dead and live load DFs are printed in beam level reports.

GAMMA/BETA FACTORS: (Table 3.22.1A)

	Service		Factored	
Gamma:	1.00		1.30	
Beta-D:	1.00		1.00	
Beta-L:	1.00	(Group 1)	1.67	(Group 1)

PROJECT PARAMETERS

MULTIPLIERS:

Trans len mult:	Bonded	=	1.00
	Debonded	=	1.00
Dev len mult:	Bonded	=	1.60
	Debonded	=	2.00

Camber & Deflection Multiplier (PCI ref.)

	Erection	Final
Prestress:	1.80	2.20
Self. Wt:	1.85	2.40
Deck + Haunch:		2.30
Diaphragm:		3.00
Prec.DL+ADL:		3.00
Comp.DL+ADL:		3.00

MOMENT AND SHEAR PROVISIONS:

Ultimate Moment Capacity, Mu-prvd computed:	Strain Compatibility method.
Horizontal Shear, Beam and Slab effects in Vu:	INCLUDED
Negative Moment Design, Non-composite Moment effects in Mu:	INCLUDED

STRESS LIMITS (Art. 9.15.2):

STRESS LIMITS AT FINAL 1 (P/S + DL + LL) (Art. 9.15.2.2 a):

	PRECAST		DECK	
Strength	7000.00	psi	4500.00	psi
Max Comp, Top	4200.00	psi	2700.00	psi
Pos Mom, Bot	4200.00	psi		
Neg Mom, Bot	4200.00	psi		
Max Tens, Top	-502.00	psi	-503.12	psi
Max Tens, Bot	-502.00	psi		
Crk Tens, Bot	-627.50	psi		
Elasticity	5072.2	ksi	4066.8	ksi

STRESS LIMITS AT FINAL 2 (P/S + DL) (Art. 9.15.2.2 b):

	PRECAST		DECK	
Max Comp, Top	2800.00	psi	1800.00	psi

	PRECAST		**DECK**
Pos Mom, Bot	2800.00	psi	
Neg Mom, Bot	2800.00	psi	

STRESS LIMITS AT FINAL 3 (50% P/S + 50% DL + LL) (Art. 9.15.2.2 c):

	PRECAST		**DECK**	
Max Comp, Top	2800.00	psi	1800.00	psi
Pos Mom, Bot	2800.00	psi		
Neg Mom, Bot	2800.00	psi		

AT RELEASE (Art. 9.15.2.1):

	PRECAST	
Strength	5500.00	psi
Max Comp, Top	5500.00	psi
Max Comp, Bot	3300.00	psi
Max Tens, Top	-200.00	psi
w/reinf	-556.21	psi
Max Tens, Bot	-0.00	psi
Elasticity	4496.1	ksi

RESISTANCE FACTORS (Art. 9.14):

Flexure Reinforced	0.90
Flexure Prestressed	1.00
Shear	0.90

PRESTRESS LOSSES:

Time Dependent Losses, Approximate Method (Art.5.9.5.3)
Hours to release = 18.00
Rel. Humid.(RH) = 75.0 %

PROPERTIES

Span: 1, Beam: 3

PRECAST DATA:

Section Id	OHIO-MOD-72-TYPE-IV						
Type	I-Girder						
Flng width		Top	36.000	in	Bot	26.000	in
thick		Top	4.000	in	Bot	8.000	in
Stems		No	1				
		Top	8.000	in			
		Bot	8.000	in			
Shear width			8.000	in			
Wide top Flange		NO					

GENERAL BRIDGE DATA:

Bridge Width	59.00	ft
Curb-to-curb	56.00	ft
Beam Spac. Lt./Rt	7.75/ 7.75	ft
Lane width	12.00	ft
Number of lanes	4	
Interior/Exterior	Interior	

TOPPING DATA:

Deck	Thickness	7.500	in	
Haunch:				
	Thickness	3.000	in	
	Width	36.000	in	
Effective	width	93.000	in	(Art. 8.10.1)

GENERAL LOAD DATA:

Dead loads on precast:
UNITS: (Point: kips, Location: ft)
(Line: klf)

Type	Mag.	Loc.
Line	0.097	-

Diaphragm loads:
(kips, ft)

Mag.	Loc.
0.42	22.50
0.42	47.50
0.42	72.50
0.42	97.50

Dead loads on composite: See Project info for composite loads

GENERAL SPAN DATA:

Overall length	121.500	ft
Release length	121.500	ft
Design length	119.500	ft

KERN POINTS:

Upper	53.17	in
Lower	17.27	in

DISTRIBUTION FACTORS (Art. 3.23):

Live Moment (Group 1)	0.705	(Calculated)
Live Shear (Group 1)	0.705	(Calculated)

Dead Loads and Pedestrian Load distributed equally to all beams (Art. 3.23.2.3.1.1)

Pedestrian	0.125	(Calculated)
Comp. DL	0.125	(Calculated)
Comp. ADL	0.125	(Calculated)

RESISTANCE FACTORS (Art. 9.14):

Flexure Reinforced	0.90
Flexure Prestressed	1.00
Shear	0.90

Span: 1, Beam: 3

SECTION PROPERTIES:

	PRECAST		COMPOSITE		
Area	956.1	in2	1601.9	in2	#
Total Height	72.00	in	82.50	in	
Mom. of Inertia (Ixx)	616736	in4	1354440	in4	#
Ht. of c.g.	34.44	in	52.02	in	#
Density	150.00	pcf	150.00	pcf	
Self-weight	995.9	plf	1835.0	plf	
Mom. of Inertia (Iyy)	37871.0	in4			
Poisson's Ratio	0.2				

(#) Of Total Section using Ect/Ec = 0.8018

Use transformed strand and rebar: No

Span: 1, Beam: 3
STRESS LIMITS (Art. 9.15.2):

STRESS LIMITS AT FINAL 1 (P/S + DL + LL) (Art. 9.15.2.2 a):

	PRECAST		DECK	
Strength	7000.00	psi	4500.00	psi
Max Comp, Top	4200.00	psi	2700.00	psi
Pos Mom, Bot	4200.00	psi		
Neg Mom, Bot	4200.00	psi		
Max Tens, Top	-502.00	psi	-503.12	psi
Max Tens, Bot	-502.00	psi		
Crk Tens, Bot	-627.50	psi		
Elasticity	5072.2	ksi	4066.8	ksi

STRESS LIMITS AT FINAL 2 (P/S + DL) (Art. 9.15.2.2 b):

	PRECAST		DECK	
Max Comp, Top	2800.00	psi	1800.00	psi
Pos Mom, Bot	2800.00	psi		
Neg Mom, Bot	2800.00	psi		

STRESS LIMITS AT FINAL 3 (50% P/S + 50% DL + LL) (Art. 9.15.2.2 c):

	PRECAST		DECK	
Max Comp, Top	2800.00	psi	1800.00	psi
Pos Mom, Bot	2800.00	psi		
Neg Mom, Bot	2800.00	psi		

AT RELEASE (Art. 9.15.2.1):

	PRECAST	
Strength	5500.00	psi
Max Comp, Top	5500.00	psi
Max Comp, Bot	3300.00	psi
Max Tens, Top	-200.00	psi
w/reinf	-556.21	psi
Max Tens, Bot	-0.00	psi
Elasticity	4496.1	ksi

Span: 1, Beam: 3

PRESTRESSED STEEL:

39 strands, 6/10-270K-LL, Low relaxation strands
Depressed at 0.35L (42.52 ft from member end)

END PATTERN (Ycg = 11.03 in):

11 @ 2.000 in 11 @ 4.000 in 8 @ 6.000 in 4 @ 8.000 in
3 @ 68.000 in 2 @ 69.000 in

MID PATTERN (Ycg = 6.13 in):
(A) Draped:

3 @ 6.000 in

(B) Straight:

11 @ 2.000 in 11 @ 4.000 in 8 @ 6.000 in 4 @ 8.000 in
2 @ 69.000 in

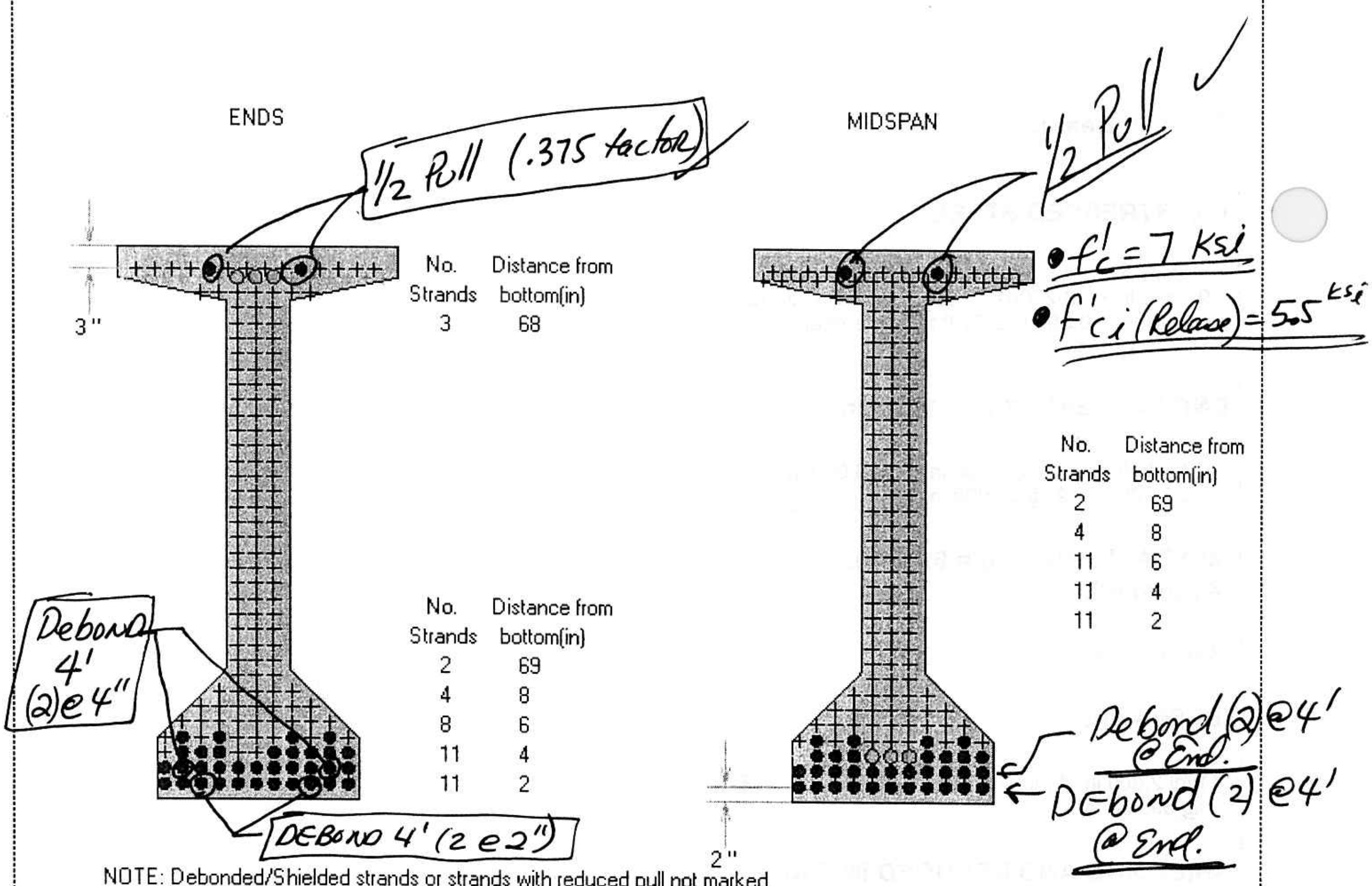
SHIELDING AND REDUCED INITIAL PULLS:

BENTLEY	KZF Design Inc 655 Eden Park Dr Cincinnati OH 45202	Sheet: DS-15 Job No: **5355**
Program: CONSPAN® Rating Version: 8.0.0	 www.leapsoft.com 1-800-451-5327	By: def Date: Oct/27/2008 CKD: Date:

File Name: SCI-823-Shumway-CSX-7.75-ft-Spacing.csl

Group	Strands	Heights End		Mid		Shielding End		Mid		Initial Frac	Pull Pull/Str
1	2	2.000	in	2.000	in	4.00	ft	0.00	ft	75.0 %	43.9
4	2	69.000	in	69.000	in	0.00	ft	0.00	ft	37.5 %	22.0
11	2	4.000	in	4.000	in	4.00	ft	0.00	ft	75.0 %	43.9

Strand Diameter	0.600	in
Strand Area	0.217	in2
Total Strand Area	8.463	in2
Trans. Len,bonded	2.500	ft
Trans. Len,debonded	2.500	ft
Dev. Len, bonded	12.408	ft
Dev. Len, debonded	15.510	ft
Holddown Force	15.900	kips
Holddown Force	270.0	ksi
Initial Prestress = 0.73f's	197.3	ksi
Initial Pull	1669.8	kips
Beam Shrtng (PL/AE)	0.518	in



ENDS:

No. Strands	Distance from bottom(in)
3	68

No. Strands	Distance from bottom(in)
2	69
4	8
8	6
11	4
11	2

MIDSPAN:

No. Strands	Distance from bottom(in)
2	69
4	8
11	6
11	4
11	2

NOTE: Debonded/Shielded strands or strands with reduced pull not marked.

Strand Pattern, Span 1, Beam 3

REINFORCING STEEL:

Tension	**steel:**	
fy	60.0	ksi
Es	29000	ksi
fs	24.0	ksi

Stirrups:

# legs	Size	fy (ksi)	Area (in2)	Spacing (in)	Start (ft)	End (ft)
2	US#4[M13]	60.0	0.40	10.00	0.0000	1.5610
2	US#4[M13]	60.0	0.40	12.00	1.5610	3.3917
2	US#4[M13]	60.0	0.40	18.00	3.3917	6.5577
2	US#4[M13]	60.0	0.40	24.00	6.5577	114.9423
2	US#4[M13]	60.0	0.40	20.00	114.9423	116.8058
2	US#4[M13]	60.0	0.40	16.00	116.8058	119.1221
2	US#4[M13]	60.0	0.40	10.00	119.1221	121.5000

LOSSES

Note: Values are calculated at Midspan

Str. area	8.4630	in2
Ycg	6.13	in
P_init	1669.8	kips
Ecc	28.31	in
Hours to release	18.00	
Rel. Humid.(RH)	75.0	%
Es	28500.0	ksi
Eci	4496	ksi

AASHTO LOSSES

		Release				**Final**	**(Art. 9.16.2)**
Steel relaxation	*	1441.83	psi	CRs (Eq 9-10A)	2183.27	psi	
Elastic Shortening		15276.77	psi	ES (Eq 9-6)	15276.77	psi	(Fcir=2410.01 psi)
Concrete shrinkage		0.00	psi	SH (Eq 9-4)	5750.00	psi	
Concrete creep		0.00	psi	CRc (Eq 9-9)	20031.14	psi	(Fcds=-1269.86 psi)
Total		16718.60	psi	(8.47 %)	43241.17	psi	(21.92 %)

* Steel relax. before release - Ref: PCI Journal Vol. 20, No. 4, Jul-Aug 1975

SHEAR/MOMENT ENVELOPE (&REACTIONS)

SHEAR AND MOMENT ENVELOPE : Span : 1, Beam : 3, SERVICE 1
Shears: kips, Moments: kft

		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
Location,	ft	0.00	1.50	3.44	11.15	23.30	35.45	47.60	59.75
Self wt. :	M	0.0	88.1	198.7	601.6	1116.2	1483.7	1704.3	1777.8
	V	59.5	58.0	56.1	48.4	36.3	24.2	12.1	0.0
Prec. :	M	0.0	8.6	19.3	58.5	108.6	144.3	165.8	172.9
DL+ADL	V	5.8	5.6	5.5	4.7	3.5	2.4	1.2	0.0
Deck :	M	0.0	74.3	167.4	506.8	940.4	1250.0	1435.8	1497.8
+ Haunch	V	50.1	48.9	47.2	40.8	30.6	20.4	10.2	0.0
Diaphragm :	M	0.0	1.3	2.9	9.3	19.2	24.2	29.2	29.2
	V	0.8	0.8	0.8	0.8	0.4	0.4	0.0	0.0
Comp. :	M	-0.0	48.9	110.2	333.7	619.2	823.1	945.4	986.2
DL+ADL	V	33.0	32.2	31.1	26.9	20.1	13.4	6.7	0.0
LL + I :	M+	0.0	103.3	232.4	699.9	1284.7	1683.8	1922.2	1984.7
	V	70.4	69.5	68.2	63.3	56.1	48.9	35.8	28.2
LL + I :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	70.4	69.5	68.2	63.3	56.1	48.9	41.5	34.0
	M	0.0	103.3	232.4	699.9	1284.7	1683.8	1897.4	1925.3
Total :	M+	0.0	324.4	730.9	2209.9	4088.1	5409.2	6202.8	6448.6
	V	219.7	215.1	209.0	184.9	147.1	109.7	66.0	28.2
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	219.7	215.1	209.0	184.9	147.1	109.7	71.7	34.0
	M	0.0	324.4	730.9	2209.9	4088.1	5409.2	6177.9	6389.2

		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
Location,	ft	71.90	84.05	96.20	108.35	116.06	118.00	119.50
Self wt. :	M	1704.3	1483.7	1116.2	601.6	198.7	88.1	0.0
	V	12.1	24.2	36.3	48.4	56.1	58.0	59.5
Prec. :	M	165.8	144.3	108.6	58.5	19.3	8.6	-0.0
DL+ADL	V	1.2	2.4	3.5	4.7	5.5	5.6	5.8
Deck :	M	1435.8	1250.0	940.4	506.8	167.4	74.3	0.0
+ Haunch	V	10.2	20.4	30.6	40.8	47.2	48.9	50.1
Diaphragm :	M	29.1	24.3	19.1	9.4	3.0	1.3	-0.0
	V	0.0	0.4	0.4	0.8	0.8	0.8	0.8
Comp. :	M	945.4	823.1	619.2	333.7	110.2	48.9	0.0
DL+ADL	V	6.7	13.4	20.1	26.9	31.1	32.2	33.0
LL + I :	M+	1922.2	1683.8	1284.7	699.9	232.4	103.3	0.0
	V	35.8	48.9	56.1	63.3	68.2	69.5	70.4
LL + I :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	41.5	48.9	56.1	63.3	68.2	69.5	70.4
	M	1897.4	1683.8	1284.7	699.9	232.4	103.3	0.0
Total :	M+	6202.7	5409.2	4088.1	2210.0	731.0	324.4	0.0
	V	66.0	109.7	147.1	184.9	209.0	215.1	219.7
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	71.7	109.7	147.1	184.9	209.0	215.1	219.7
	M	6177.8	5409.2	4088.1	2210.0	731.0	324.4	0.0

REACTIONS (kips), SERVICE 1

Load Type	Left Support	Right Support
Self Wt.	59.5	59.5
Deck+Haunch	50.1	50.1
Diaphragm	0.8	0.8
Prec.DL+ADL	5.8	5.8
Comp. DL+ADL	264.1	264.1
Live	83.0	83.0
Pedestrian	0.0	0.0

$$R_{BM} = 59.5^k + 50.1^k + 0.8^k + 5.8 + \frac{264.1^k}{8} + (83^k)\underbrace{(1.409)}_{\left(\frac{S}{5.5}\right)}\underbrace{(1.205)}_{\left(\frac{50}{L+125}\right)}\underbrace{(0.5)}_{\text{(Wheel LOAD) (1/2 Lane)}} = \boxed{220.1^k/BM.}$$

1.41 1.21

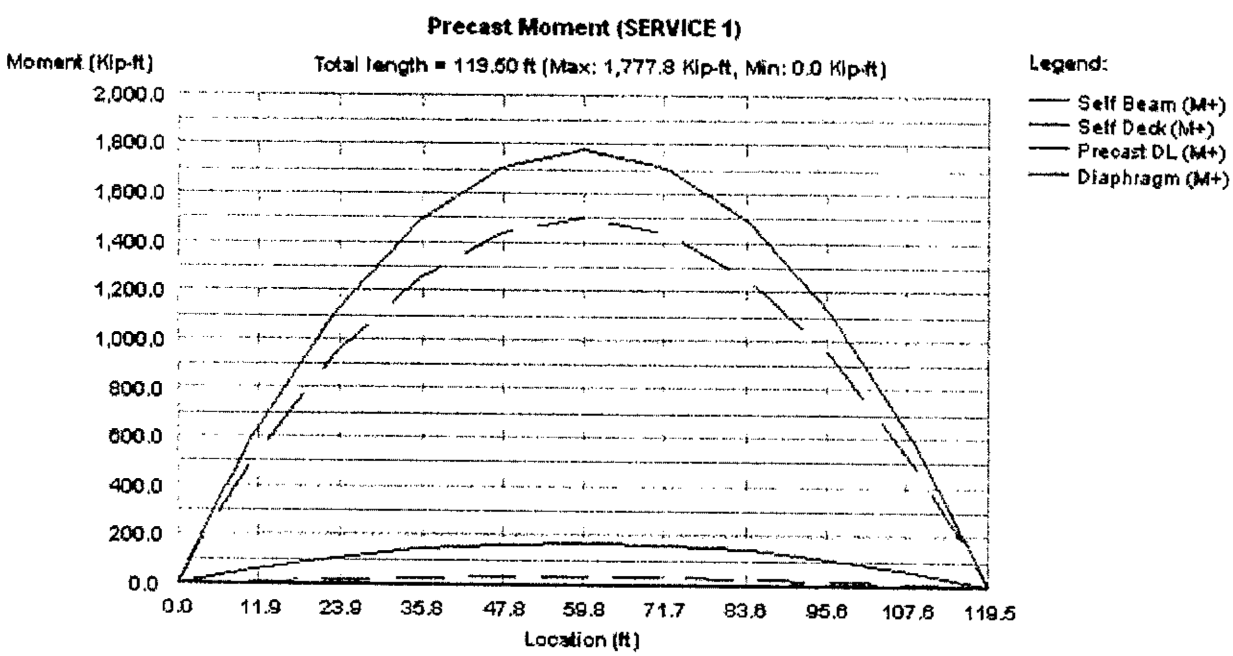
L(119.5')

$\boxed{LL+I = 70.8^k \simeq 71^k}$

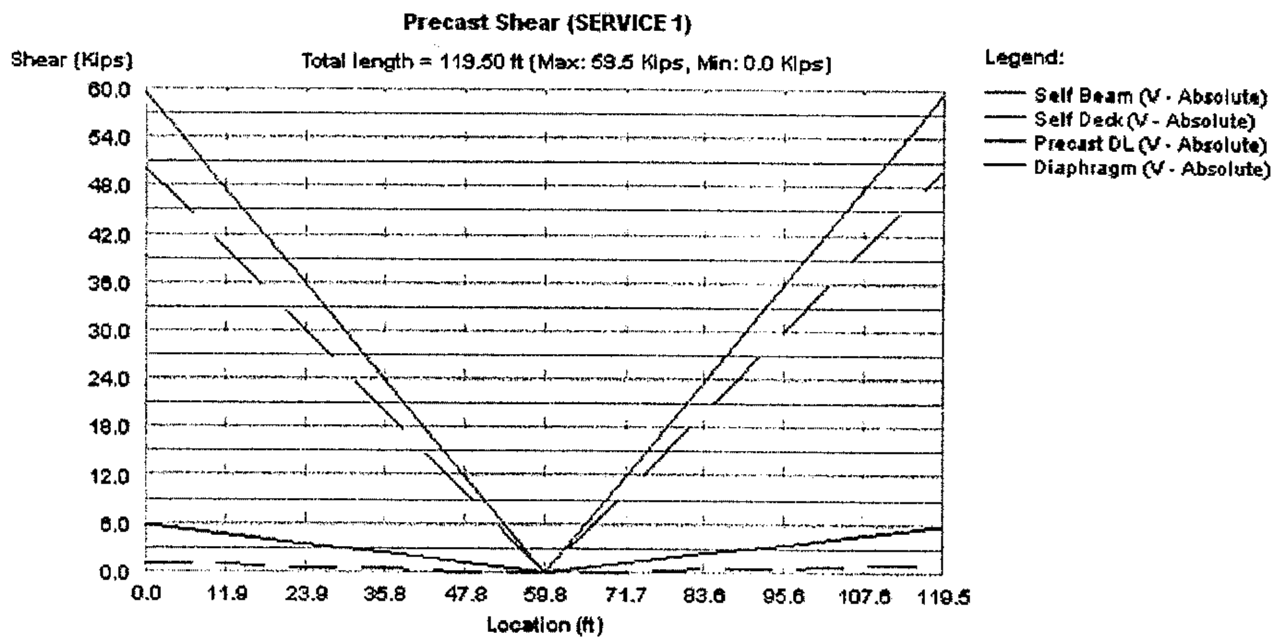
$\boxed{DL+Comp = 149^k}$

$=220^k/BM.$

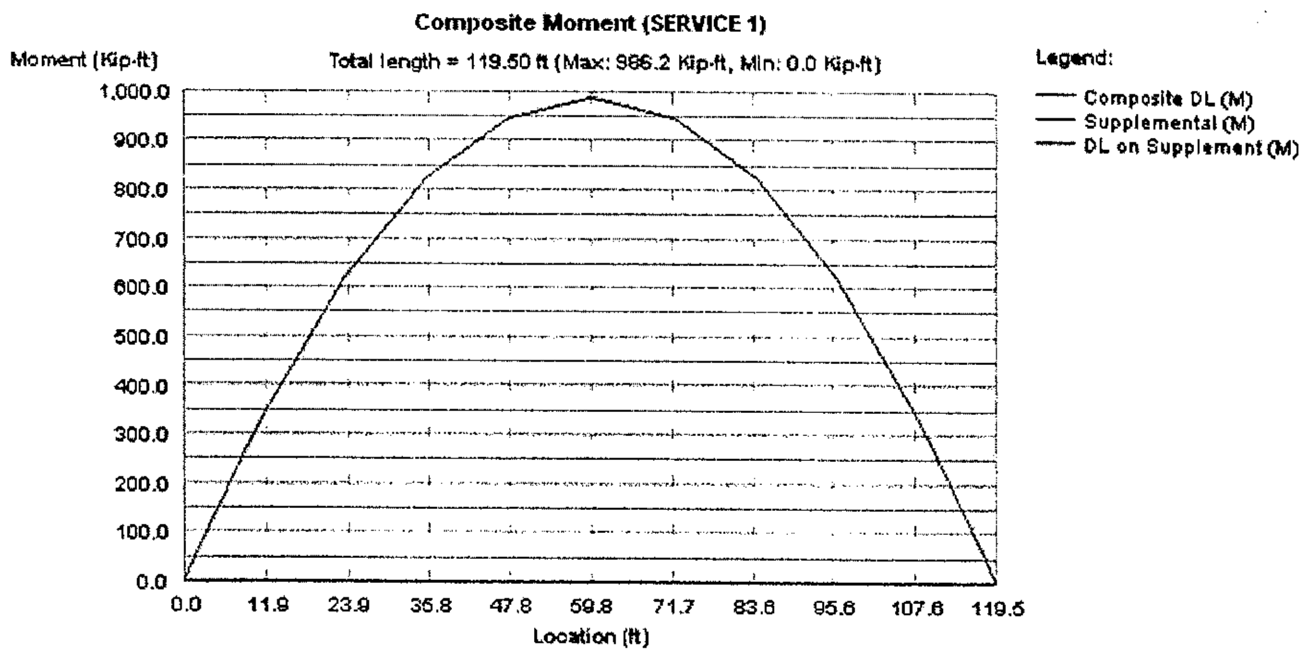
Upward reactions are positive.
Live Load reactions are per lane with no distribution factor and no impact.
Non-composite load types are per beam.
Composite and Pedestrian load types are per total bridge width.



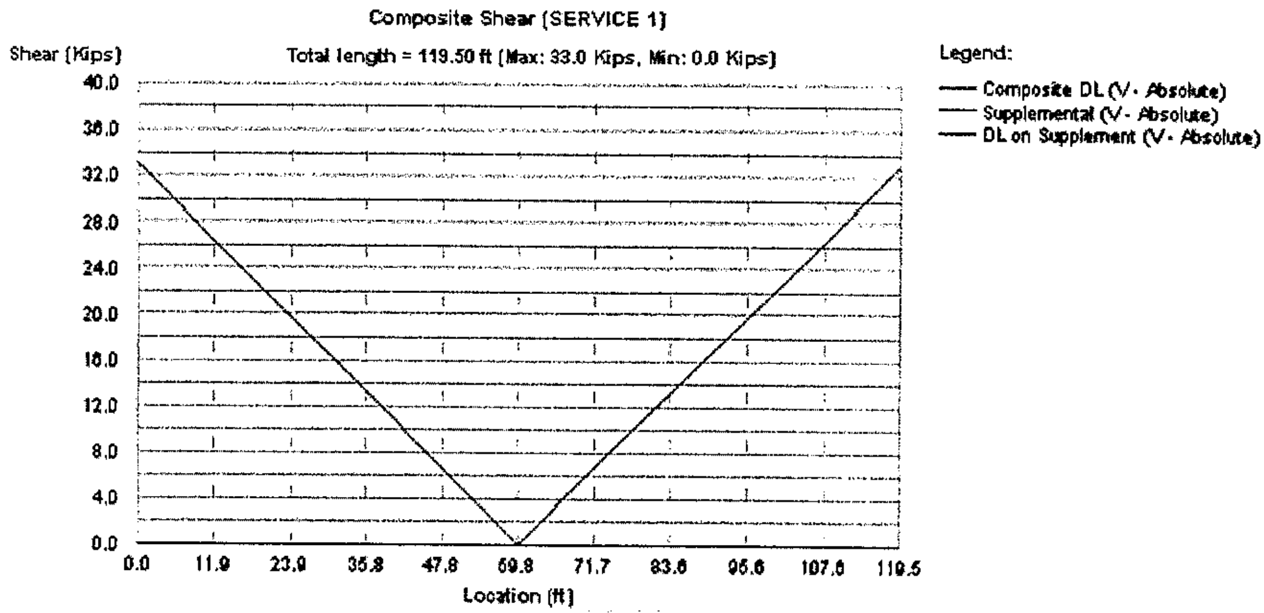
Precast Moment, Span 1, Beam 3, SERVICE 1



Precast Shear, Span 1, Beam 3, SERVICE 1



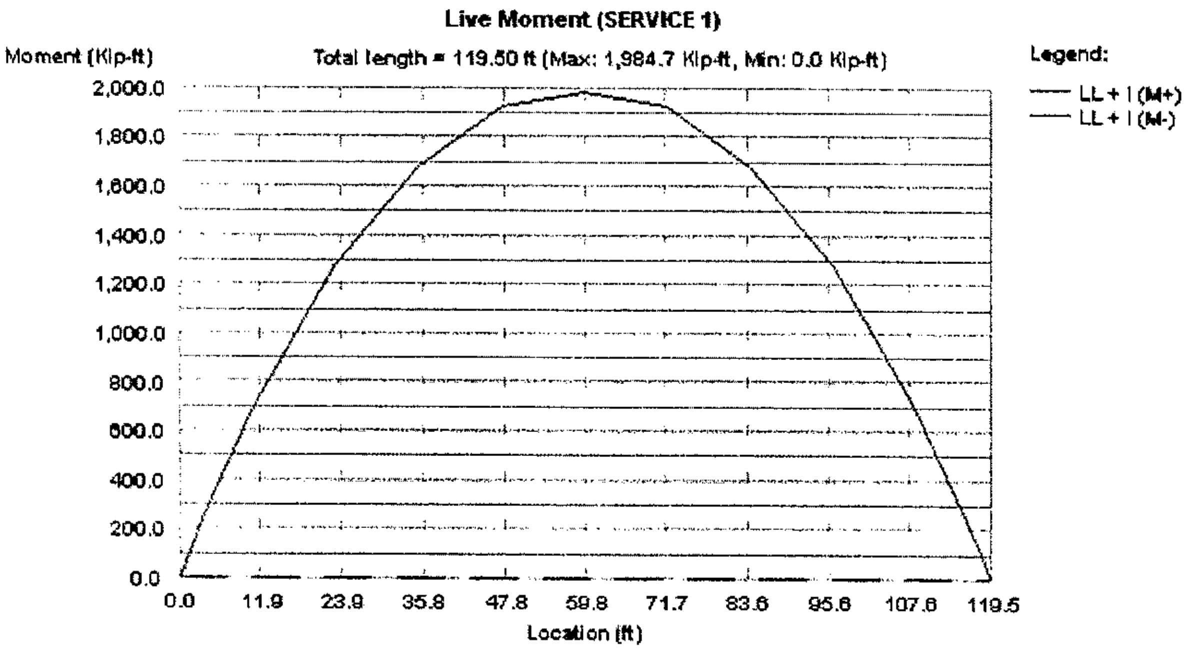
Composite Moment, Span 1, Beam 3, SERVICE 1



Composite Shear, Span 1, Beam 3, SERVICE 1

BENTLEY	KZF Design Inc 655 Eden Park Dr Cincinnati OH 45202	Sheet: DS-22 Job No: **5355**
Program: CONSPAN® Rating Version: 8.0.0	 www.leapsoft.com 1-800-451-5327	By: def Date: Oct/27/2008 CKD: Date:

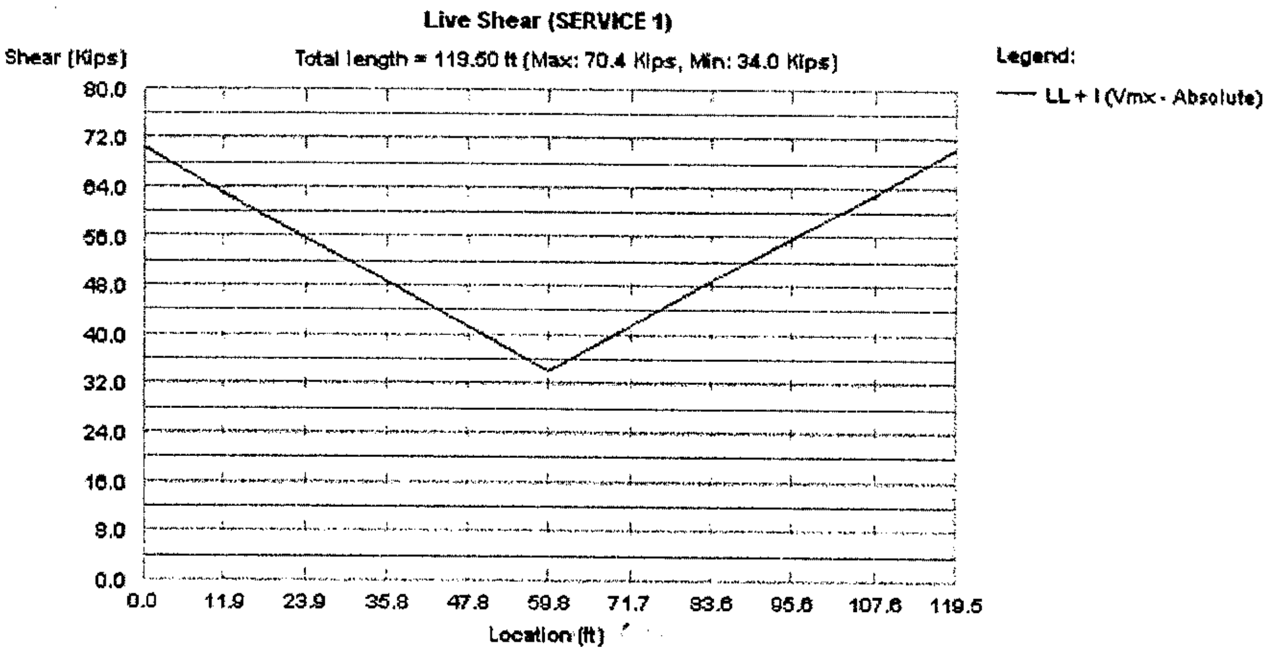
File Name: SCI-823-Shumway-CSX-7.75-ft-Spacing.csl



Live Moment, Span 1, Beam 3, SERVICE 1

Units: U.S. Units

Design Code: AASHTO Standard



Live Shear, Span 1, Beam 3, SERVICE 1

SHEAR AND MOMENT ENVELOPE : Span : 1, Beam : 3, FACTORED 1
Shears: kips, Moments: kft

		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
Location,	ft	0.00	1.50	3.44	11.15	23.30	35.45	47.60	59.75
Self wt. :	M	0.0	114.6	258.3	782.1	1451.0	1928.9	2215.5	2311.1
	V	77.4	75.4	72.9	62.9	47.2	31.5	15.7	0.0
Prec. :	M	0.0	11.1	25.1	76.1	141.1	187.6	215.5	224.8
DL+ADL	V	7.5	7.3	7.1	6.1	4.6	3.1	1.5	0.0
Deck :	M	0.0	96.5	217.6	658.9	1222.5	1625.0	1866.6	1947.1
+ Haunch	V	65.2	63.5	61.4	53.0	39.8	26.5	13.3	0.0
Diaphragm :	M	0.0	1.7	3.8	12.1	24.9	31.5	38.0	37.9
	V	1.1	1.1	1.1	1.1	0.5	0.5	0.0	0.0
Comp. :	M	-0.0	63.6	143.3	433.9	805.0	1070.0	1229.1	1282.1
DL+ADL	V	42.9	41.8	40.4	34.9	26.2	17.5	8.7	0.0
LL + I :	M+	0.0	224.2	504.6	1519.5	2789.0	3655.5	4173.1	4308.8
	V	152.9	150.9	148.1	137.4	121.8	106.1	77.7	61.2
LL + I :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	152.9	150.9	148.1	137.4	121.8	106.1	90.1	73.8

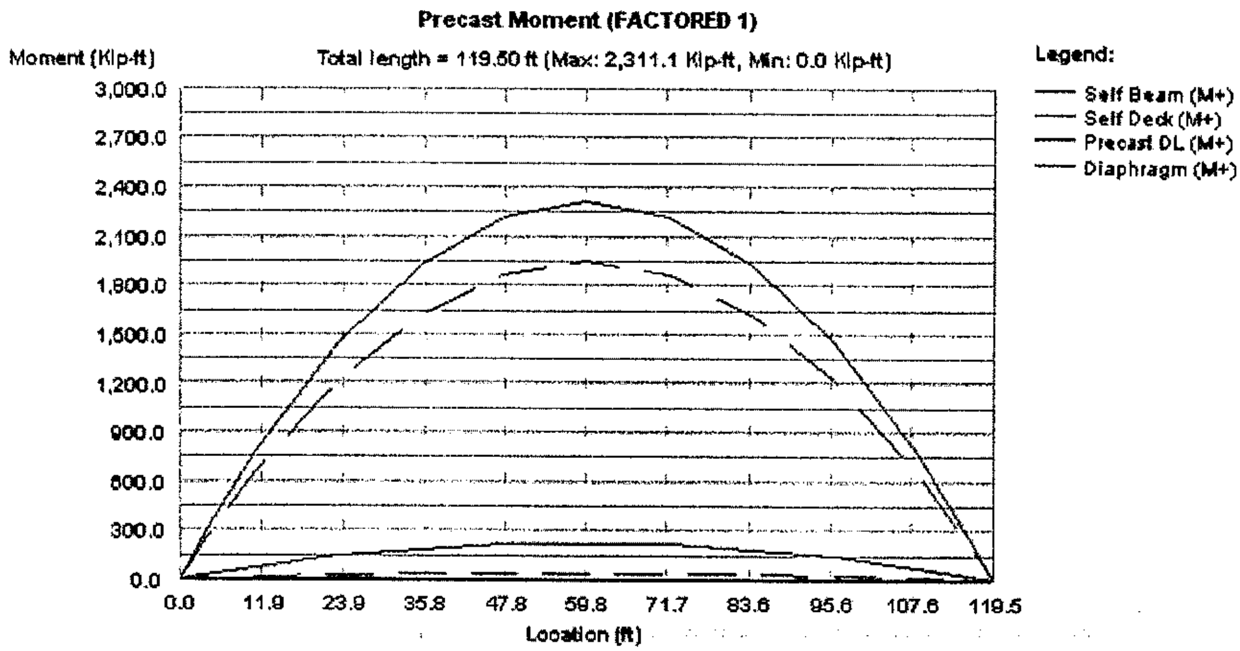
		Bearing	Trans	H/2	0.10L	0.20L	0.30L	0.40L	Midspan
	M	0.0	224.2	504.6	1519.5	2789.0	3655.5	4119.2	4179.9
Total :	M+	0.0	511.7	1152.7	3482.6	6433.5	8498.6	9737.9	10111.9
	V	346.9	340.1	331.1	295.4	240.1	185.1	116.9	61.2
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	346.9	340.1	331.1	295.4	240.1	185.1	129.3	73.8
	M	0.0	511.7	1152.7	3482.6	6433.5	8498.6	9683.9	9982.9

		0.60L	0.70L	0.80L	0.90L	H/2	Trans	Bearing
Location,	ft	71.90	84.05	96.20	108.35	116.06	118.00	119.50
Self wt. :	M	2215.5	1928.9	1451.0	782.1	258.3	114.6	0.0
	V	15.7	31.5	47.2	62.9	72.9	75.4	77.4
Prec. :	M	215.5	187.6	141.1	76.1	25.1	11.1	-0.0
DL+ADL	V	1.5	3.1	4.6	6.1	7.1	7.3	7.5
Deck :	M	1866.6	1625.0	1222.5	658.9	217.6	96.5	0.0
+ Haunch	V	13.3	26.5	39.8	53.0	61.4	63.5	65.2
Diaphragm :	M	37.9	31.5	24.8	12.2	3.8	1.7	-0.0
	V	0.0	0.6	0.6	1.1	1.1	1.1	1.1
Comp. :	M	1229.1	1070.0	805.0	433.9	143.3	63.6	0.0
DL+ADL	V	8.7	17.5	26.2	34.9	40.4	41.8	42.9
LL + I :	M+	4173.1	3655.5	2789.0	1519.5	504.6	224.2	0.0
	V	77.7	106.1	121.8	137.4	148.1	150.9	152.9
LL + I :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
LL + I :	Vmx	90.1	106.1	121.8	137.4	148.1	150.9	152.9
	M	4119.2	3655.5	2789.0	1519.5	504.6	224.2	0.0
Total :	M+	9737.7	8498.6	6433.4	3482.7	1152.7	511.7	0.0
	V	116.9	185.1	240.1	295.4	331.1	340.1	346.9
Total :	M-	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	V	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total :	Vmx	129.3	185.1	240.1	295.4	331.1	340.1	346.9
	M	9683.8	8498.6	6433.4	3482.7	1152.7	511.7	0.0

REACTIONS (kips), FACTORED 1

Load Type	Left Support	Right Support
Self Wt.	77.4	77.4
Deck+Haunch	65.2	65.2
Diaphragm	1.1	1.1
Prec.DL+ADL	7.5	7.5
Comp. DL+ADL	343.3	343.3
Live	180.1	180.1
Pedestrian	0.0	0.0

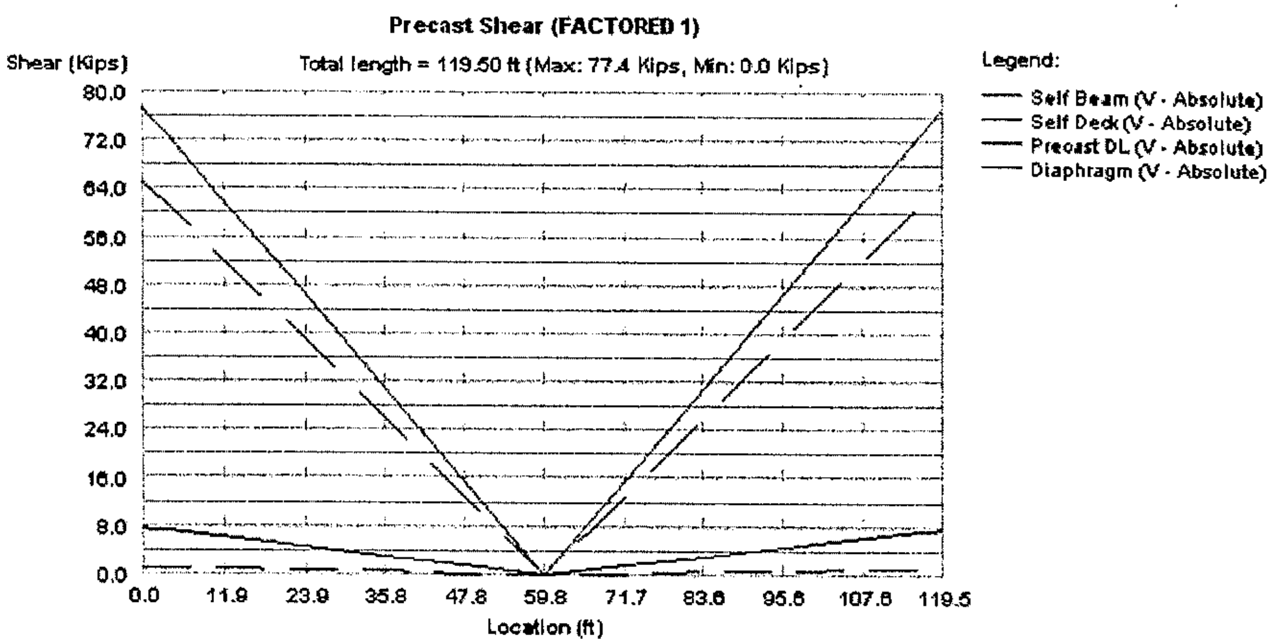
Upward reactions are positive.
Live Load reactions are per lane with no distribution factor and no impact.
Non-composite load types are per beam.
Composite and Pedestrian load types are per total bridge width.



Precast Moment, Span 1, Beam 3, FACTORED 1

BENTLEY	KZF Design Inc 655 Eden Park Dr Cincinnati OH 45202	Sheet: DS-26 Job No: **5355**
Program: CONSPAN® Rating Version: 8.0.0	 www.leapsoft.com 1-800-451-5327	By: def Date: Oct/27/2008 CKD: Date:

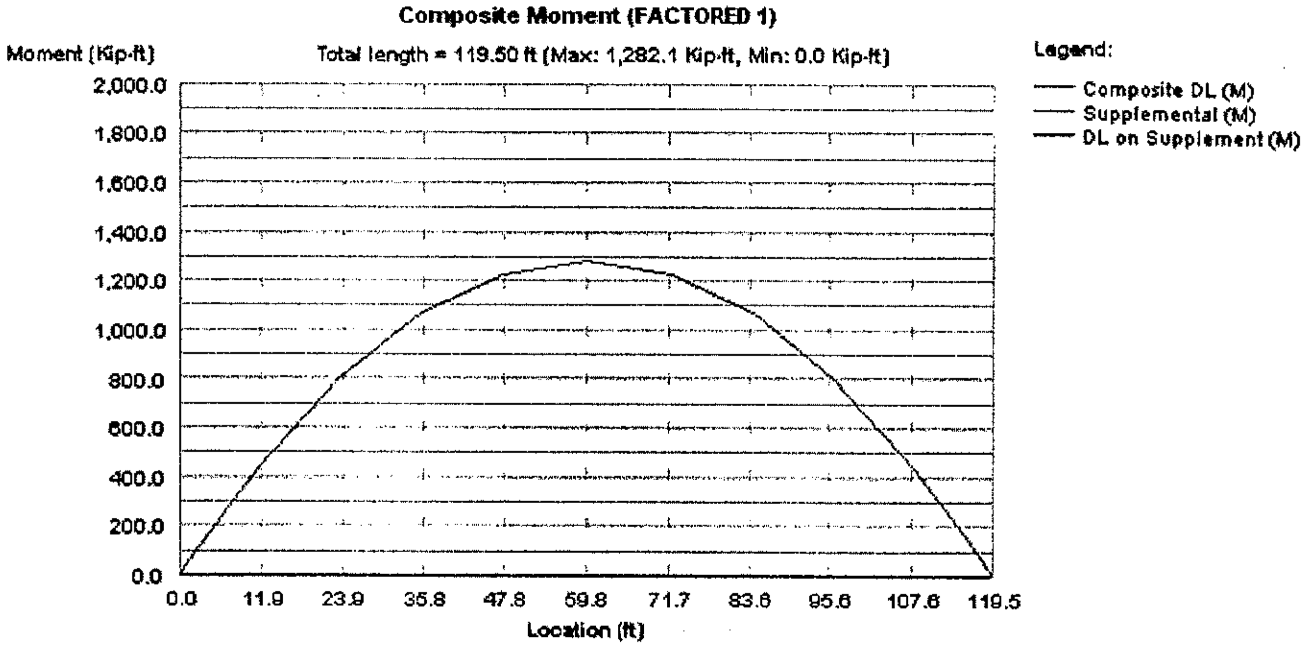
File Name: SCI-823-Shumway-CSX-7.75-ft-Spacing.csl



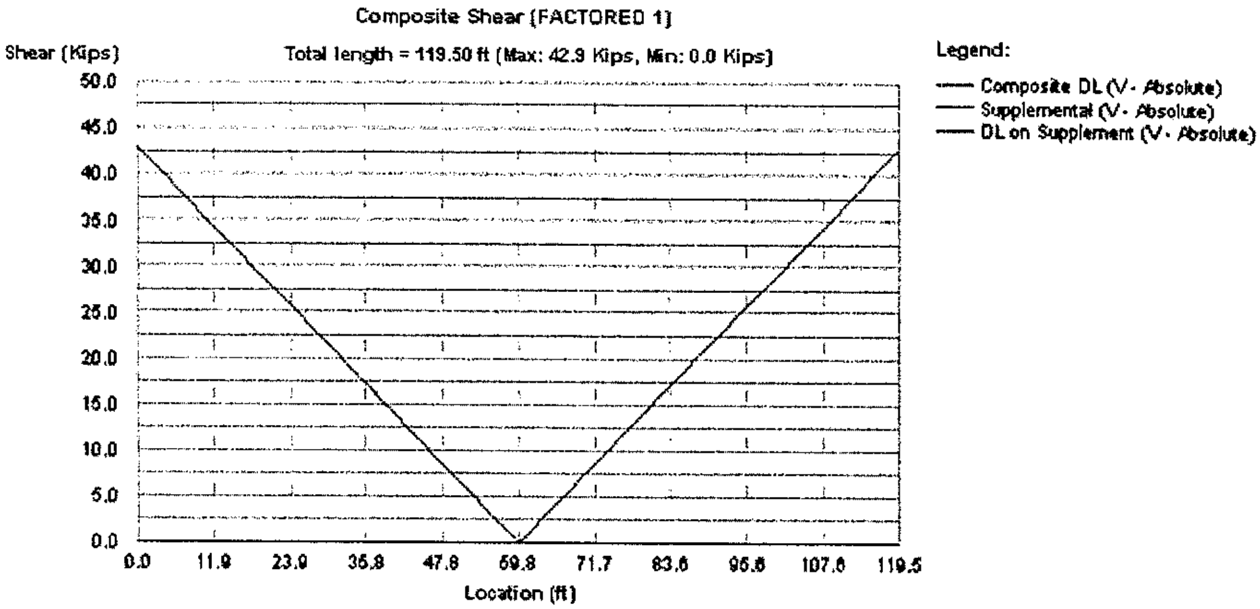
Precast Shear, Span 1, Beam 3, FACTORED 1

Units: U.S. Units

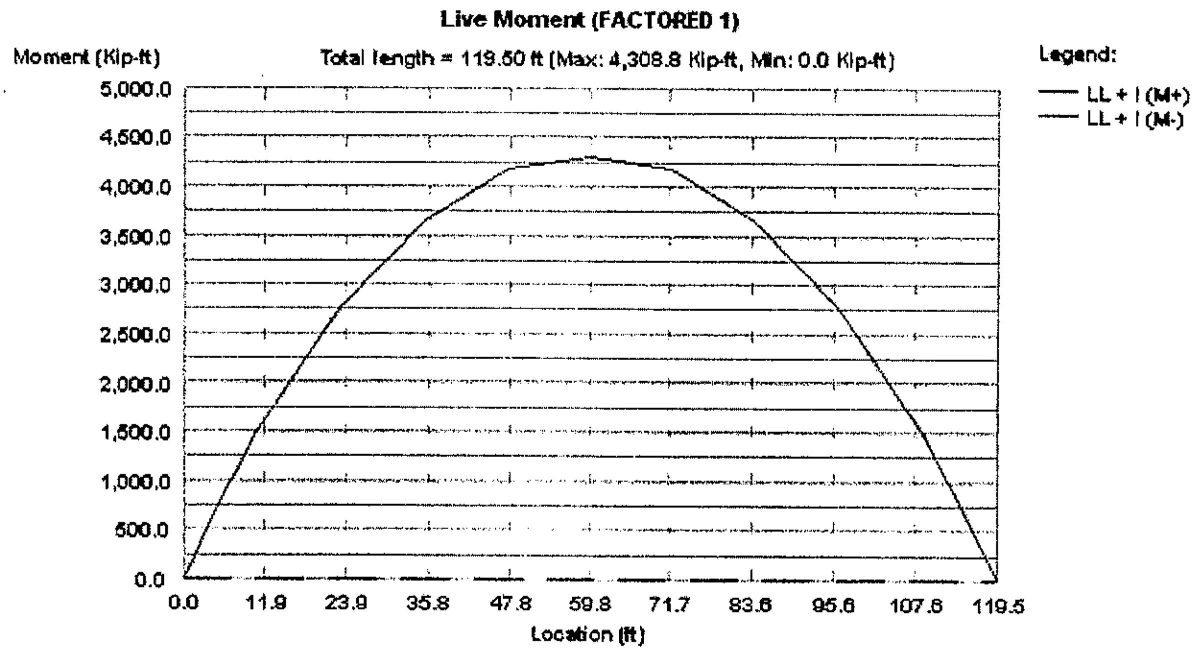
Design Code: AASHTO Standard



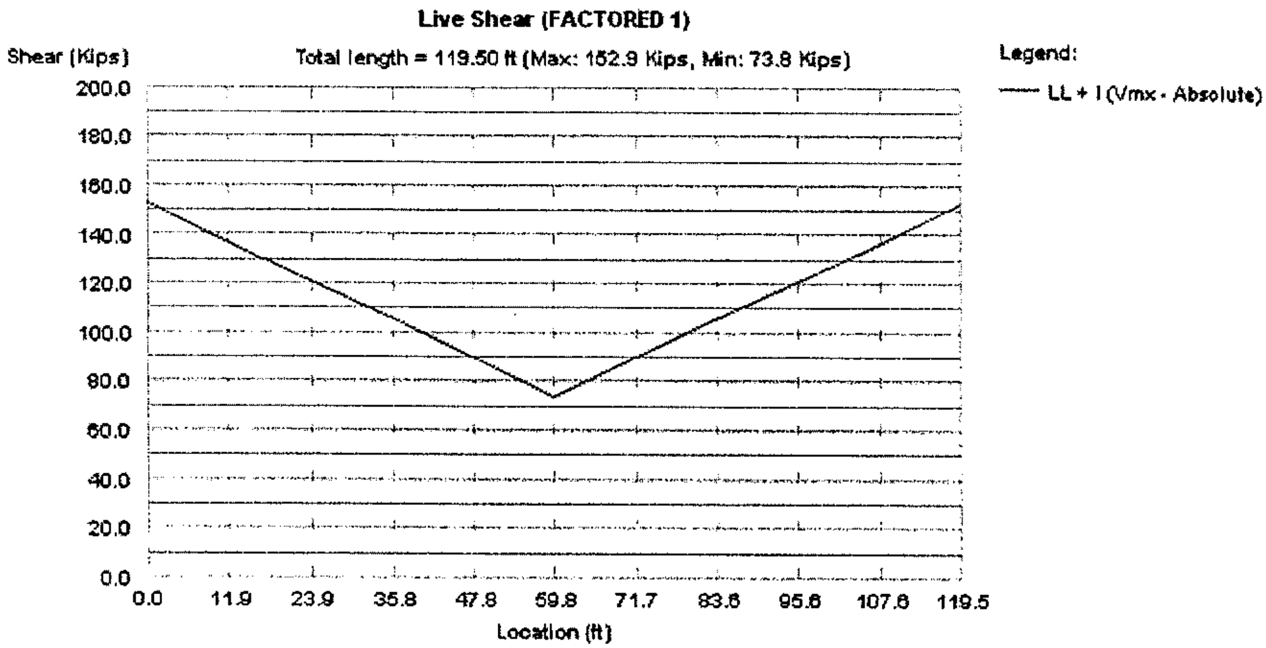
Composite Moment, Span 1, Beam 3, FACTORED 1



Composite Shear, Span 1, Beam 3, FACTORED 1



Live Moment, Span 1, Beam 3, FACTORED 1



Live Shear, Span 1, Beam 3, FACTORED 1

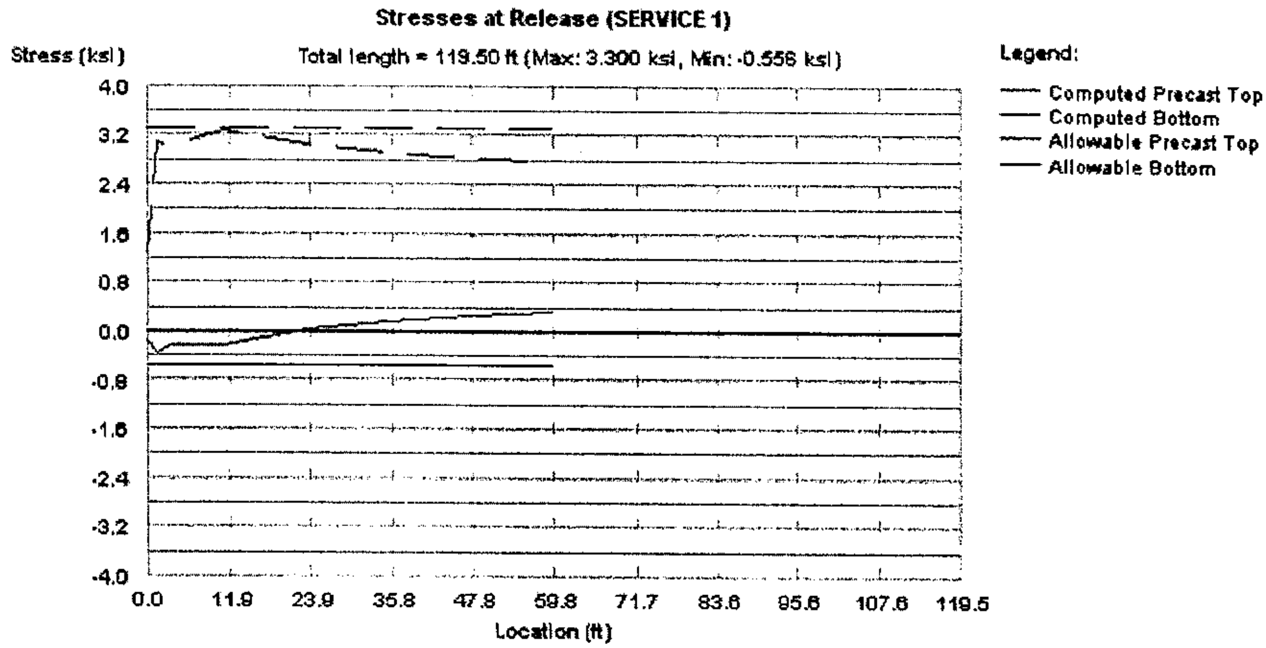
POSITIVE ENVELOPE STRESSES

Span : 1, Beam : 3, SERVICE 1

RELEASE STRESSES, (psi) (LOSS = 8.47 %)

	Trans	0.10L /0.90L	0.20L /0.80L	0.30L /0.70L	0.40L /0.60L	Midspan	Depress.
Location, ft	2.50	12.15	24.30	36.45	48.60	60.75	42.53
Self Wt.							
Precast-top	108.3	483.5	859.6	1128.2	1289.4	1343.1	1222.2
Bottom	-99.3	-443.3	-788.2	-1034.5	-1182.3	-1231.5	-1120.7
Prestress							
Precast-top	-467.8	-710.9	-841.1	-971.3	-1036.4	-1036.4	-1036.4
Bottom	3170.6	3716.1	3835.5	3954.8	4014.5	4014.5	4014.5
Total							
Precast-top	-359.5	-227.4	18.5	156.9	253.0	306.7	185.8
Bottom	3071.3	3272.8	3047.3	2920.3	2832.2	2783.0	2893.8
As_top (in2)	1.857	0.797	0.000	0.000	0.000	0.000	0.0

Span : 1, Beam : 3, SERVICE 1



Stresses at Release, Span 1, Beam 3, SERVICE 1

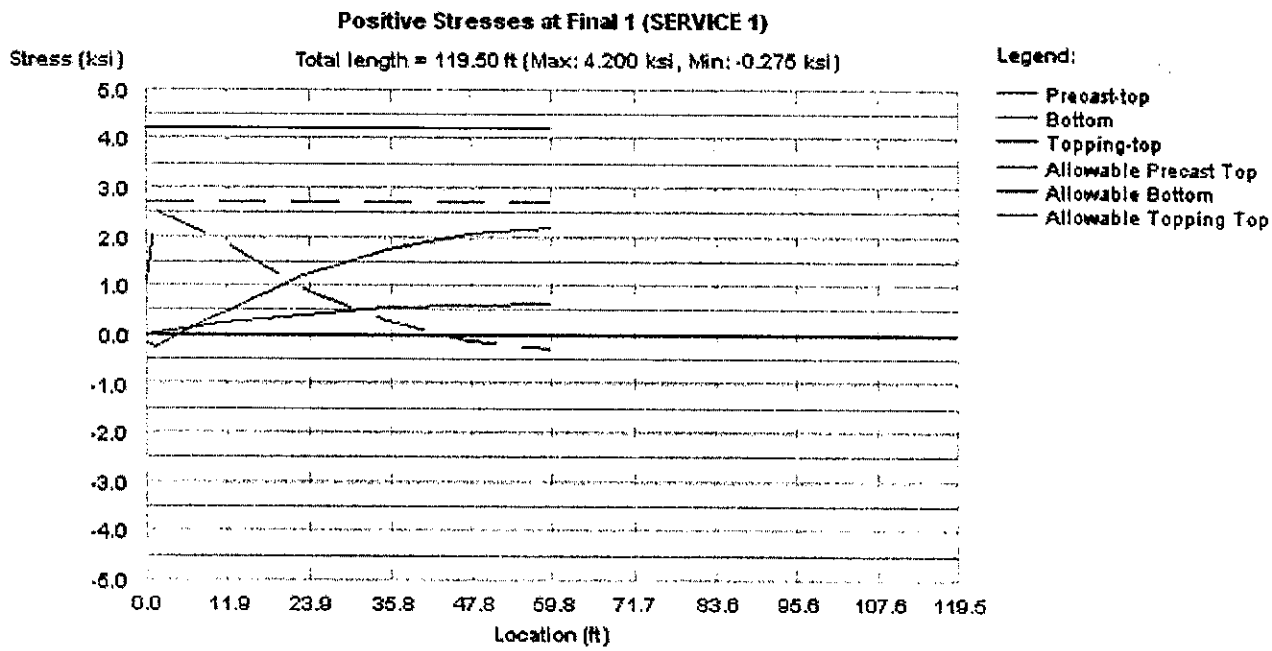
POSITIVE ENVELOPE STRESSES, (psi) (LOSS = 21.92%)

	Bearing	Trans	H/2	0.10L /0.90L	0.20L /0.80L	0.30L /0.70L	0.40L /0.60L	Midspan
Location, ft	0.00	1.50	3.44	11.15	23.30	35.45	47.60	59.75
Prestress								
Precast-top	-154.1	-399.1	-437.7	-606.5	-717.6	-828.6	-884.2	-884.2
Bottom	1076.9	2704.9	2788.5	3170.3	3272.2	3374.0	3424.9	3424.9
Self wt.								
Precast-top	-0.0	64.4	145.2	439.7	815.7	1084.3	1245.5	1299.2
Bottom	-0.0	-59.1	-133.1	-403.1	-748.0	-994.3	-1142.0	-1191.3
Prec. DL+ADL								
Precast-top	0.0	6.3	14.1	42.8	79.3	105.5	121.2	126.4
Bottom	-0.0	-5.7	-12.9	-39.2	-72.8	-96.7	-111.1	-115.9
Diaphragm								
Precast-top	0.0	0.9	2.2	6.9	14.0	17.7	21.4	21.3
Bottom	-0.0	-0.9	-2.0	-6.3	-12.8	-16.3	-19.6	-19.6

	Bearing	Trans	H/2	0.10L /0.90L	0.20L /0.80L	0.30L /0.70L	0.40L /0.60L	Midspan
Deck + Haunch								
Precast-top	0.0	54.3	122.3	370.4	687.2	913.5	1049.3	1094.6
Bottom	-0.0	-49.8	-112.2	-339.6	-630.1	-837.7	-962.2	-1003.7
Comp. DL+ADL								
Topping-top	-0.0	10.6	23.9	72.3	134.1	178.2	204.7	213.5
Precast-top	-0.0	8.7	19.5	59.1	109.6	145.7	167.4	174.6
Bottom	0.0	-22.5	-50.8	-153.8	-285.4	-379.4	-435.7	-454.5
LL+I(+)								
Topping-top	0.0	22.4	50.3	151.5	278.2	364.6	416.2	429.7
Precast-top	0.0	18.3	41.1	123.9	227.4	298.1	340.3	351.3
Bottom	-0.0	-47.6	-107.1	-322.6	-592.1	-776.0	-885.9	-914.7
Final 1 (P/S + DL + LL)								
Topping-top	-0.0	32.9	74.2	223.8	412.2	542.8	620.9	643.3
Precast-top	-154.1	-246.3	-93.2	436.1	1215.7	1736.2	2060.8	2183.3
Bottom	1076.9	2519.4	2370.3	1905.6	931.0	273.7	-131.6	-274.8
Final 2 (P/S + DL)								
Topping-top	-0.0	10.6	23.9	72.3	134.1	178.2	204.7	213.5
Precast-top	-154.1	-264.5	-134.4	312.2	988.3	1438.1	1720.5	1831.9
Bottom	1076.9	2567.0	2477.5	2228.2	1523.1	1049.7	754.3	640.0
Final 3 (50% P/S + 50% DL + LL)								
Topping-top	-0.0	27.7	62.3	187.7	345.2	453.7	518.5	536.5
Precast-top	-77.1	-114.0	-26.0	280.0	721.6	1017.1	1200.5	1267.3
Bottom	538.5	1235.9	1131.6	791.5	169.5	-251.2	-508.8	-594.8

$= 3.3\sqrt{f'_c} < 6\sqrt{f'_c}$

$\therefore$ o.K.



Positive Stresses at Final 1, Span 1, Beam 3, SERVICE 1

VERTICAL/HORIZONTAL SHEAR

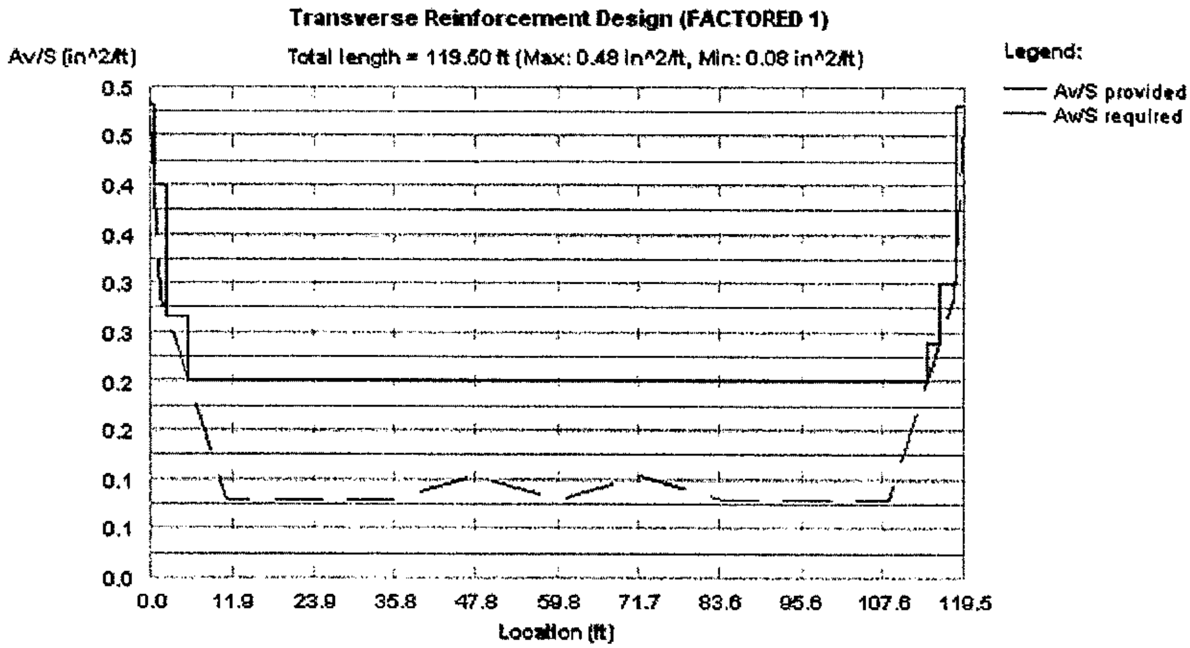
VERTICAL SHEAR (Art. 9.20) - Span : 1, Beam : 3, FACTORED 1

Location (ft)		Vd(kips)	Md(k.ft)	Ml(k.ft)	Vu(kips)	Mu(k.ft)	Vmu(kips)	Mmax(k.ft)	Vi(kips)
fpe (psi)	fd (psi)	Mcr (k.ft)	d (in)	Vci-com (kips)	Vci-min (kips)	Vci (kips)	fpc (psi)	Vp (kips)	Vcw (kips)
Vc (kips)	Vs-rqrd (kips)	Vs-max (kips)	Av-com (in2/ft)	Av-min (in2/ft)	Av (in2/ft)	Av-prvd (in2/ft)	pVn/Vu	MaxSpc (in)	Vs-crit (kips)
Bearing :	1.00	149.3	0.0	0.0	331.1	0.0	346.9	0.0	197.6
1076.9	-0.0	3425.9	70.66	10000.0	80.4	10000.0	187.5	5.0	202.3
202.3	165.6	378.3	0.47	0.08	0.47	0.48	1.011	24.0	189.2
Transfer :	2.50	145.6	221.1	103.3	331.1	511.7	340.1	290.5	194.6
2704.9	-138.0	6658.9	70.85	4633.5	80.6	4633.5	521.2	12.4	267.0
267.0	100.8	379.4	0.28	0.08	0.28	0.40	1.111	24.0	189.7
H/2 :	4.44	140.7	498.5	232.4	331.1	1152.7	331.1	654.2	190.3
2786.5	-311.0	6460.4	71.24	2048.9	81.1	2048.9	591.4	12.4	280.4
280.4	87.4	381.5	0.25	0.08	0.25	0.27	1.021	24.0	190.7
0.1L :	12.15	121.6	1510.0	699.9	295.4	3482.6	295.4	1972.5	173.9
3145.7	-942.0	5870.6	72.53	668.1	82.5	668.1	856.4	12.4	331.4
331.4	0.0	388.4	0.00	0.08	0.08	0.20	1.231	24.0	194.2
0.2L :	24.30	91.0	2803.5	1284.7	240.1	6433.5	240.1	3630.0	149.1
3272.2	-1749.1	4393.9	74.27	301.3	84.5	301.3	1136.7	12.4	389.0
301.3	0.0	397.7	0.00	0.08	0.08	0.20	1.408	24.0	198.8
0.3L :	36.45	60.8	3725.4	1683.8	185.1	8498.6	185.1	4773.2	124.3
3374.0	-2324.2	3366.9	75.67	178.9	86.1	178.9	1330.4	12.4	431.3
178.9	26.8	405.2	0.07	0.08	0.08	0.20	1.238	24.0	202.6
0.4L :	48.60	30.2	4280.5	1922.2	129.3	9737.9	116.9	5457.3	86.7
3424.9	-2670.6	2725.8	76.37	104.2	86.9	104.2	1452.4	0.0	445.1
104.2	39.5	408.9	0.10	0.08	0.10	0.20	1.256	24.0	204.5
0.5L :	60.75	0.0	4463.9	1984.7	73.8	10111.9	61.2	5648.0	61.2
3424.9	-2784.9	2477.7	76.37	57.5	86.9	86.9	1501.2	0.0	454.0
86.9	0.0	408.9	0.00	0.08	0.08	0.20	1.991	24.0	204.5
0.6L :	72.90	30.2	4280.5	1922.2	129.3	9737.7	116.9	5457.3	86.7
3424.9	-2670.6	2725.9	76.37	104.2	86.9	104.2	1452.4	0.0	445.1
104.2	39.5	408.9	0.10	0.08	0.10	0.20	1.256	24.0	204.5
0.7L :	85.05	60.8	3725.4	1683.8	185.1	8498.6	185.1	4773.2	124.3
3374.0	-2324.2	3366.9	75.67	178.9	86.1	178.9	1330.4	12.4	431.3
178.9	26.8	405.2	0.07	0.08	0.08	0.20	1.238	24.0	202.6
0.8L :	97.20	91.0	2803.4	1284.7	240.1	6433.4	240.1	3630.0	149.1
3272.2	-1749.0	4393.9	74.27	301.3	84.5	301.3	1136.7	12.4	389.0
301.3	0.0	397.7	0.00	0.08	0.08	0.20	1.408	24.0	198.8

	Location (ft)	Vd(kips)	Md(k.ft)	Ml(k.ft)	Vu(kips)	Mu(k.ft)	Vmu(kips)	Mmax(k.ft)	Vi(kips)
fpe (psi)	fd (psi)	Mcr (k.ft)	d (in)	Vci-com (kips)	Vci-min (kips)	Vci (kips)	fpc (psi)	Vp (kips)	Vcw (kips)
Vc (kips)	Vs-rqrd (kips)	Vs-max (kips)	Av-com (in2/ft)	Av-min (in2/ft)	Av (in2/ft)	Av-prvd (in2/ft)	pVn/Vu	MaxSpc (in)	Vs-crit (kips)
0.9L :	109.35	121.6	1510.1	699.9	295.4	3482.7	295.4	1972.6	173.9
3145.7	-942.1	5870.5	72.53	668.1	82.5	668.1	856.5	12.4	331.4
331.4	0.0	388.4	0.00	0.08	0.08	0.20	1.231	24.0	194.2
H/2 :	117.06	140.7	498.5	232.4	331.1	1152.7	331.1	654.2	190.3
2786.5	-311.0	6460.4	71.24	2048.9	81.1	2048.9	591.4	12.4	280.4
280.4	87.4	381.5	0.25	0.08	0.25	0.30	1.053	24.0	190.7
Transfer :	119.00	145.6	221.2	103.3	331.1	511.7	340.1	290.5	194.6
2704.9	-138.0	6658.8	70.85	4633.4	80.6	4633.4	521.2	12.4	267.0
267.0	100.8	379.4	0.28	0.08	0.28	0.30	1.015	24.0	189.7
Bearing :	120.50	149.3	0.0	0.0	331.1	0.0	346.9	0.0	197.6
1076.9	0.0	3425.9	70.66	10000.0	80.4	10000.0	187.5	5.0	202.3
202.3	165.6	378.3	0.47	0.08	0.47	0.48	1.011	24.0	189.2

ANCHORAGE ZONE REINFORCEMENT (Art. 9.22.1)
Span : 1, Beam : 3

Fpi (kips)	fs (ksi)	d/4 (in)	Abrst_rqrd (in2)
1494.04	20.00	15.24	2.99



Vertical Shear, Span 1, Beam 3, FACTORED 1

HORIZONTAL SHEAR (Art. 9.20.4) - Span : 1, Beam : 3

(Beam and Slab effects are INCLUDED in Vu).

Location (ft)	bv (in)	fsy (ksi)	Vu (kips)	Vnh-req (psi)	d (in)	Surf (in2/ft)	s_max (in)	Avh-min (in2/ft)	Avh-sm (in2/ft)	Avh-rg (in2/ft)
Bearing : 0.00	36.00	60.00	346.9	151.54	70.66	432.00	24.00	0.360	1.648	0.360
Transfer : 1.50	36.00	60.00	340.1	148.16	70.85	432.00	24.00	0.360	1.587	0.360
H/2 : 3.44	36.00	60.00	331.1	143.43	71.24	432.00	24.00	0.360	1.502	0.360
0.1L : 11.15	36.00	60.00	295.4	125.71	72.53	432.00	24.00	0.360	1.183	0.360

Location (ft)	bv (in)	fsy (ksi)	Vu (kips)	Vnh-req (psi)	d (in)	Surf (in2/ft)	s_max (in)	Avh-min (in2/ft)	Avh-sm (in2/ft)	Avh-rg (in2/ft)
0.2L : 23.30	36.00	60.00	240.1	99.78	74.27	432.00	24.00	0.360	0.716	0.360
0.3L : 35.45	36.00	60.00	185.1	75.50	75.67	432.00	24.00	0.360	0.360	0.000
0.4L : 47.60	36.00	60.00	129.3	52.27	76.37	432.00	24.00	0.360	0.360	0.000
0.5L : 59.75	36.00	60.00	73.8	29.83	76.37	432.00	24.00	0.360	0.360	0.000
0.6L : 71.90	36.00	60.00	129.3	52.27	76.37	432.00	24.00	0.360	0.360	0.000
0.7L : 84.05	36.00	60.00	185.1	75.51	75.67	432.00	24.00	0.360	0.360	0.000
0.8L : 96.20	36.00	60.00	240.1	99.78	74.27	432.00	24.00	0.360	0.716	0.360
0.9L : 108.35	36.00	60.00	295.4	125.72	72.53	432.00	24.00	0.360	1.183	0.360
H/2 : 116.06	36.00	60.00	331.1	143.44	71.24	432.00	24.00	0.360	1.502	0.360
Transfer : 118.00	36.00	60.00	340.1	148.16	70.85	432.00	24.00	0.360	1.587	0.360
Bearing : 119.50	36.00	60.00	346.9	151.54	70.66	432.00	24.00	0.360	1.648	0.360

CAMBER/DEFLECTION

CAMBER AND DEFLECTIONS: SERVICE 1
(Span : 1, Beam : 3; Units: in)

	Release	Mult	Erection	Mult	Final
At 0.1 x L =	11.15 ft				
Prestress	1.411	1.80	2.540	2.20	3.104
Self Wt.	-0.553	1.85	-1.023	2.40	-1.327
Deck + Haunch			-0.361	2.30	-0.831
Prec. DL+ADL			-0.042	3.00	-0.125
Diaphragm			-0.007	3.00	-0.021
Comp. DL+ADL			-0.110	3.00	-0.330
Live Load(+I)					-0.188
Total	0.858		0.997		0.281

	Release	Mult	Erection	Mult	Final
At 0.2 x L =	23.30 ft				
Prestress	2.538	1.80	4.568	2.20	5.583
Self Wt.	-1.046	1.85	-1.935	2.40	-2.510
Deck + Haunch			-0.715	2.30	-1.645
Prec. DL+ADL			-0.083	3.00	-0.248
Diaphragm			-0.014	3.00	-0.042
Comp. DL+ADL			-0.218	3.00	-0.653
Live Load(+I)					-0.376
Total	1.492		1.603		0.109

	Release	Mult	Erection	Mult	Final
At 0.3 x L =	35.45 ft				
Prestress	3.358	1.80	6.044	2.20	7.387
Self Wt.	-1.432	1.85	-2.649	2.40	-3.437
Deck + Haunch			-0.993	2.30	-2.284
Prec. DL+ADL			-0.115	3.00	-0.344
Diaphragm			-0.020	3.00	-0.059
Comp. DL+ADL			-0.302	3.00	-0.907
Live Load(+I)					-0.527
Total	1.926		1.965		-0.171

	Release	Mult	Erection	Mult	Final
At 0.4 x L =	47.60 ft				
Prestress	3.855	1.80	6.939	2.20	8.481
Self Wt.	-1.677	1.85	-3.103	2.40	-4.025
Deck + Haunch			-1.170	2.30	-2.691
Prec. DL+ADL			-0.135	3.00	-0.405
Diaphragm			-0.023	3.00	-0.069
Comp. DL+ADL			-0.356	3.00	-1.068
Live Load(+I)					-0.626
Total	2.178		2.152		-0.404

	Release	Mult	Erection	Mult	Final
At 0.5 x L =	59.75 ft				
Prestress	4.021	1.80	7.237	2.20	8.846
Self Wt.	-1.761	1.85	-3.258	2.40	-4.227
Deck + Haunch			-1.231	2.30	-2.831

	Release	Mult	Erection	Mult	Final
Prec. DL+ADL			-0.142	3.00	-0.426
Diaphragm			-0.024	3.00	-0.073
Comp. DL+ADL			-0.374	3.00	-1.123
Live Load(+I)					-0.658
Total	2.260		2.208		-0.492

	Release	Mult	Erection	Mult	Final
At 0.6 x L =	71.90 ft				
Prestress	3.855	1.80	6.939	2.20	8.481
Self Wt.	-1.677	1.85	-3.103	2.40	-4.025
Deck + Haunch			-1.170	2.30	-2.691
Prec. DL+ADL			-0.135	3.00	-0.405
Diaphragm			-0.023	3.00	-0.069
Comp. DL+ADL			-0.356	3.00	-1.068
Live Load(+I)					-0.626
Total	2.178		2.152		-0.404

	Release	Mult	Erection	Mult	Final
At 0.7 x L =	84.05 ft				
Prestress	3.358	1.80	6.044	2.20	7.387
Self Wt.	-1.432	1.85	-2.649	2.40	-3.437
Deck + Haunch			-0.993	2.30	-2.284
Prec. DL+ADL			-0.115	3.00	-0.344
Diaphragm			-0.020	3.00	-0.059
Comp. DL+ADL			-0.302	3.00	-0.907
Live Load(+I)					-0.527
Total	1.926		1.965		-0.171

	Release	Mult	Erection	Mult	Final
At 0.8 x L =	96.20 ft				
Prestress	2.538	1.80	4.568	2.20	5.583
Self Wt.	-1.046	1.85	-1.935	2.40	-2.510
Deck + Haunch			-0.715	2.30	-1.645
Prec. DL+ADL			-0.083	3.00	-0.248
Diaphragm			-0.014	3.00	-0.042
Comp. DL+ADL			-0.218	3.00	-0.653
Live Load(+I)					-0.376
Total	1.492		1.603		0.109

	Release	Mult	Erection	Mult	Final
At 0.9 x L =	108.35 ft				
Prestress	1.411	1.80	2.540	2.20	3.104
Self Wt.	-0.553	1.85	-1.023	2.40	-1.327
Deck + Haunch			-0.361	2.30	-0.831
Prec. DL+ADL			-0.042	3.00	-0.125
Diaphragm			-0.007	3.00	-0.021
Comp. DL+ADL			-0.110	3.00	-0.330
Live Load(+I)					-0.188
Total	0.858		0.997		0.281

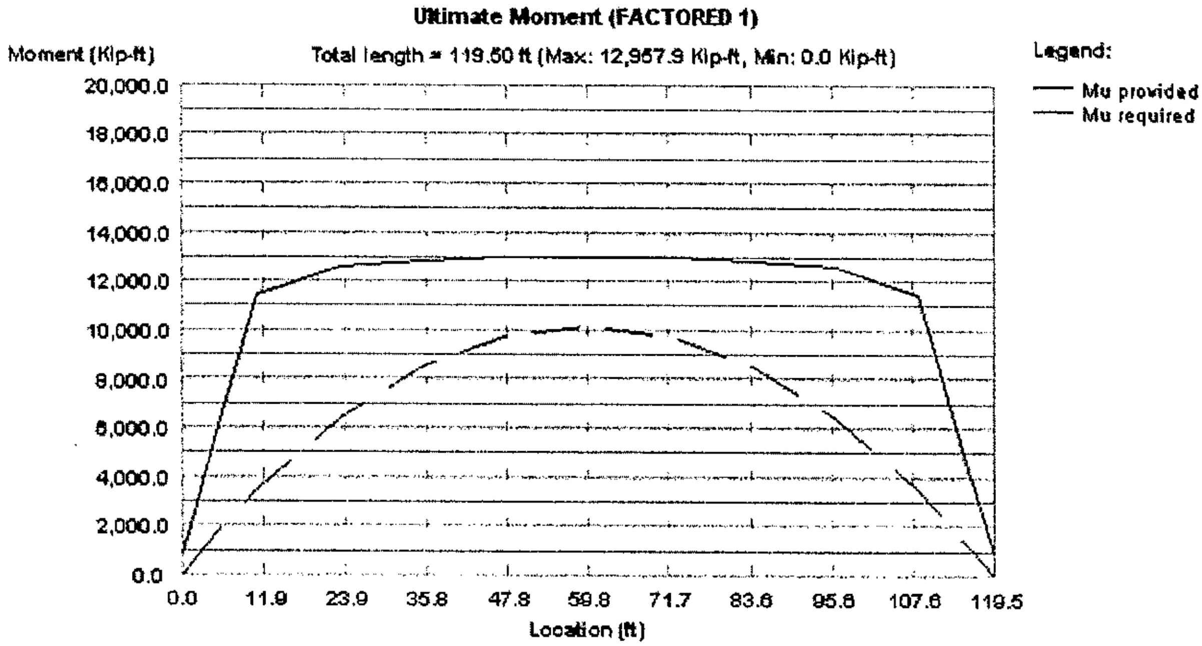
Positive values indicate upward deflection.

ULTIMATE MOMENT

ULTIMATE - Span : 1, Beam : 3, FACTORED 1
(Mu-prvd computed by Strain Compatibility method. Ult. Conc. Strain = 0.00300)
(f'c_eff, ksi = 4.50; beta1 = 0.825)

Location (ft)	A*s in2	Ycg in	p*(A*s/bd)	f*su ksi	a in	Mu-prvd k.ft	Mu-rqrd k.ft	Mcr k.ft	Crkg Ratio	Mu-p/r Ratio
Transfer 1.50	1.530	11.65	0.00025	269.5	1.2	2257.3	511.7	7152.3	0.316	4.41
H/2 3.44	2.741	11.26	0.00044	269.0	2.1	4031.1	1152.7	7235.5	0.557	3.50
0.1L 11.15	7.893	9.97	0.00122	260.4	5.8	11448.4	3482.6	7706.3	1.486	-
0.2L 23.30	8.463	8.23	0.00127	260.4	6.2	12568.4	6433.5	7469.6	1.683	-
0.3L 35.45	8.463	6.83	0.00125	260.5	6.2	12828.1	8498.6	7364.6	1.742	-
0.4L 47.60	8.463	6.13	0.00124	260.5	6.2	12957.9	9737.9	7278.6	1.780	-
0.5L 59.75	8.463	6.13	0.00124	260.5	6.2	12957.9	10111.9	7213.9	1.796	-
0.6L 71.90	8.463	6.13	0.00124	260.5	6.2	12957.9	9737.7	7278.7	1.780	-
0.7L 84.05	8.463	6.83	0.00125	260.5	6.2	12828.1	8498.6	7364.6	1.742	-
0.8L 96.20	8.463	8.23	0.00127	260.4	6.2	12568.4	6433.4	7469.7	1.683	-
0.9L 108.35	7.893	9.97	0.00122	260.4	5.8	11448.4	3482.7	7706.3	1.486	-
H/2 116.06	2.741	11.26	0.00044	269.0	2.1	4031.1	1152.7	7235.4	0.557	3.50
Transfer 118.00	1.530	11.65	0.00025	269.5	1.2	2257.3	511.7	7152.3	0.316	4.41

BENTLEY	KZF Design Inc 655 Eden Park Dr Cincinnati OH 45202	Sheet: DS-42 Job No: **5355**
Program: CONSPAN® Rating Version: 8.0.0	Copyright © Bentley Systems, Inc. 1984 - 2008. www.leapsoft.com 1-800-451-5327	By: def Date: Oct/27/2008 CKD:
File Name: SCI-823-Shumway-CSX-7.75-ft-Spacing.csl		Date:



Ultimate Moment, Span 1, Beam 3, FACTORED 1

Units: U.S. Units

Design Code: AASHTO Standard

DETENSIONING

Span : 1, Beam : 3; Groups 1-21; Units: psi

Grp	Str		Ys,in		2.50ft	6.50ft
1	2	E	2.00	Ft	108.400	
		M	2.00	Fb	-99.40*	
2	2	E	68.00	Ft	339.079	
		M	6.00	Fb	-149.64*	
3	1	E	68.00	Ft	454.419	
		M	6.00	Fb	-174.76*	
4	2	E	69.00	Ft	581.135	
		M	69.00	Fb	-210.31*	
5	2	E	8.00	Ft	535.743	
		M	8.00	Fb	-7.42*	
6	2	E	8.00	Ft	490.351	536.678
		M	8.00	Fb	195.480	314.275
7	2	E	6.00	Ft	435.161	481.488
		M	6.00	Fb	407.360	526.155
8	2	E	6.00	Ft	379.971	426.298
		M	6.00	Fb	619.239	738.035
9	2	E	6.00	Ft	324.781	371.108
		M	6.00	Fb	831.119	949.915
10	2	E	6.00	Ft	269.591	315.918
		M	6.00	Fb	1042.999	1161.794
11	2	E	4.00	Ft	269.591	250.931
		M	4.00	Fb	1042.999	1382.658
12	2	E	4.00	Ft	204.604	185.943
		M	4.00	Fb	1263.863	1603.522
13	2	E	4.00	Ft	139.616	120.956
		M	4.00	Fb	1484.727	1824.385
14	2	E	4.00	Ft	74.629	55.968
		M	4.00	Fb	1705.590	2045.249
15	2	E	4.00	Ft	9.641	-9.019
		M	4.00	Fb	1926.454	2266.113
16	1	E	4.00	Ft	-22.853	-41.513
		M	4.00	Fb	2036.886	2376.545
17	2	E	2.00	Ft	-97.638	-116.298
		M	2.00	Fb	2266.733	2606.393
18	2	E	2.00	Ft	-172.423	-191.083
		M	2.00	Fb	2496.581	2836.240
19	2	E	2.00	Ft	-247.208	-265.868
		M	2.00	Fb	2726.429	3066.088
20	2	E	2.00	Ft	-321.993	-340.653
		M	2.00	Fb	2956.276	3295.935
21	1	E	2.00	Ft	-359.386	-378.046
		M	2.00	Fb	3071.200	3410.86*

DESIGN SUMMARY

Span: 1, Beam: 3, Interior beam

Beam type:		
	I-Girder,	OHIO-MOD-72-TYPE-IV
Precast Length,	ft	121.50
Release Length,	ft	121.50
Strand Pattern:	Straight/Draped	Depr. Point: 0.35 L
Strand:	6/10-270K-LL	
Strand Type:	Low Relaxation	
Strand Es,	ksi:	28500.0
No. of strands:	39	
	Draped:	3
	Straight:	36
Concrete Strength:		
	f'ci:	5500.0 psi
	f'c:	7000.0 psi
	f'ct:	4500.0 psi
Initial losses:	8.47 %	
Final losses:	21.92 %	

Specification	Allowable	Computed	Status
Release Stresses (psi) (Art. 9.15.2.1)			
Precast Top w/ no reinf.	-200.00		
Precast Top w/ reinf.	-556.21	-359.52	OK
Precast Bot (compression)	3300.00	3272.76	OK
Factored 1	Provided	Required	
Ult. Moment (k.ft)	12957.89	10111.87	OK
Debonding Limits	Allowable	Computed	
Max. Debond per Row	40.00 %	18.18 %	OK
Max. Debond Total	25.00 %	10.26 %	OK

Positive Moment Envelope Stresses (psi)

Specification	Final 1 Allow	Comp	Final 2 Allow	Comp	Final 3 Allow	Comp
Service 1						
Topping Top	2700.0/-503.1	643.3 / -0.0	1800.0	213.5	1800.0	536.5

	Final 1		Final 2		Final 3	
Specification	**Allow**	**Comp**	**Allow**	**Comp**	**Allow**	**Comp**
Precast Top	4200.0/-502.0	2183.3 /-246.3	2800.0	1831.9	2800.0	1267.3
Precast Bot	4200.0/-502.0	2519.4 /-274.8	2800.0	2567.0	2800.0	1235.9

CAMBER / DEFLECTION: (PCI Design Handbook - 4th Ed.- Table 4.6.2)
0.5 x L = 59.75 ft

	Release	**Mult**	**Erection**	**Mult**	**Final**
Prestress	4.021	1.80	7.237	2.20	8.846
Self Wt.	-1.761	1.85	-3.258	2.40	-4.227
Deck + Haunch			-1.231	2.30	-2.831
Prec. DL+ADL			-0.142	3.00	-0.426
Diaphragm			-0.024	3.00	-0.073
Comp. DL+ADL			-0.374	3.00	-1.123
Live Load					-0.658
Total	2.260		2.208		-0.492

Positive values indicate upward deflection.

BENTLEY	KZF Design Inc 655 Eden Park Dr Cincinnati OH 45202	Sheet: DS-1 Job No: **5355**
Program: CONSPAN® Rating Version: 8.0.0	Copyright © Bentley Systems, Inc. 1984 - 2008. www.leapsoft.com 1-800-451-5327	By: def Date: Oct/27/2008 CKD: D.A.T. Date: 2-10-09
File Name: SCI-823-Shumway-CSX-7.75-ft-Spacing-NoFWS... .csl		

DESIGN STATUS

Span:1, Beam:3

NO FWS APPLIED

RELEASE STRESSES (psi)

Allowable Stresses

Comp(top)	Comp(bot)	Tens with Reinf	Tens without Reinf(top)	Tens without Reinf(bot)
3300.000	3300.000	-556.2	-200.0	-0.0

Computed Stresses

	Trans	0.10L/0.90L	0.20L/0.80L	0.30L/0.70L	0.40L/0.60L	Midspan	Depress.
Location, ft	2.500	12.150	24.300	36.450	48.600	60.750	42.525
Precast-top	-359.5	-227.4	18.5	156.9	253.0	306.7	185.8
Bottom	3071.3	3272.8	3047.3	2920.3	2832.2	2783.0	2893.8
As_top, in2	1.857	0.797	0.000	0.000	0.000	0.000	0.000

FINAL STRESSES (psi)

Allowable Stresses

		Topping-Top	Precast-Top	Bottom	Bottom (M-)
Final 1	Compression	2700.0	4200.0	4200.0	4200.0
Final 1	Tension	-503.1	-502.0	-502.0	-
Final 2	Compression	1800.0	2800.000	2800.000	2800.000
Final 3	Compression	1800.0	2800.0	2800.0	2800.0

Computed Stresses

POSITIVE MOMENT ENVELOPE : SERVICE 1 (Final 1)

	Bearing	Trans	H/2	0.10L/0.90L	0.20L/0.80L	0.30L/0.70L	0.40L/0.60L	Midspan
Location, ft	0.000	1.500	3.438	11.150	23.300	35.450	47.600	59.750
Topping-top	0.0	24.9	56.1	168.9	310.3	407.3	465.3	480.9
Precast-top	-152.2	-247.8	-102.5	398.9	1141.5	1636.0	1944.8	2061.8
Bottom	1063.3	2502.2	2373.5	1982.3	1106.4	519.3	156.1	27.3

POSITIVE MOMENT ENVELOPE : SERVICE 1 (Final 2)

	Bearing	Trans	H/2	0.10L/0.90L	0.20L/0.80L	0.30L/0.70L	0.40L/0.60L	Midspan
Topping-top	0.0	2.5	5.7	17.3	32.2	42.7	49.1	51.2
Precast-top	-152.2	-266.0	-143.6	275.0	914.1	1337.9	1604.5	1710.5
Bottom	1063.3	2549.8	2480.7	2304.9	1698.5	1295.3	1042.0	942.0

POSITIVE MOMENT ENVELOPE : SERVICE 1 (Final 3)

	Bearing	Trans	H/2	0.10L/0.90L	0.20L/0.80L	0.30L/0.70L	0.40L/0.60L	Midspan
Topping-top	0.0	23.6	53.2	160.2	294.2	385.9	440.7	455.3
Precast-top	-76.1	-114.7	-30.7	261.4	684.5	967.0	1142.5	1206.6
Bottom	531.6	1227.3	1133.2	829.9	257.2	-128.4	-364.9	-443.7

ULTIMATE MOMENT (k.ft)

FACTORED 1

	Trans	H/2	0.10L/0.90L	0.20L/0.80L	0.30L/0.70L	0.40L/0.60L	Midspan	Depress
Location, ft	1.500	3.438	11.150	23.300	35.450	47.600	59.750	41.525
Mu-req'd	463.4	1043.8	3152.8	5821.6	7685.2	8803.5	9137.2	8244.4
Mu-prv'd	2238.9	3998.6	11358.9	12568.0	12827.7	12957.5	12957.5	12892.6

CAMBER / DEFLECTION (in) at Midspan (0.5 x L = 59.75 ft)

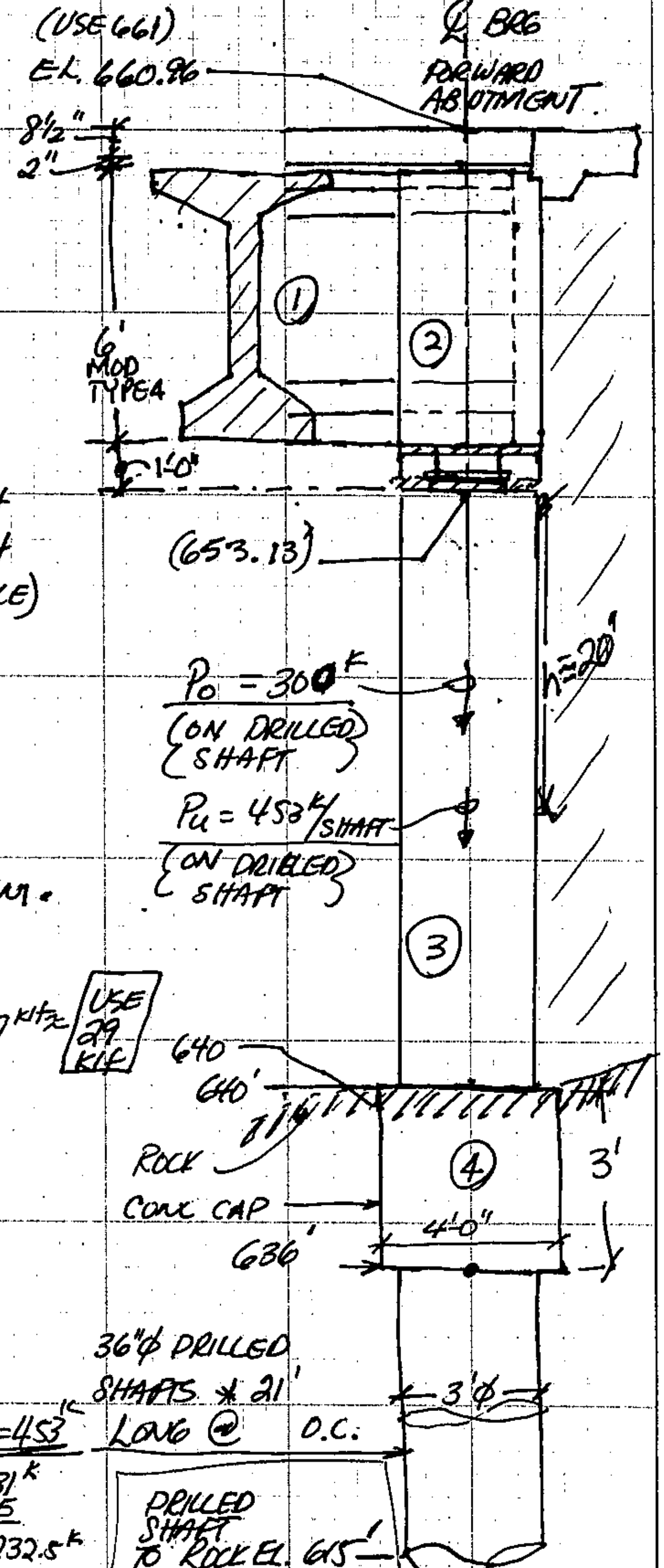
SERVICE 1

	Release	Mult	Erection	Mult	Final
Prestress	4.021	1.80	7.237	2.20	8.846
Self Beam	-1.761	1.85	-3.258	2.40	-4.227
Self Deck			-1.231	2.30	-2.831
Prec. DL+ADL			-0.142	3.00	-0.426
Diaphragm			-0.024	3.00	-0.073
Comp. DL+ADL			-0.090	3.00	-0.269
Live Load					-0.658
Total	2.260		$\boxed{2.492}$		0.362

$\delta = 2.5''$ ↑

No Future WS APPLIED

D. ABUTMENT DESIGN



FORWARD ABUTMENT:

ELEV @ PGL = 660.96
SLOPE = 2%
DIST $\delta = 29.5(.02) = 0.59'$

BORING B-26 ROCK @ 642.7, $\gamma \approx 147$ pcf
B-27 ROCK @ 640.3, $\gamma \approx 144$ pcf
BEARING = 80 ksf (ALLOWABLE)

PHASE 1 - EXCAVATE
2 - DRILLED SHAFT.
3 - CAP & WALL CONST.
4 - BACK FILL.
5 - SET BM'S & CONST DIAPHRAGM.
6 - COMPLETE BACK FILL.

① DEAD LOAD BM = 149^k (176)
LIVE LOAD + I BM = 71^k (71) } @ $7.75' = 28.387^{k/ft} \approx$ $\boxed{\text{USE } 29 \text{ klf}}$
② DIAPHRAGM = $7 \times 3 \times .15 \times 1' = \underline{3.15 \text{ klf}}$
③ WALL = $13.13 \times 3 \times 0.15 = \underline{5.91}$
④ CAP = $4' \times 4' \times 1' \times 0.15 = \underline{2.4 \text{ klf}}$ (3)

$\underline{TOTAL = 29 + 3.15 + 5.91 + 2.4 \approx 40.0 \text{ klf}}$

TRY SHAFTS @ 7.5' O.C.:
TRIAL = 7.5' O.C. $P_o = 40.0(7.5) = 300^k (\downarrow)$

$P_u = 1.3(DL + 1.67(LL+I)) = \underline{453^k} \downarrow$
$= 1.3(DL) + 2.17(LL+I) = 1.3(233) + 2.17(69) = \underline{453}^k$

• $DL = \frac{149}{7.75} + 3.15 + 5.91 + 2.4 \approx 30.69 \approx 31^k$
$\times 7.5$
$= 232.5^k$

• $LL+I = \frac{71}{7.75} \times 7.5' = 68.71 \approx \underline{69^k}$

KZF DESIGN Architecture | Engineering | Interiors | Planning

project name: SCI-823 – TR234/CSXT-RR

project #: #5355

subject: FORWARD ABUTMENT

date: 1/09

designed by: DSF

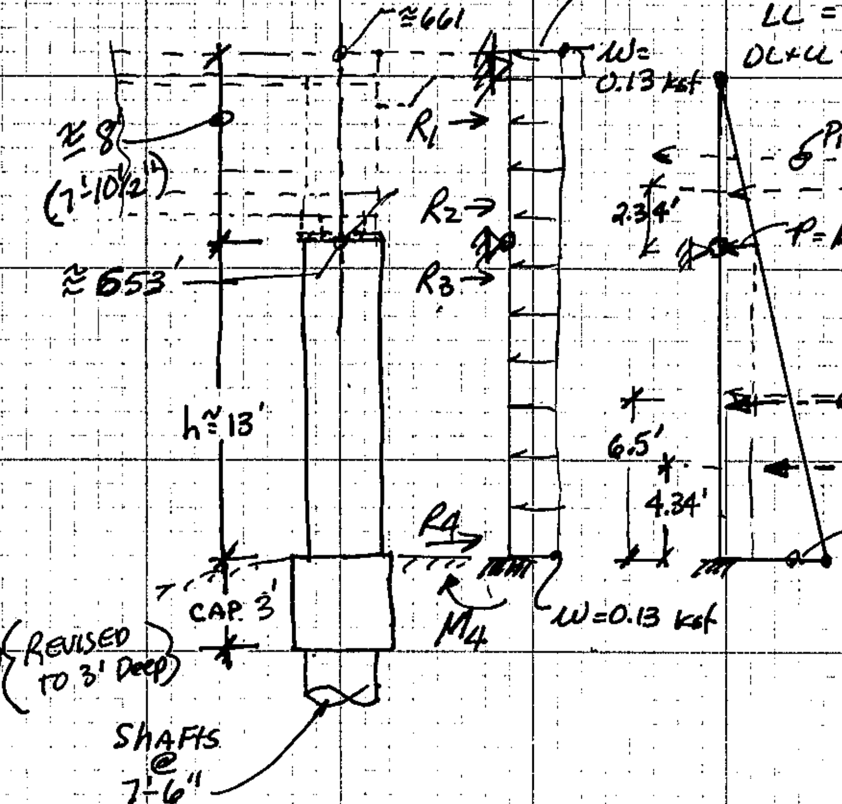
checked by: D.A.

(Use $k_a = .33$)

Surcharge DL Conc = $1' \times .15 = .150$ ksf $\times .33 = .050$ ksf

LL = Use 240 psf = $.24 \times .33 = .08$ ksf

DL+LL = $.05 + .08 = 0.13$ ksf



$P_1 = .13(8) = 1.04^k$

$P_2 = (.28)(7)(.5) = .98^k$ @ 2.34'

$P = k_a \gamma h = .33 \times 120 \times 7' = 277$ psf/Lf ≈ .28 ksf/Lf

$P_3 = .13 \times 13' = 1.69^k$

$P_4 = .28 \times 13 = 3.64^k$

$P_5 = (.80 - .28 = .52 \times 13 \times .5) = 3.38$

$P = k_a \gamma h = .33(120)(20) = 792$ psf/Lf

$\boxed{P = .80 \text{ Klf/Lf}}$

CHECK Sum $P_1 + P_3 = 1.04 + 1.69 = 2.73^k \Rightarrow 21 \times .13 = 2.73^k$ ✓

Sum $P_2, P_4, P_5 = .98 + 3.64 + 3.38 = \underline{8.0}\ k \Rightarrow \frac{1}{2} k_a \gamma h^2 = .5(.33)(.12)(20)^2 = \underline{7.92}^k$ OK

CHECK CONCRETE SLAB STRESS: (COMPRESSION @ TOP)

$R_1 = 1.04^k/2 + .98^k(2.34/7) = 0.848$ Klf.

Conc Added Stress Compression $= \frac{848}{12'' \times 8.5''} = \underline{8.4 \text{ psi}}$ Very Very Low Conc. COMPRESSION

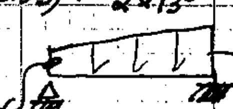
BEARING SHEAR (BM SPACING = 7.75' O.C.)

(A) HAND EQUATION

$R_2 = (1.04^k/2) + 0.98\left(\frac{4.66'}{7}\right) = (0.52 + 0.653) = 1.173$ Klf $\times 7.75' = \underline{9.10}$ kips

$R_3 = \frac{5}{16}(P) + \frac{Pb^2}{2l^3}(a+2l) = .3125(5.33) + \frac{3.38(4.34)^2}{2 \times 13^3}(8.66 + (2)13) = 1.67^k + 0.50 = 2.17$ klf $\times 7.75 =$ 16.[illegible] kip

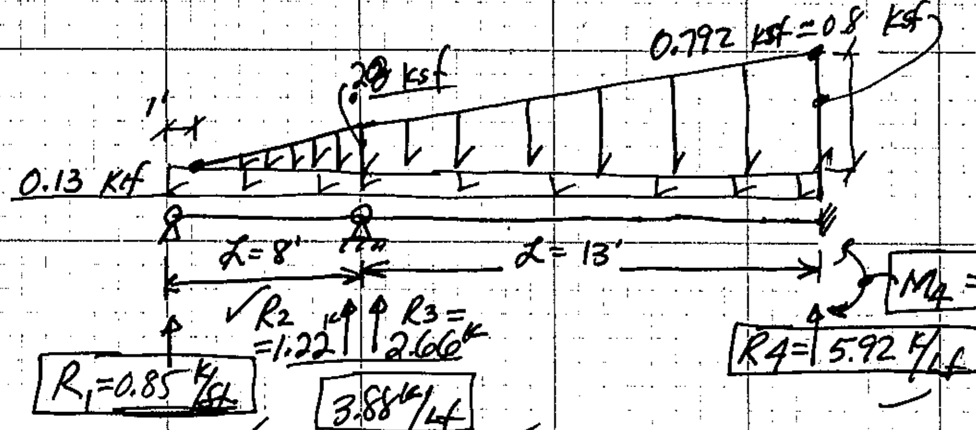
(B) ⇒ Use DIST. LOAD ANALYSIS:



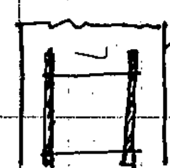
$w_1 = 0.13 + .28 = \underline{0.41}$ klf

$w_2 = 0.78 + 0.13 = \underline{0.91}$ klf

Use = $\boxed{R_3 = 2.66^k}$ $R_4 = 5.92^k$, $M = -14.2^{k'}$



COMPUTER ANALYSIS ok



ρ_{MIN} = TRIAL 1%

$As_{MIN} = 36" \times 12" \times .01 = 4.32\ in^2/ft. \div 2$ FACES $= 2.16\ in^2/ft.$

As, TRIAL, use #8 BARS @ 6" o.c. = As = 1.58 $in^2/ft.$

Use $Mu = 1.55(14.2) = 22$ k-FT/ft., $\phi = 0.9$, $b = 12$, $d = 30"$ min.

$Ru = \frac{Mu}{\phi b d^2} = \frac{(22)(12)(1000)}{(0.9)(12)(30)^2} = 27.2\ psi \Rightarrow \rho_{min} = 0.0018$

$As = \rho bd = .0018 \times 12 \times 30 = 0.648\ in^2/ft. \Rightarrow$ $\boxed{\#8 @ 12" \text{ o.c. Preliminary OK}}$

$\boxed{As = 0.79\ in^2/ft}$

ADD BEARING PAD:

BREAKING FORCE = $(.64 * 121.5 + 18)(0.05) * 1.25 \times 4 = 24^K(2/3)$ TO ONE END / 8 Bms = $2^K/Bm.$

DL + LL = 220^K (↓)

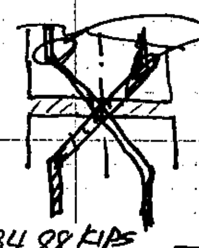
EARTH PRESSURE SHEAR = 3.88 + BREAKING FORCE = $2^K/7.75' = 4.14$ Klf ⇒ Use 4.5

DL DIAPHRAGM = $3.15^K(8') = 25.2^K$ (↓)

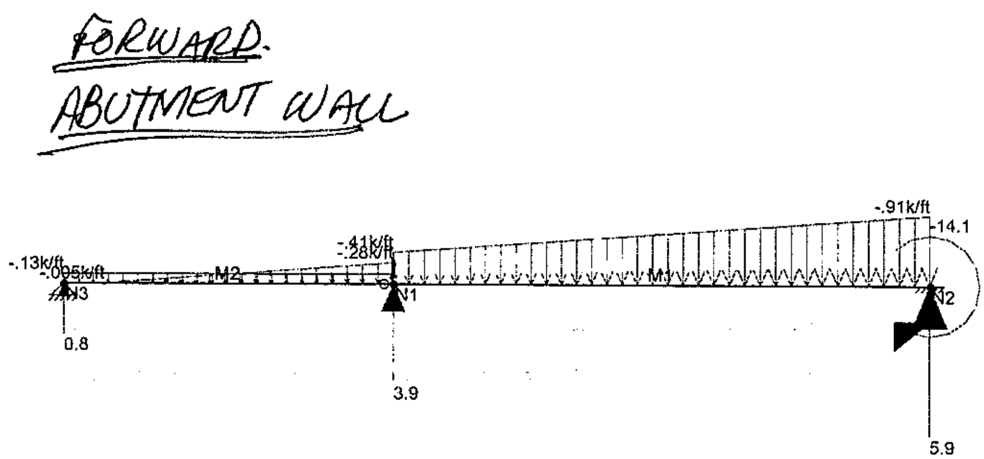
TRIAL DIAPHRAGM STEEL TO WALL:

#6 BAR = .44 (24 ksi SHEAR)

= 10.5 KIPS/Lf. > 4.5 Klf OK



V Shear @ BEARING PAD = 4.5 Klf (7.75') = 34.88 KIPS → 35^K → Shear @ BRG Pad OK



Loads: BLC 1, Load #1
Results for LC 1, LOAD #1
Reaction units are k and k-ft

		Dec 8, 2008 at 7:15 PM
		SCI-RR-Forward-Abutment.r3d

Global

Display Sections for Member Calcs	5
Max Internal Sections for Member Calcs	97
Include Shear Deformation	Yes
Include Warping	Yes
Area Load Mesh (in^2)	144
Merge Tolerance (in)	.12
P-Delta Analysis Tolerance	0.50%
Vertical Axis	Y

Hot Rolled Steel Code	AISC: ASD 9th
Cold Formed Steel Code	AISI 99: ASD
Wood Code	NDS 91/97: ASD
Wood Temperature	< 100F
Concrete Code	ACI 2002

Number of Shear Regions	4
Region Spacing Increment (in)	4
Biaxial Column Method	PCA Load Contour
Parme Beta Factor (PCA)	.65
Concrete Stress Block	Rectangular
Use Cracked Sections	Yes
Bad Framing Warnings	No
Unused Force Warnings	Yes

General Material Properties

	Label	E [ksi]	G [ksi]	Nu	Therm (\1E5 F)	Density[k/ft^3]
1	gen_Conc3NW	3155	1372	.15	.6	.145
2	gen_Conc4NW	3644	1584	.15	.6	.145
3	gen_Conc3LW	2085	906	.15	.6	.11
4	gen_Conc4LW	2408	1047	.15	.6	.11
5	gen_Alum	10600	4077	.3	1.29	.173
6	gen_Steel	29000	11154	.3	.65	.49
7	RIGID	1e+6		.3	0	0

Hot Rolled Steel Properties

	Label	E [ksi]	G [ksi]	Nu	Therm (\1E5 F)	Density[k/ft^3]	Yield[ksi]
1	A36 Gr.36	29000	11154	.3	.65	.49	36
2	A572 Gr.50	29000	11154	.3	.65	.49	50
3	A992	29000	11154	.3	.65	.49	50
4	A500 Gr.42	29000	11154	.3	.65	.49	42
5	A500 Gr.46	29000	11154	.3	.65	.49	46

General Section Sets

	Label	Shape	Type	Material	A [in2]	Iyy [in4]	Izz [in4]	J [in4]
1	GEN1A	RE4X4	Beam	gen_Conc3NW	16	21.333	21.333	31.573
2	RIGID		None	RIGID	1e+6	1e+6	1e+6	1e+6

Hot Rolled Steel Section Sets

	Label	Shape	Type	Design List	Material	Design Rules	A [in2]	Iyy [in4]	Izz [in4]	J [in4]
1	HR1A	W10X17	Beam	Wide Flange	A36 Gr.36	Typical	9.71	36.6	170	.58

Member Primary Data

	Label	I Joint	J Joint	K Joint	Rotate(deg)	Section/Shape	Type	Design List	Material	Design Rules
1	M1	N1	N2			HR1A	Beam	Wide Flange	A36 Gr.36	Typical
2	M2	N3	N1			HR1A	Beam	Wide Flange	A36 Gr.36	Typical

Member Advanced Data

	Label	I Release	J Release	I Offset[in]	J Offset[in]	T/C Only	Physical	TOM	Inactive
1	M1						Yes		
2	M2		BenPIN				Yes		

Joint Coordinates and Temperatures

	Label	X [ft]	Y [ft]	Z [ft]	Temp [F]	Detach From Diap...
1	N1	0	0	0	0	
2	N2	13	0	0	0	
3	N3	-8	0	0	0	

Joint Boundary Conditions

	Joint Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot.[k-ft/rad]	Y Rot.[k-ft/rad]	Z Rot.[k-ft/rad]	Footing
1	N1		Reaction					
2	N2	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction	
3	N3	Reaction	Reaction	Reaction				

Hot Rolled Steel Design Parameters

	Label	Shape	Length...	Lbyy[ft]	Lbzz[ft]	Lcomp to...	Lcomp bo...	Kyy	Kzz	Cm-yy	Cm-zz	Cb	y sway	z sway	Function
1	M1	HR1A	13												Lateral
2	M2	HR1A	8												Lateral

Joint Loads and Enforced Displacements

Joint Label	L,D,M	Direction	Magnitude[(k,k-ft), (in,rad), (k*s...
No Data to Print ...			

Member Point Loads (BLC 2 : Load #2)

	Member Label	Direction	Magnitude[k,k-ft]	Location[ft,%]
1	M1	Y	-5.33	6.5
2	M1	Y	-3.25	8.67

Member Distributed Loads (BLC 1 : Load #1)

	Member Label	Direction	Start Magnitude[k/ft,d...	End Magnitude[k/ft,d...	Start Location[ft,%]	End Location[ft,%]
1	M1	Y	-.28	-.78	0	0
2	M1	Y	-.13	-.13	0	0
3	M2	Y	-.13	-.13	0	0
4	M2	Y	-.005	-.28	1	0

Plate Surface Loads

Plate Label	Direction	Magnitude[ksf,deg]
No Data to Print ...		

Member Area Loads

Joint A	Joint B	Joint C	Joint D	Direction	Distribution	Magnitude[ksf]
No Data to Print ...						

Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Z Gravity	Joint	Point	Distributed	Area (Me...	Surface (...
1	Load #1	None						4		
2	Load #2	None					2			

Load Combinations

	Description	Sol...	PD...	SR...	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	LOAD #1	Yes			1	1														
2	LOAD #2	Yes			2	1														

Load Combination Design

	Description	ASIF	CD	ABIF	Service	Hot Rolled	Cold Formed	Wood	Concrete	Footings
1	LOAD #1					Yes	Yes	Yes	Yes	Yes
2	LOAD #2					Yes	Yes	Yes	Yes	Yes

Joint Deflections

	LC	Joint Label	X [in]	Y [in]	Z [in]	X Rotation [rad]	Y Rotation [rad]	Z Rotation [rad]
1	1	N1	0	0	0	0	0	-1.751e-3
2	1	N2	0	0	0	0	0	0
3	1	N3	0	0	0	0	0	-3.236e-4

Joint Reactions (By Combination)

	LC	Joint Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
1	1	N1	0	3.881	0	0	0	0
2	1	N2	0	5.92	0	0	0	-14.148
3	1	N3	0	.816	0	0	0	0
4	1	Totals:	0	10.617	0			
5	1	COG (ft):	X: 5.301	Y: 0	Z: 0			

Member Section Forces (By Combination)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
1	1	M1	1	0	2.66	0	0	0	0
2			2	0	1.124	0	0	0	-6.26
3			3	0	-.817	0	0	0	-6.868
4			4	0	-3.166	0	0	0	-.506
5			5	0	-5.92	0	0	0	14.148
6	1	M2	1	0	.816	0	0	0	0
7			2	0	.531	0	0	0	-1.363
8			3	0	.104	0	0	0	-2.025
9			4	0	-.48	0	0	0	-1.675
10			5	0	-1.221	0	0	0	0

Member Section Stresses

	LC	Member Label	Sec	Axial[ksi]	y Shear[ksi]	z Shear[ksi]	y top Bendin...	y bot Bendin...	z top Bendin...	z bot Bendin...
1	1	M1	1	0	1.096	0	0	0	0	0
2			2	0	.463	0	4.636	-4.636	0	0
3			3	0	-.337	0	5.087	-5.087	0	0
4			4	0	-1.305	0	.375	-.375	0	0
5			5	0	-2.44	0	-10.479	10.479	0	0
6	1	M2	1	0	.336	0	0	0	0	0
7			2	0	.219	0	1.01	-1.01	0	0
8			3	0	.043	0	1.5	-1.5	0	0
9			4	0	-.198	0	1.241	-1.241	0	0
10			5	0	-.503	0	0	0	0	0

Member Torsion Stresses

	LC	Member Label	Sec	Torque[k-ft]	Shear[ksi]	y Warp Shear...	z Warp Shear...	z-Bot Warp B...	z-Top Warp B...
1	1	M1	1	0	0	NC	NC	NC	NC
2			2	0	0	NC	NC	NC	NC
3			3	0	0	NC	NC	NC	NC
4			4	0	0	NC	NC	NC	NC
5			5	0	0	NC	NC	NC	NC
6	1	M2	1	0	0	NC	NC	NC	NC
7			2	0	0	NC	NC	NC	NC
8			3	0	0	NC	NC	NC	NC
9			4	0	0	NC	NC	NC	NC
10			5	0	0	NC	NC	NC	NC

Member AISC ASD Steel Code Checks

	LC	Member	Shape	UC Max	Loc[ft]	Shear U...	Loc[ft]	Dir	Fa[ksi]	Ft[ksi]	Fby[ksi]	Fbz[ksi]	Cb	Cmy	Cmz	Eqn
1	1	M1	W10X17	.595	13	.169	13	y	4.378	21.6	27	17.62	1.75	.6	.85	H1-3
2	1	M2	W10X17	.093	4.417	.035	8	y	11.174	21.6	27	16.361	1	.6	1	H1-1

KZF DESIGN Architecture | Engineering | Interiors | Planning

project name: SCI-823/TR234

project #: #5355

subject: FORWARD ABUTMENT
DRILLED SHAFT COLUMN

date: 1/09

designed by: PSF

checked by: D.A.T.

COLUMN: (ADDITIONAL CHECK)

36"Φ DRILLED SHAFT w/ 3'x4' CONC CAP.

LOADS	P_o		P_u	
✓ DL BM	= 19.3 klf	× 1.3 ≅	25.1	✓
✓ LL BM	= 9.2 klf	* 2.17 ≅	20.0	✓
✓ DIAPHRAGM	= 3.15 klf	× 1.3 =	4.1	✓
✓ WALL	= 5.91 klf	× 1.3 =	7.7	✓
✓ CAP =	2.4 klf	× 1.3 =	3.2	
(REVISED TO 3'x4')	P_o ≅ 40.0 klf		P_u = 60.1 klf × 7.5' = 451^K ✓ OK	

MOMENT = 14.2 klf * 1.6 = 22.72 k-ft/lf

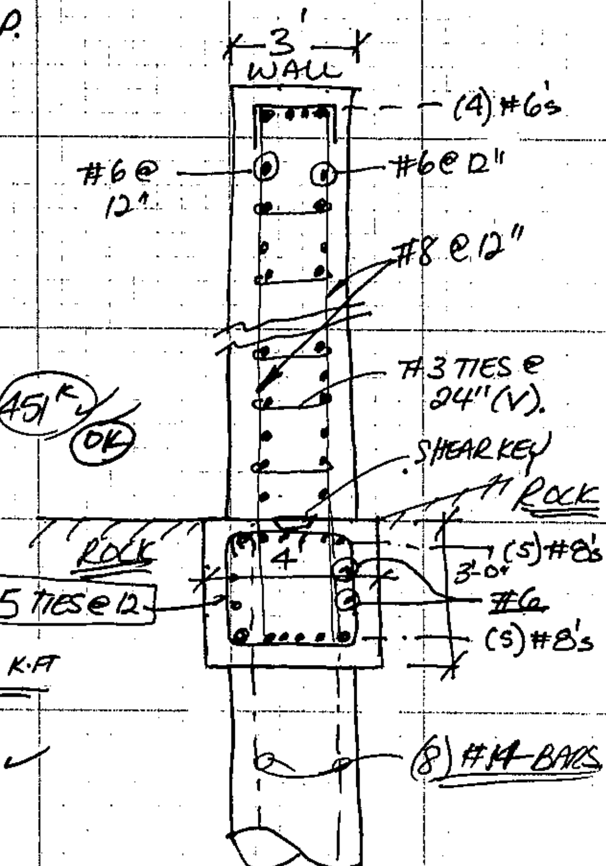
M_o = 14.2×7.5 = 106.5 k' ✓

M_u = 22.72×7.5' = 170.4 k' ✓
≅ 171 k'

TRIAL:

SHAFTS @ 7'-6" O.C. P_u ≅ 451^K, M_u = 171 K-FT

$\boxed{\text{36"Φ CONC SHAFT w/ (8) \#14 BARS}}$ OK ✓



WALL:

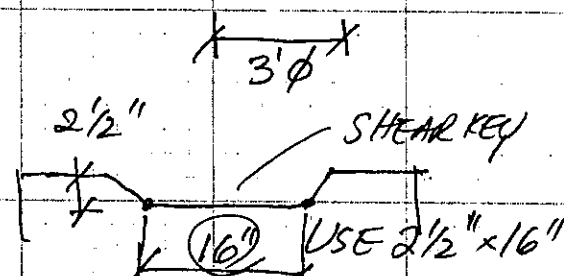
$$\frac{M_u}{\phi b d^2} = \frac{22.72(12000)}{(.9)(12)(30)^2} = 29 \Rightarrow \rho = \rho_{min.}$$

$\rho_{min} = .0018$

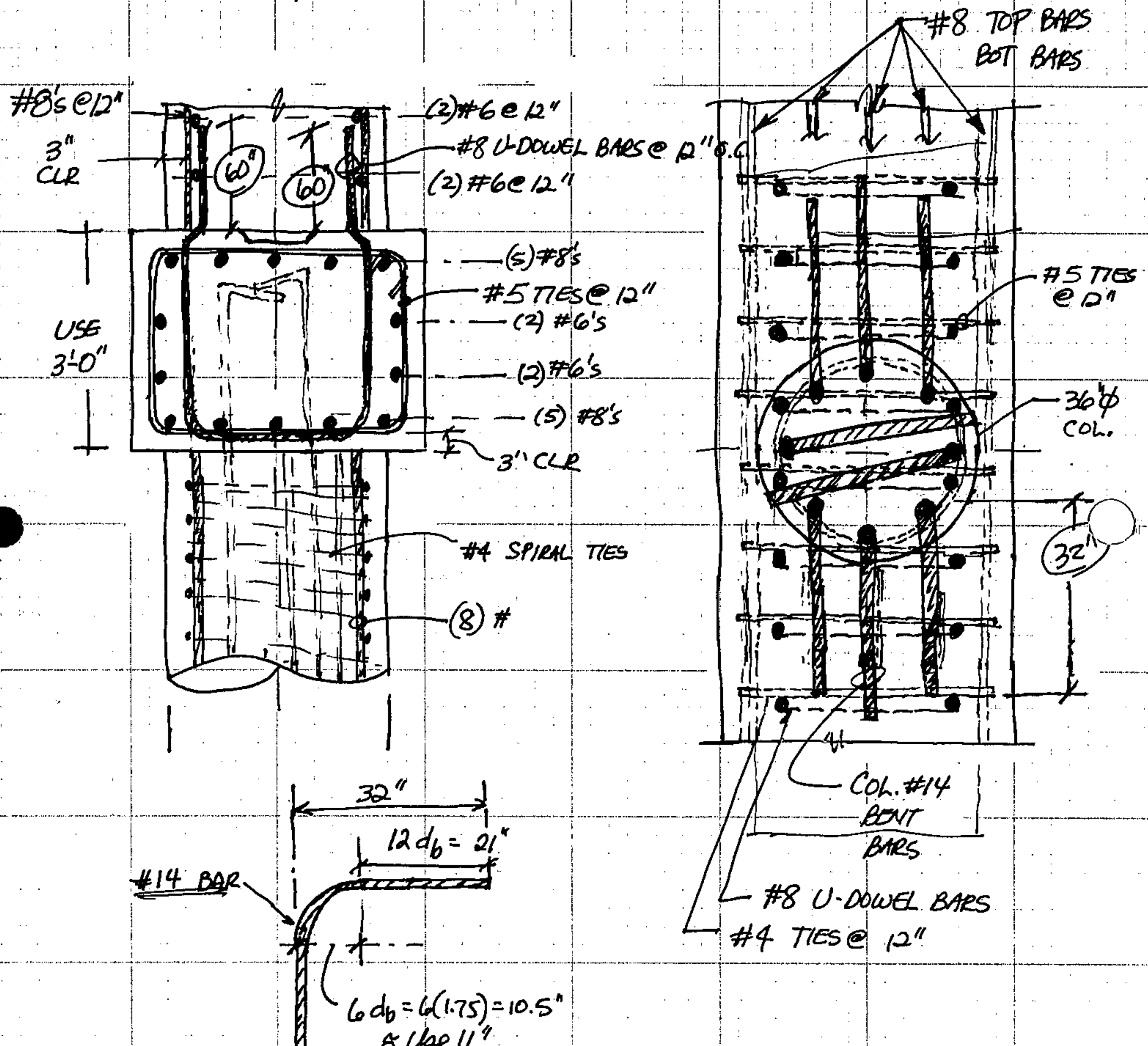
$A_s = \rho b d = (.0018)(12)(30) = 0.65$

$\boxed{\text{Use \#8 BARS @ 12" O.C.}}$

$\boxed{A_s = 0.79 \text{ in}^2 > .65 \text{ in}^2}$ OK



COLUMN/CAP/BM REINFORCEMENT



Concrete Column

Lic. # : KW-06004130 | License Owner : kzf, inc.

Description : --None--

General Information

Code Ref : 2006 IBC, ACI 318-05

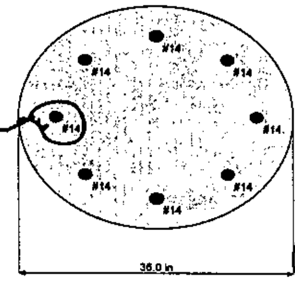
f'c : Concrete 28 day strength = 4.0 ksi
E = 3,122.0 ksi
Density = 145.0 pcf
β = 0.850
Fy - Main Rebar = 60.0 ksi
E - Main Rebar = 29,000.0 ksi
Allow. Reinforcing Limits *ASTM A615 Bars Used*
Min. Reinf. = 1.0 %
Max. Reinf. = 8.0 %
Load Combination :2006 IBC & ASCE 7-05

Overall Column Height = 10.0 ft
End Fixity Top Free, Bottom Fixed
ACI Code Year ACI 318-05
Brace condition for deflection (buckling) along columns :
X-X (width) axis : Fully braced against buckling along X-X Axis
Y-Y (depth) axis :Fully braced against buckling along Y-Y Axis
Type of Stirrups used : Spirals
Fy - Stirrups = 40.0 ksi
E - Stirrups = 29,000.0 ksi

Column Cross Section

Column Dimensions :36.0in Diameter, Column Edge to Rebar Edge Cover = 4.0in

Column Reinforcing : 8.0 - #14 bars



36"Ø

Applied Loads

Entered loads are factored per load combinations specified by user.

Column self weight included : 10,249.5 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 10.0 ft above base, Yecc = 3.50in, D = 300.0, L = 186.0 k
BENDING LOADS . . .
Lat. Point Load at 10.0 ft creating Mx-x, H = 11.0 k

DESIGN SUMMARY

Maximum Stress Ratio = 0.31334 : 1
Load Combination +1.20D+0.50Lr+1.60L+1.60H
Location of max.above base 0.0 ft
At maximum location values are . . .
Pu 669.899 k
φ * Pn 669.899 k
Mu-x 367.80 k-ft
φ* Mn-x : 1,173.82 k-ft
Mu-y 0.0 k-ft
φ * Mn-y : 0.0 k-ft

Column Capacities . . .
Pnmax : Nominal Max. Compressive Axial Capacity 4,479.58 k
Pnmin : Nominal Min. Tension Axial Capacity -1,080.0 k
φ Pn, max : Usable Compressive Axial Capacity 2,665.35 k
φ Pn, min : Usable Tension Axial Capacity -756.0 k

Maximum SERVICE Load Reactions . .
Top along Y-Y 0.0 k Bottom along Y-Y 0.0 k
Top along X-X 0.0 k Bottom along X-X 11.0 k

Maximum SERVICE Load Deflections . . .
Along Y-Y -0.047263 in at 10.0 ft above base
for load combination : D+L+Lr+S
Along X-X 0.0 in at 0.0 ft above base
for load combination :

General Section Information . φ = 0.70 β =0.850 θ = 0.850
ρ : % Reinforcing 1.7684 % Rebar % Ok
Reinforcing Area 18.0 in^2
Concrete Area 1,017.88 in^2

Governing Load Combination Results

Governing Factored Load Combination	Dist. from base ft	Pu	φ * Pn	Pu/ φPn	Dist. from base ft	δx * Mux	φ Mnx	δy * Muy	φ Mny	(Mux/ φMnx + Muy/ φMny)
		Axial Load Analysis k				Bending Analysis k-ft				
+1.40D	0.00	434.35	434.35	**1.000**	0.00	122.50	1,191.43			0.103
+1.20D+0.50Lr+1.60L+1.60H	0.00	669.90	669.90	**1.000**	0.00	367.80	1,173.82			0.313
+1.20D+0.50L+0.50S+1.60H	0.00	465.30	465.30	**1.000**	0.00	308.13	1,190.01			0.259
+1.20D+1.60Lr+0.50L	0.00	465.30	465.30	**1.000**	0.00	132.13	1,190.01			0.111
+0.90D+1.60W+1.60H	0.00	279.22	279.22	**1.000**	0.00	254.75	1,191.53			0.214

Concrete Column

Description : --None--

Governing Load Combination Results

Governing Factored Load Combination	Dist. from base ft	Axial Load Analysis k Pu	φ * Pn	Pu/φPn	Dist. from base ft	Bending Analysis k-ft δx * Mux	φ Mnx	δy * Muy	φ Mny	(Mux/ φMnx + Muy/ φMny)
+0.90D+E+1.60H	0.00	279.22	279.22	**1.000**	0.00	254.75	1,191.53			**0.214**

Maximum Reactions - Unfactored

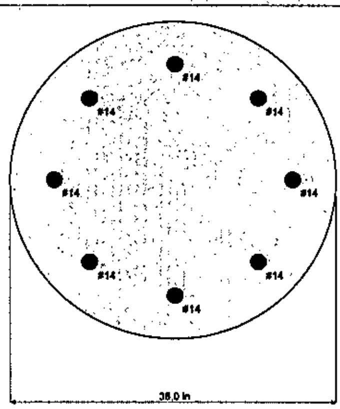
Note: Only non-zero reactions are listed.

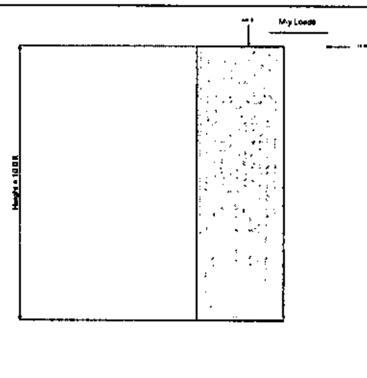
Load Combination	Reaction along X-X Axis @ Base	Reaction along X-X Axis @ Top	Reaction along Y-Y Axis @ Base	Reaction along Y-Y Axis @ Top
D Only				
Lr Only				
L Only				
S Only				
Lr+L+S				
W Only				
E Only				
H Only	11.000			
D+L+Lr+S				

Maximum Deflections for Load Combinations - Unfactored Loads

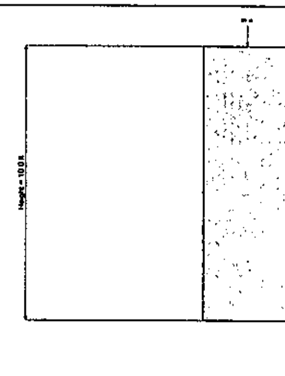
Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
Lr+L+S	0.0000 in	0.000 ft	0.000 in	0.000 ft
W Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
D+L+Lr+S	0.0000 in	0.000 ft	0.000 in	0.000 ft

Sketches





Looking along X-X Axis

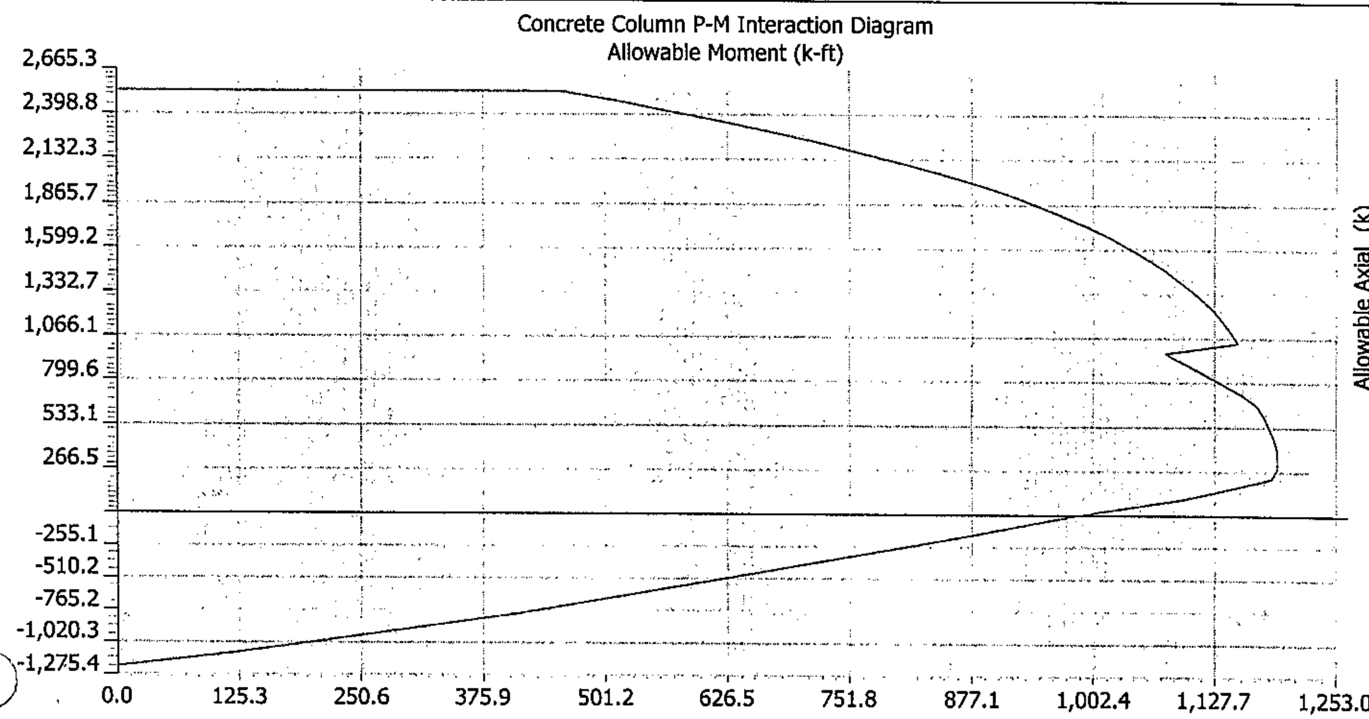


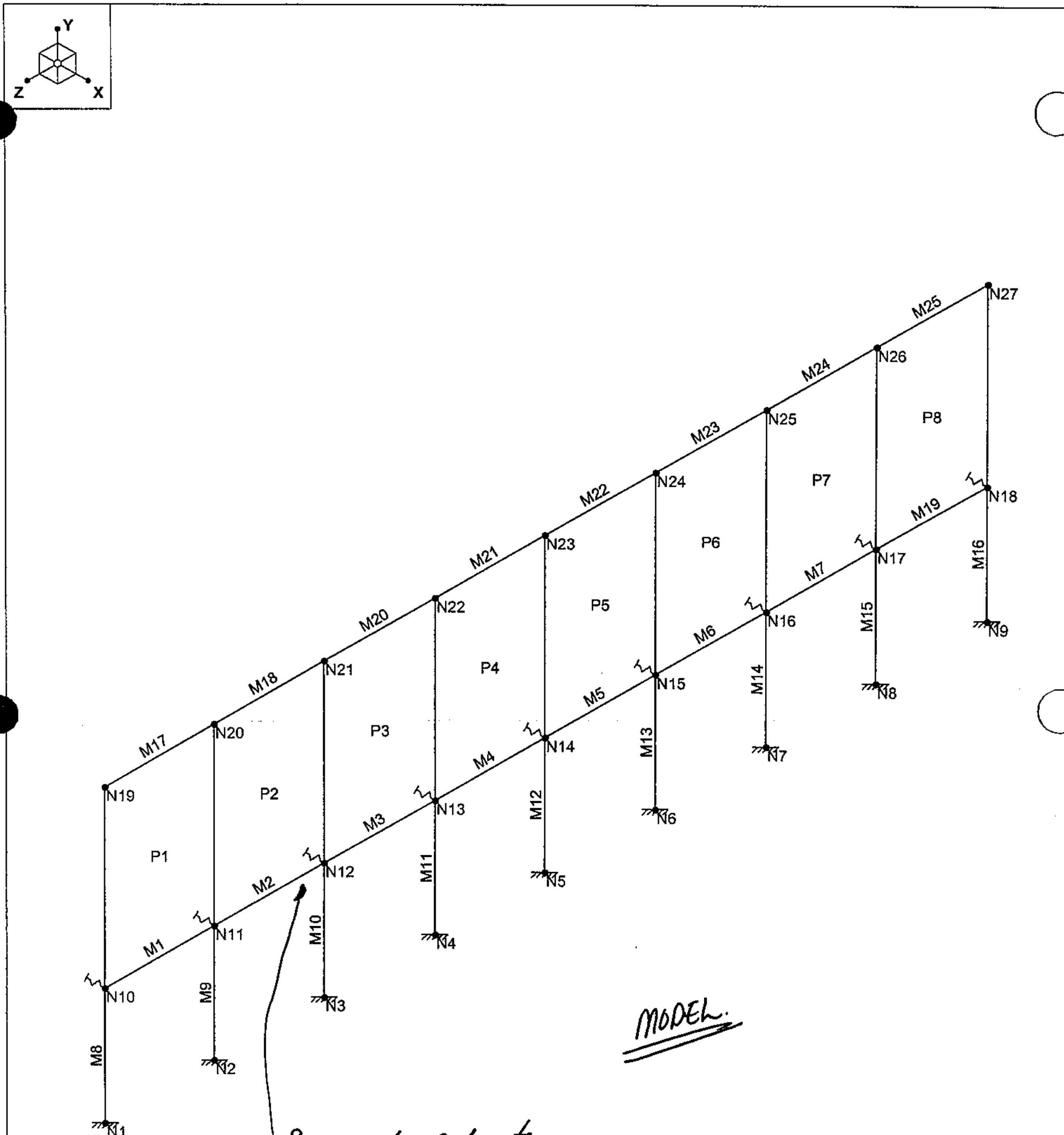
Looking along Y-Y Axis

Concrete Column

Description : --None--

Interaction Diagram



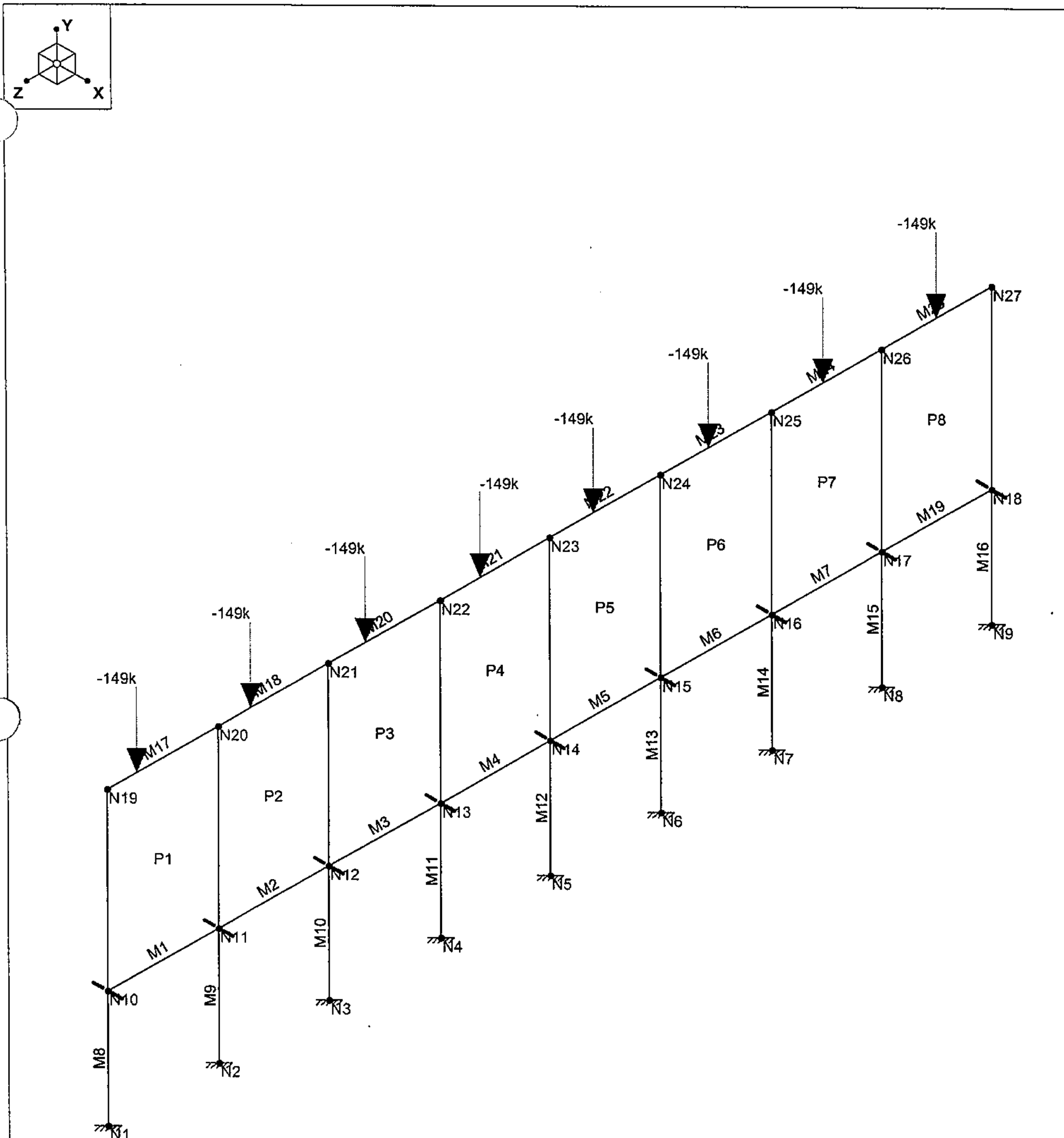


MODEL.

Spring Load Limits Bearing Against the Rock to 2500 psf — OK

Results for LC 1, LOAD COMBO

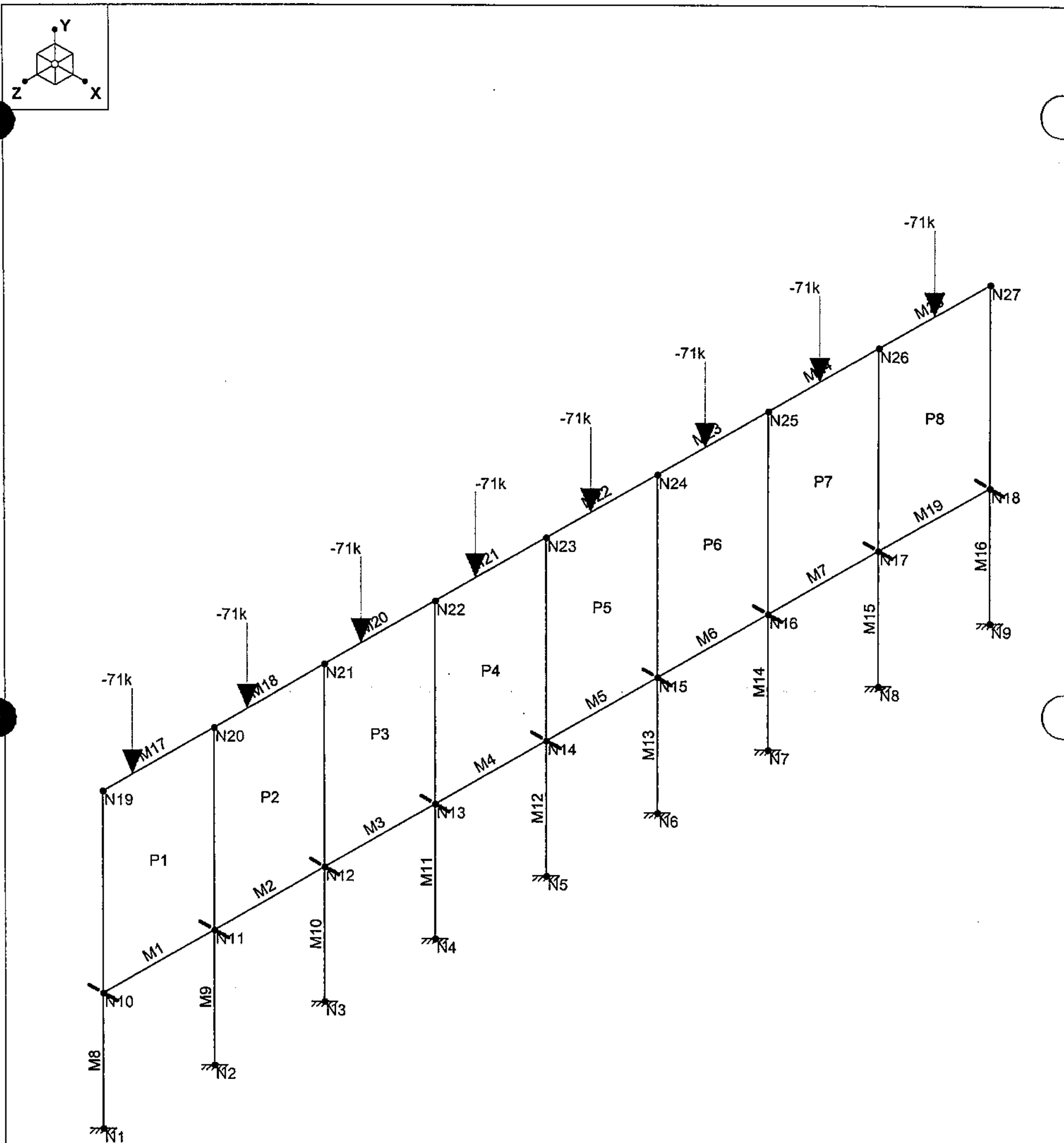
		Jan 28, 2009 at 2:52 PM
		SCI-RR-Forward-Abutment.r3d



Loads: BLC 1, DL BEAM
Results for LC 1, LOAD COMBO

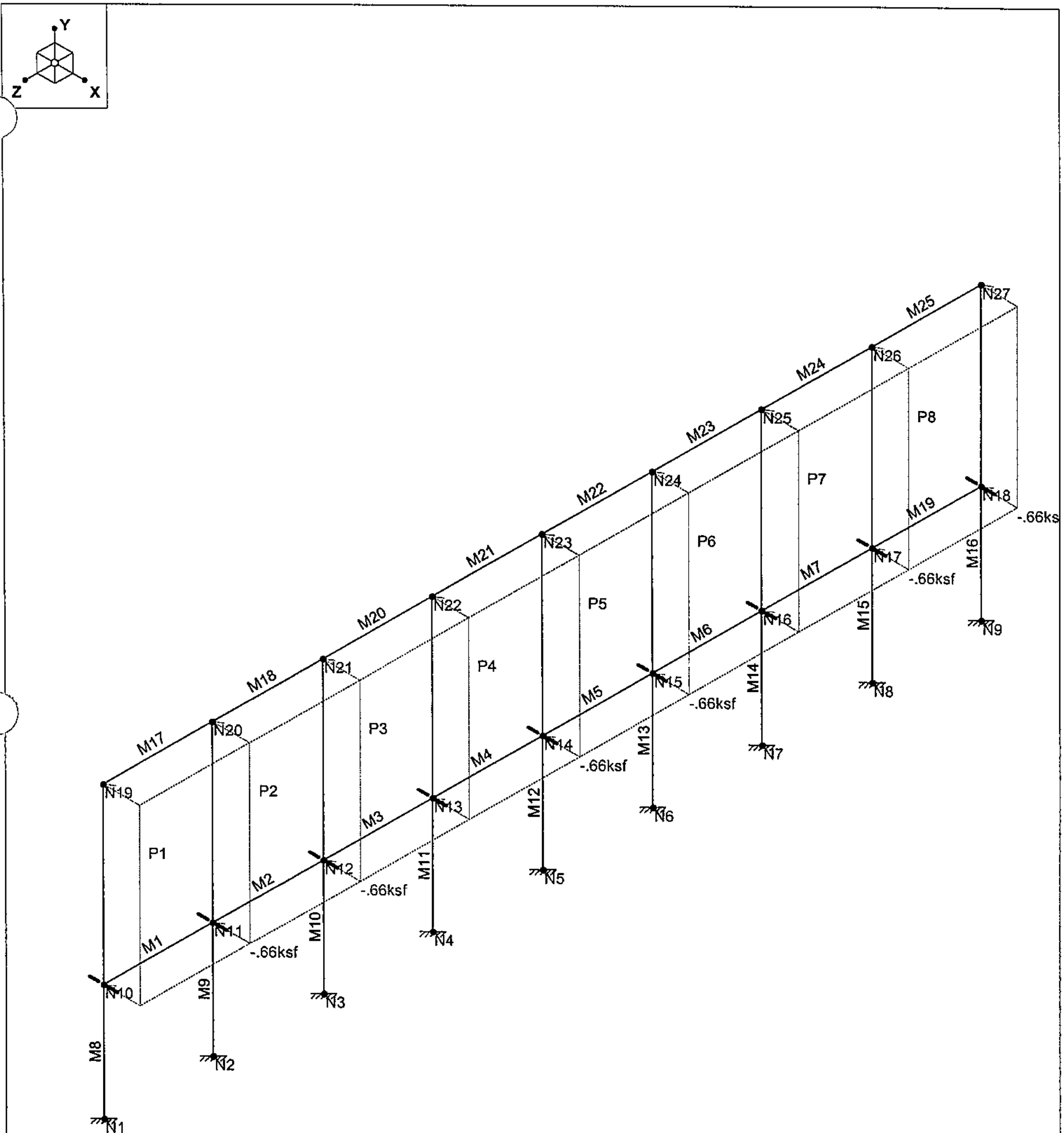
Jan 28, 2009 at 2:34 PM

SCI-RR-Forward-Abutment.r3d



Loads: BLC 2, LL BEAM
Results for LC 1, LOAD COMBO

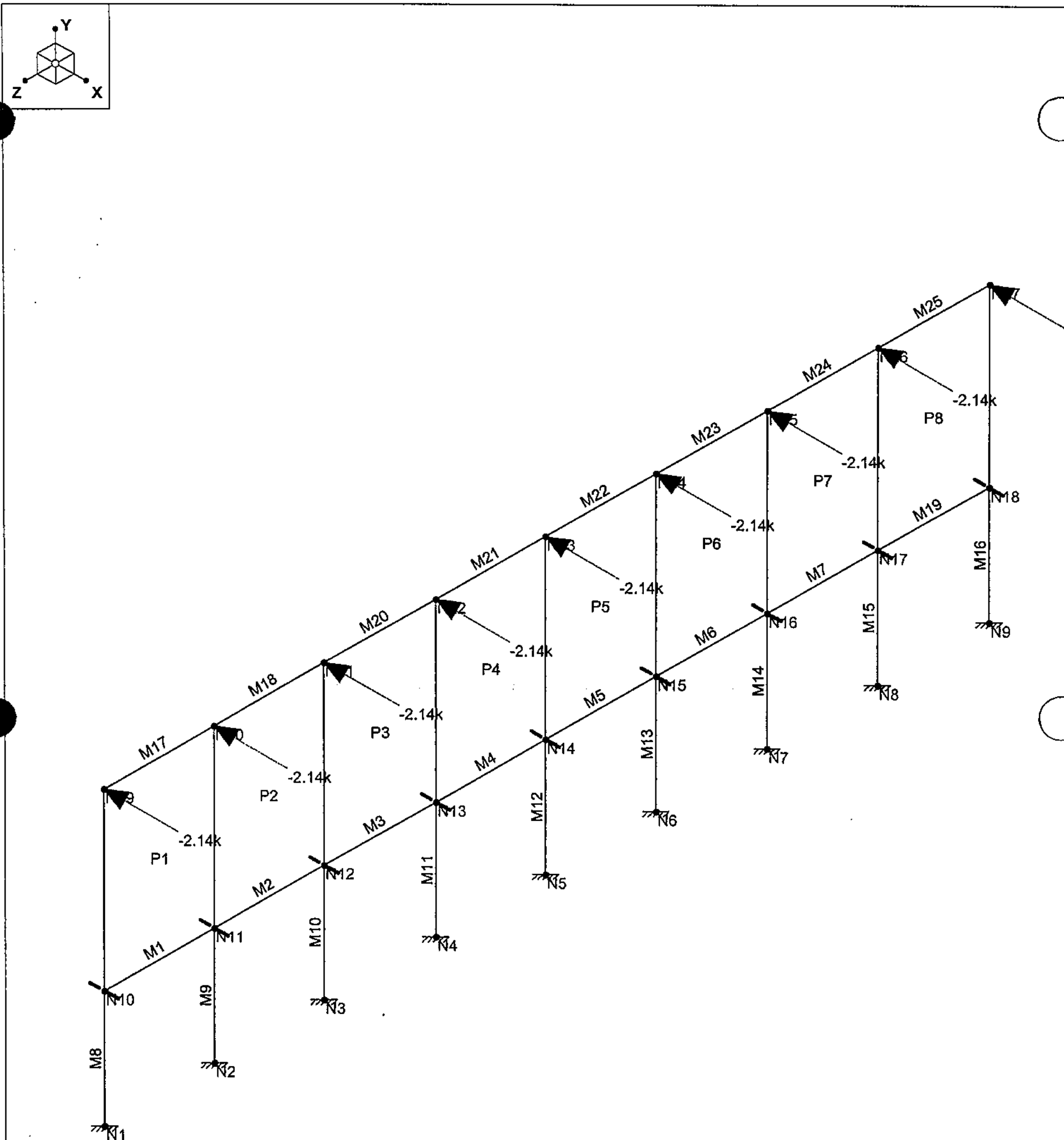
Jan 28, 2009 at 2:34 PM
SCI-RR-Forward-Abutment.r3d



Loads: BLC 3, EARTH
Results for LC 1, LOAD COMBO

Jan 28, 2009 at 2:34 PM

SCI-RR-Forward-Abutment.r3d



Loads: BLC 4, BRAKING FORCE
Results for LC 1, LOAD COMBO

		Jan 28, 2009 at 2:34 PM
		SCI-RR-Forward-Abutment.r3d

Global

Display Sections for Member Calcs	5
Max Internal Sections for Member Calcs	97
Include Shear Deformation	Yes
Include Warping	Yes
Area Load Mesh (in^2)	144
Merge Tolerance (in)	.12
P-Delta Analysis Tolerance	0.50%
Vertical Axis	Y

Hot Rolled Steel Code	AISC: ASD 9th
Cold Formed Steel Code	AISI 99: ASD
Wood Code	NDS 91/97: ASD
Wood Temperature	< 100F
Concrete Code	ACI 2002

Number of Shear Regions	4
Region Spacing Increment (in)	4
Biaxial Column Method	PCA Load Contour
Parme Beta Factor (PCA)	.65
Concrete Stress Block	Rectangular
Use Cracked Sections	Yes
Bad Framing Warnings	No
Unused Force Warnings	Yes

Concrete Properties

	Label	E [ksi]	G [ksi]	Nu	Therm (\1E5 F)	Density[k/ft^3]	f'c[ksi]
1	Conc3000NW	3156	1372	.15	.6	.145	3
2	Conc3500NW	3409	1482	.15	.6	.145	3.5
3	Conc4000NW	3644	1584	.15	.6	.145	4
4	Conc3000LW	2085	907	.15	.6	.11	3
5	Conc3500LW	2252	979	.15	.6	.11	3.5
6	Conc4000LW	2408	1047	.15	.6	.11	4

Concrete Section Sets

	Label	Shape	Type	Design List	Material	Design Rules	A [in2]	Iyy [in4]	Izz [in4]	J [in4]
1	CONC1A	CRECT12X8	Beam	Rectangular	Conc3000N...	Typical	96	512	1152	1187.84

Member Primary Data

	Label	I Joint	J Joint	K Joint	Rotate(deg)	Section/Shape	Type	Design List	Material	Design Rules
1	M1	N10	N11			CRECT36X48	Beam	Rectangular	Conc3000...	Typical
2	M2	N11	N12			CRECT36X48	Beam	Rectangular	Conc3000...	Typical
3	M3	N12	N13			CRECT36X48	Beam	Rectangular	Conc3000...	Typical
4	M4	N13	N14			CRECT36X48	Beam	Rectangular	Conc3000...	Typical
5	M5	N14	N15			CRECT36X48	Beam	Rectangular	Conc3000...	Typical
6	M6	N15	N16			CRECT36X48	Beam	Rectangular	Conc3000...	Typical
7	M7	N16	N17			CRECT36X48	Beam	Rectangular	Conc3000...	Typical
8	M8	N1	N10			CRND36	Beam	Rectangular	Conc3000...	Typical
9	M9	N2	N11			CRND36	Beam	Rectangular	Conc3000...	Typical
10	M10	N3	N12			CRND36	Beam	Rectangular	Conc3000...	Typical
11	M11	N4	N13			CRND36	Beam	Rectangular	Conc3000...	Typical
12	M12	N5	N14			CRND36	Beam	Rectangular	Conc3000...	Typical
13	M13	N6	N15			CRND36	Beam	Rectangular	Conc3000...	Typical
14	M14	N7	N16			CRND36	Beam	Rectangular	Conc3000...	Typical
15	M15	N8	N17			CRND36	Beam	Rectangular	Conc3000...	Typical

Member Primary Data (Continued)

	Label	I Joint	J Joint	K Joint	Rotate(deg)	Section/Shape	Type	Design List	Material	Design Rules
16	M16	N9	N18			CRND36	Beam	Rectangular	Conc3000...	Typical
17	M17	N19	N20			CRECT72X36	Beam	Rectangular	Conc3000...	Typical
18	M18	N20	N21			CRECT72X36	Beam	Rectangular	Conc3000...	Typical
19	M19	N17	N18			CRECT36X48	Beam	Rectangular	Conc3000...	Typical
20	M20	N21	N22			CRECT72X36	Beam	Rectangular	Conc3000...	Typical
21	M21	N22	N23			CRECT72X36	Beam	Rectangular	Conc3000...	Typical
22	M22	N23	N24			CRECT72X36	Beam	Rectangular	Conc3000...	Typical
23	M23	N24	N25			CRECT72X36	Beam	Rectangular	Conc3000...	Typical
24	M24	N25	N26			CRECT72X36	Beam	Rectangular	Conc3000...	Typical
25	M25	N26	N27			CRECT72X36	Beam	Rectangular	Conc3000...	Typical

Member Advanced Data

	Label	I Release	J Release	I Offset[in]	J Offset[in]	T/C Only	Physical	TOM	Inactive
1	M1						Yes		
2	M2						Yes		
3	M3						Yes		
4	M4						Yes		
5	M5						Yes		
6	M6						Yes		
7	M7						Yes		
8	M8						Yes		
9	M9						Yes		
10	M10						Yes		
11	M11						Yes		
12	M12						Yes		
13	M13						Yes		
14	M14						Yes		
15	M15						Yes		
16	M16						Yes		
17	M17						Yes		
18	M18						Yes		
19	M19						Yes		
20	M20						Yes		
21	M21						Yes		
22	M22						Yes		
23	M23						Yes		
24	M24						Yes		
25	M25						Yes		

Joint Coordinates and Temperatures

	Label	X [ft]	Y [ft]	Z [ft]	Temp [F]	Detach From Diap...
1	N1	0	-10	0	0	
2	N2	0	-10	-7.5	0	
3	N3	0	-10	-15	0	
4	N4	0	-10	-22.5	0	
5	N5	0	-10	-30	0	
6	N6	0	-10	-37.5	0	
7	N7	0	-10	-45	0	
8	N8	0	-10	-52.5	0	
9	N9	0	-10	-60	0	
10	N10	0	-2	0	0	
11	N11	0	-2	-7.5	0	
12	N12	0	-2	-15	0	
13	N13	0	-2	-22.5	0	
14	N14	0	-2	-30	0	

Joint Coordinates and Temperatures (Continued)

	Label	X [ft]	Y [ft]	Z [ft]	Temp [F]	Detach From Diap...
15	N15	0	-2	-37.5	0	
16	N16	0	-2	-45	0	
17	N17	0	-2	-52.5	0	
18	N18	0	-2	-60	0	
19	N19	0	10	0	0	
20	N20	0	10	-7.5	0	
21	N21	0	10	-15	0	
22	N22	0	10	-22.5	0	
23	N23	0	10	-30	0	
24	N24	0	10	-37.5	0	
25	N25	0	10	-45	0	
26	N26	0	10	-52.5	0	
27	N27	0	10	-60	0	

Joint Boundary Conditions

	Joint Label	X [k/in]	Y [k/in]	Z [k/in]	X Rot.[k-ft/rad]	Y Rot.[k-ft/rad]	Z Rot.[k-ft/rad]	Footing
1	N1	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction	
2	N2	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction	
3	N3	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction	
4	N4	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction	
5	N5	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction	
6	N6	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction	
7	N7	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction	
8	N8	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction	
9	N10	S250						
10	N11	S250						
11	N12	S250						
12	N13	S250						
13	N14	S250						
14	N15	S250						
15	N16	S250						
16	N17	S250						
17	N18	S250						
18	N9	Reaction	Reaction	Reaction	Reaction	Reaction	Reaction	

Concrete Column Design Parameters

Label	Shape	Length[ft]	Lu-yy[ft]	Lu-zz[ft]	Cm-yy	Cm-zz	Kyy	Kzz	y sway	z sway	Icr Fac...	Flexur...	Shear ...
No Data to Print ...													

Concrete Beam Design Parameters

	Label	Shape	Length[ft]	B-eff Left[in]	B_eff Right[in]	Slab Thic...	Slab Thic...	Icr Factor	Flexural L...	Shear Lay...
1	M1	CRECT36X48	7.5						Default	Default
2	M2	CRECT36X48	7.5						Default	Default
3	M3	CRECT36X48	7.5						Default	Default
4	M4	CRECT36X48	7.5						Default	Default
5	M5	CRECT36X48	7.5						Default	Default
6	M6	CRECT36X48	7.5						Default	Default
7	M7	CRECT36X48	7.5						Default	Default
8	M8	CRND36	8						Default	Default
9	M9	CRND36	8						Default	Default
10	M10	CRND36	8						Default	Default
11	M11	CRND36	8						Default	Default
12	M12	CRND36	8						Default	Default

Concrete Beam Design Parameters (Continued)

	Label	Shape	Length[ft]	B-eff Left[in]	B_eff Right[in]	Slab Thic...	Slab Thic...	Icr Factor	Flexural L...	Shear Lay...
13	M13	CRND36	8						Default	Default
14	M14	CRND36	8						Default	Default
15	M15	CRND36	8						Default	Default
16	M16	CRND36	8						Default	Default
17	M17	CRECT72X36	7.5						Default	Default
18	M18	CRECT72X36	7.5						Default	Default
19	M19	CRECT36X48	7.5						Default	Default
20	M20	CRECT72X36	7.5						Default	Default
21	M21	CRECT72X36	7.5						Default	Default
22	M22	CRECT72X36	7.5						Default	Default
23	M23	CRECT72X36	7.5						Default	Default
24	M24	CRECT72X36	7.5						Default	Default
25	M25	CRECT72X36	7.5						Default	Default

Plate Primary Data

	Label	A Joint	B Joint	C Joint	D Joint	Material	Thickness[in]
1	P1	N10	N11	N20	N19	gen_Conc3NW	36
2	P2	N11	N12	N21	N20	gen_Conc3NW	36
3	P3	N21	N12	N13	N22	gen_Conc3NW	36
4	P4	N13	N14	N23	N22	gen_Conc3NW	36
5	P5	N14	N15	N24	N23	gen_Conc3NW	36
6	P6	N24	N15	N16	N25	gen_Conc3NW	36
7	P7	N16	N17	N26	N25	gen_Conc3NW	36
8	P8	N17	N18	N27	N26	gen_Conc3NW	36

Rigid Diaphragms

Joint Label	Plane	Type	Inactive
	No Data to Print ...		

Joint Loads and Enforced Displacements (BLC 4 : BRAKING FORCE)

	Joint Label	L,D,M	Direction	Magnitude[(k,k-ft), (in,rad), (k*s...
1	N19	L	X	-2.14
2	N20	L	X	-2.14
3	N21	L	X	-2.14
4	N22	L	X	-2.14
5	N23	L	X	-2.14
6	N24	L	X	-2.14
7	N25	L	X	-2.14
8	N26	L	X	-2.14
9	N27	L	X	-2.14

Member Point Loads (BLC 1 : DL BEAM)

	Member Label	Direction	Magnitude[k,k-ft]	Location[ft,%]
1	M17	Y	-149	2
2	M18	Y	-149	2.25
3	M20	Y	-149	2.5
4	M21	Y	-149	2.75
5	M22	Y	-149	3
6	M23	Y	-149	3.25
7	M24	Y	-149	3.5
8	M25	Y	-149	3.75

Member Point Loads (BLC 2 : LL BEAM)

	Member Label	Direction	Magnitude[k,k-ft]	Location[ft,%]
1	M17	Y	-71	2
2	M18	Y	-71	2.25
3	M20	Y	-71	2.5
4	M21	Y	-71	2.75
5	M22	Y	-71	3
6	M23	Y	-71	3.25
7	M24	Y	-71	3.5
8	M25	Y	-71	3.75

Plate Surface Loads (BLC 3 : EARTH)

	Plate Label	Direction	Magnitude[ksf,deg]
1	P1	X	-.66
2	P2	X	-.66
3	P3	X	-.66
4	P4	X	-.66
5	P5	X	-.66
6	P6	X	-.66
7	P8	X	-.66
8	P7	X	-.66

Member Area Loads

Joint A	Joint B	Joint C	Joint D	Direction	Distribution	Magnitude[ksf]
No Data to Print ...						

Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Z Gravity	Joint	Point	Distributed	Area (Me...	Surface (...
1	DL BEAM	None					8			
2	LL BEAM	None					8			
3	EARTH	None								8
4	BRAKING FORCE	None				9				

Load Combinations

	Description	Sol...	PD...	SR...	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor	BLC	Factor
1	LOAD COMBO	Yes			Y	-1	1	1	2	1	3	1	4	1						

Load Combination Design

	Description	ASIF	CD	ABIF	Service	Hot Rolled	Cold Formed	Wood	Concrete	Footings
1	LOAD COMBO					Yes	Yes	Yes	Yes	Yes

Joint Deflections (By Combination)

	LC	Joint Label	X [in]	Y [in]	Z [in]	X Rotation [rad]	Y Rotation [rad]	Z Rotation [rad]
1	1	N1	0	0	0	0	0	0
2	1	N2	0	0	0	0	0	0
3	1	N3	0	0	0	0	0	0
4	1	N4	0	0	0	0	0	0
5	1	N5	0	0	0	0	0	0
6	1	N6	0	0	0	0	0	0
7	1	N7	0	0	0	0	0	0
8	1	N8	0	0	0	0	0	0

Joint Deflections (By Combination) (Continued)

	LC	Joint Label	X [in]	Y [in]	Z [in]	X Rotation [rad]	Y Rotation [rad]	Z Rotation [rad]
9	1	N9	0	0	0	0	0	0
10	1	N10	-.2	-.006	.002	-1.948e-5	1.309e-4	4.288e-3
11	1	N11	-.211	-.008	.001	-8.145e-6	9.817e-5	4.4e-3
12	1	N12	-.217	-.009	0	9.371e-8	5.523e-5	4.45e-3
13	1	N13	-.221	-.009	0	-5.409e-8	2.462e-5	4.48e-3
14	1	N14	-.222	-.009	0	5.658e-7	0	4.49e-3
15	1	N15	-.221	-.009	0	1.404e-6	-2.462e-5	4.48e-3
16	1	N16	-.217	-.008	0	3.422e-6	-5.523e-5	4.45e-3
17	1	N17	-.211	-.008	-.001	1.164e-5	-9.817e-5	4.4e-3
18	1	N18	-.2	-.005	-.002	2.069e-5	-1.309e-4	4.288e-3
19	1	N19	-.854	-.011	0	-7.682e-5	1.809e-4	4.753e-3
20	1	N20	-.868	-.012	0	-1.418e-5	1.431e-4	4.724e-3
21	1	N21	-.88	-.012	0	-2.118e-5	9.776e-5	4.731e-3
22	1	N22	-.886	-.012	0	-1.561e-5	4.73e-5	4.745e-3
23	1	N23	-.888	-.012	0	-1.106e-5	0	4.751e-3
24	1	N24	-.886	-.012	0	-5.497e-6	-4.73e-5	4.745e-3
25	1	N25	-.88	-.012	0	2.532e-6	-9.776e-5	4.731e-3
26	1	N26	-.868	-.011	0	2.88e-6	-1.431e-4	4.724e-3
27	1	N27	-.854	-.009	0	9.385e-5	-1.809e-4	4.753e-3

Joint Reactions (By Combination)

	LC	Joint Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
1	1	N1	-6.301	216.587	-2.824	-9.756	-25.699	-313.775
2	1	N2	-.619	278.941	-1.822	-6.646	-19.279	-345.335
3	1	N3	4.212	292.574	-.949	-3.804	-10.846	-368.619
4	1	N4	6.437	297.641	-.561	-2.241	-4.834	-379.93
5	1	N5	7.08	299.007	-.128	-.557	0	-383.286
6	1	N6	6.437	295.448	.359	1.324	4.834	-379.93
7	1	N7	4.212	284.772	.982	3.656	10.846	-368.619
8	1	N8	-.619	256.361	2.014	7.137	19.279	-345.335
9	1	N10	50.055	0	0	0	0	0
10	1	N11	52.658	0	0	0	0	0
11	1	N12	54.331	0	0	0	0	0
12	1	N13	55.192	0	0	0	0	0
13	1	N14	55.453	0	0	0	0	0
14	1	N15	55.192	0	0	0	0	0
15	1	N16	54.331	0	0	0	0	0
16	1	N17	52.658	0	0	0	0	0
17	1	N18	50.055	0	0	0	0	0
18	1	N9	-6.301	186.666	2.93	10.085	25.699	-313.775
19	1	Totals:	494.46	2407.996	0			
20	1	COG (ft):	X: 0	Y: 8.209	Z: -29.36			

Member Section Forces (By Combination)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
1	1	M1	1	-20.932	8.35	.494	55.901	9.239	12.836
2			2	-20.932	5.088	.494	55.901	10.166	.238
3			3	-20.932	1.825	.494	55.901	11.093	-6.242
4			4	-20.932	-1.437	.494	55.901	12.019	-6.606
5			5	-20.932	-4.7	.494	55.901	12.946	-.852
6	1	M2	1	-23.039	6.658	-.986	25.051	18.27	7.081
7			2	-23.039	3.395	-.986	25.051	16.421	-2.343
8			3	-23.039	.133	-.986	25.051	14.571	-5.651
9			4	-23.039	-3.13	-.986	25.051	12.722	-2.841
10			5	-23.039	-6.392	-.986	25.051	10.872	6.086

Member Section Forces (By Combination) (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
11	1	M3	1	-21.313	6.976	-.723	15.262	13.1	9.875
12			2	-21.313	3.713	-.723	15.262	11.744	-.146
13			3	-21.313	.451	-.723	15.262	10.388	-4.05
14			4	-21.313	-2.812	-.723	15.262	9.032	-1.837
15			5	-21.313	-6.074	-.723	15.262	7.677	6.494
16	1	M4	1	-21.514	6.713	-.29	4.949	9.443	8.743
17			2	-21.514	3.45	-.29	4.949	8.898	-.785
18			3	-21.514	.188	-.29	4.949	8.353	-4.196
19			4	-21.514	-3.075	-.29	4.949	7.809	-1.49
20			5	-21.514	-6.337	-.29	4.949	7.264	7.333
21	1	M5	1	-23.757	6.473	.29	-4.949	7.264	7.8
22			2	-23.757	3.21	.29	-4.949	7.809	-1.277
23			3	-23.757	-.052	.29	-4.949	8.353	-4.238
24			4	-23.757	-3.315	.29	-4.949	8.898	-1.082
25			5	-23.757	-6.577	.29	-4.949	9.443	8.192
26	1	M6	1	-27.664	6.225	.723	-15.262	7.677	6.647
27			2	-27.664	2.963	.723	-15.262	9.032	-1.967
28			3	-27.664	-.3	.723	-15.262	10.388	-4.463
29			4	-27.664	-3.562	.723	-15.262	11.744	-.843
30			5	-27.664	-6.825	.723	-15.262	13.1	8.895
31	1	M7	1	-31.681	6.021	.986	-25.051	10.872	4.698
32			2	-31.681	2.759	.986	-25.051	12.722	-3.533
33			3	-31.681	-.504	.986	-25.051	14.571	-5.647
34			4	-31.681	-3.766	.986	-25.051	16.421	-1.643
35			5	-31.681	-7.029	.986	-25.051	18.27	8.477
36	1	M8	1	216.587	6.301	-2.824	-25.699	9.756	-313.775
37			2	214.537	6.301	-2.824	-25.699	4.108	-326.377
38			3	212.487	6.301	-2.824	-25.699	-1.54	-338.979
39			4	210.437	6.301	-2.824	-25.699	-7.188	-351.581
40			5	208.388	6.301	-2.824	-25.699	-12.836	-364.183
41	1	M9	1	278.941	.619	-1.822	-19.279	6.646	-345.335
42			2	276.891	.619	-1.822	-19.279	3.001	-346.574
43			3	274.841	.619	-1.822	-19.279	-.644	-347.813
44			4	272.792	.619	-1.822	-19.279	-4.289	-349.052
45			5	270.742	.619	-1.822	-19.279	-7.934	-350.291
46	1	M10	1	292.574	-4.212	-.949	-10.846	3.804	-368.619
47			2	290.524	-4.212	-.949	-10.846	1.906	-360.195
48			3	288.474	-4.212	-.949	-10.846	.007	-351.772
49			4	286.425	-4.212	-.949	-10.846	-1.891	-343.349
50			5	284.375	-4.212	-.949	-10.846	-3.789	-334.925
51	1	M11	1	297.641	-6.437	-.561	-4.834	2.241	-379.93
52			2	295.591	-6.437	-.561	-4.834	1.118	-367.057
53			3	293.541	-6.437	-.561	-4.834	-.004	-354.184
54			4	291.491	-6.437	-.561	-4.834	-1.127	-341.311
55			5	289.441	-6.437	-.561	-4.834	-2.249	-328.438
56	1	M12	1	299.007	-7.08	-.128	0	.557	-383.286
57			2	296.957	-7.08	-.128	0	.301	-369.126
58			3	294.907	-7.08	-.128	0	.045	-354.966
59			4	292.857	-7.08	-.128	0	-.211	-340.806
60			5	290.807	-7.08	-.128	0	-.467	-326.646
61	1	M13	1	295.448	-6.437	.359	4.834	-1.324	-379.93
62			2	293.398	-6.437	.359	4.834	-.606	-367.057
63			3	291.348	-6.437	.359	4.834	.111	-354.184
64			4	289.298	-6.437	.359	4.834	.828	-341.311
65			5	287.248	-6.437	.359	4.834	1.546	-328.438
66	1	M14	1	284.772	-4.212	.982	10.846	-3.656	-368.619
67			2	282.722	-4.212	.982	10.846	-1.693	-360.195

Member Section Forces (By Combination) (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
68			3	280.672	-4.212	.982	10.846	.271	-351.772
69			4	278.622	-4.212	.982	10.846	2.234	-343.349
70			5	276.572	-4.212	.982	10.846	4.197	-334.925
71	1	M15	1	256.361	.619	2.014	19.279	-7.137	-345.335
72			2	254.311	.619	2.014	19.279	-3.109	-346.574
73			3	252.261	.619	2.014	19.279	.92	-347.813
74			4	250.211	.619	2.014	19.279	4.949	-349.052
75			5	248.161	.619	2.014	19.279	8.978	-350.291
76	1	M16	1	186.666	6.301	2.93	25.699	-10.085	-313.775
77			2	184.616	6.301	2.93	25.699	-4.224	-326.377
78			3	182.566	6.301	2.93	25.699	1.636	-338.979
79			4	180.516	6.301	2.93	25.699	7.496	-351.581
80			5	178.466	6.301	2.93	25.699	13.357	-364.183
81	1	M17	1	-9.514	143.966	1.016	-27.922	7.02	0
82			2	-9.514	139.072	1.016	-27.922	8.924	-265.348
83			3	-9.514	-85.821	1.016	-27.922	10.828	-136.521
84			4	-9.514	-90.715	1.016	-27.922	12.732	28.982
85			5	-9.514	-95.609	1.016	-27.922	14.636	203.661
86	1	M18	1	.752	166.494	1.226	6.472	8.374	203.661
87			2	.752	161.601	1.226	6.472	10.673	-103.928
88			3	.752	-63.293	1.226	6.472	12.972	-72.341
89			4	.752	-68.187	1.226	6.472	15.271	50.922
90			5	.752	-73.081	1.226	6.472	17.57	183.36
91	1	M19	1	-22.962	4.677	-.494	-55.901	12.946	-.5
92			2	-22.962	1.415	-.494	-55.901	12.019	-6.212
93			3	-22.962	-1.848	-.494	-55.901	11.093	-5.806
94			4	-22.962	-5.11	-.494	-55.901	10.166	.717
95			5	-22.962	-8.373	-.494	-55.901	9.239	13.357
96	1	M20	1	-1.048	154.902	.607	13.699	12.172	183.36
97			2	-1.048	150.009	.607	13.699	13.31	-102.494
98			3	-1.048	-74.885	.607	13.699	14.447	-104.172
99			4	-1.048	-79.779	.607	13.699	15.585	40.826
100			5	-1.048	-84.673	.607	13.699	16.722	194.999
101	1	M21	1	-.928	148.158	.211	5.529	12.75	194.999
102			2	-.928	143.264	.211	5.529	13.146	-78.209
103			3	-.928	-81.63	.211	5.529	13.542	-122.241
104			4	-.928	-86.523	.211	5.529	13.938	35.402
105			5	-.928	-91.417	.211	5.529	14.334	202.221
106	1	M22	1	2.004	141.35	-.211	-5.529	14.334	202.221
107			2	2.004	136.456	-.211	-5.529	13.938	-58.221
108			3	2.004	-88.438	-.211	-5.529	13.542	-144.488
109			4	2.004	-93.332	-.211	-5.529	13.146	25.921
110			5	2.004	-98.225	-.211	-5.529	12.75	205.506
111	1	M23	1	7.308	134.423	-.607	-13.699	16.722	205.506
112			2	7.308	129.529	-.607	-13.699	15.585	-41.95
113			3	7.308	-95.364	-.607	-13.699	14.447	-170.229
114			4	7.308	-100.258	-.607	-13.699	13.31	13.167
115			5	7.308	-105.152	-.607	-13.699	12.172	205.739
116	1	M24	1	12.036	124.073	-1.226	-6.472	17.57	205.739
117			2	12.036	119.179	-1.226	-6.472	15.271	-22.31
118			3	12.036	-105.715	-1.226	-6.472	12.972	-186.182
119			4	12.036	-110.608	-1.226	-6.472	10.673	16.621
120			5	12.036	-115.502	-1.226	-6.472	8.374	228.599
121	1	M25	1	-.805	150.267	-1.016	27.922	14.636	228.599
122			2	-.805	145.374	-1.016	27.922	12.732	-48.564
123			3	-.805	-79.52	-1.016	27.922	10.828	-316.552
124			4	-.805	-84.414	-1.016	27.922	8.924	-162.864

Member Section Forces (By Combination) (Continued)

	LC	Member Label	Sec	Axial[k]	y Shear[k]	z Shear[k]	Torque[k-ft]	y-y Moment[k-...	z-z Moment[k-...
125			5	-.805	-89.308	-1.016	27.922	7.02	0

Concrete Beam Design Results

	Member	Shape	UC Max Top	Loc[ft]	UC Ma...	Loc[ft]	Shear ...	Loc[ft]	Phi*Mnz T...	Phi*Mnz B...	Phi*Vny[k]
1	M1	CRECT36X48	Warning: No desi...								
2	M2	CRECT36X48	Warning: No desi...								
3	M3	CRECT36X48	Warning: No desi...								
4	M4	CRECT36X48	Warning: No desi...								
5	M5	CRECT36X48	Warning: No desi...								
6	M6	CRECT36X48	Warning: No desi...								
7	M7	CRECT36X48	Warning: No desi...								
8	M8	CRND36	Round Shapes fo...								
9	M9	CRND36	Round Shapes fo...								
10	M10	CRND36	Round Shapes fo...								
11	M11	CRND36	Round Shapes fo...								
12	M12	CRND36	Round Shapes fo...								
13	M13	CRND36	Round Shapes fo...								
14	M14	CRND36	Round Shapes fo...								
15	M15	CRND36	Round Shapes fo...								
16	M16	CRND36	Round Shapes fo...								
17	M17	CRECT72X36	.532	7.5	.732	2.031	.479	0	382.623	382.623	300.844
18	M18	CRECT72X36	.532	0	.427	2.266	.553	0	382.623	382.623	300.844
19	M19	CRECT36X48	Warning: No desi...								
20	M20	CRECT72X36	.51	7.5	.512	2.5	.515	0	382.623	382.623	300.844
21	M21	CRECT72X36	.529	7.5	.524	2.734	.492	0	382.623	382.623	300.844
22	M22	CRECT72X36	.537	7.5	.538	3.047	.477	7.5	382.623	382.623	206.115
23	M23	CRECT72X36	.538	7.5	.561	3.281	.498	6.563	382.623	382.623	206.115
24	M24	CRECT72X36	.597	7.5	.551	3.516	.412	0	382.623	382.623	300.844
25	M25	CRECT72X36	.597	0	.577	3.75	.499	0	382.623	549.077	300.844

Concrete Column Design Results

Column	Shape	UC Max	Loc[ft]	UC LC	Shear ...	Loc[ft]	Dir	Phi used	Pn[k]	Mny[k-ft]	Mnz[k-ft]	Vny[k]	Vnz[k]
No Data to Print ...													

Concrete Beam Bending Reinforcement

	Member	Shape	Span	Left Top	Left Bot	Mid Top	Mid Bot	Right Top	Right Bot
1	M1	CRECT36X48	1						
2	M2	CRECT36X48	1						
3	M3	CRECT36X48	1						
4	M4	CRECT36X48	1						
5	M5	CRECT36X48	1						
6	M6	CRECT36X48	1						
7	M7	CRECT36X48	1						
8	M8	CRND36	1						
9	M9	CRND36	1						
10	M10	CRND36	1						
11	M11	CRND36	1						
12	M12	CRND36	1						
13	M13	CRND36	1						
14	M14	CRND36	1						
15	M15	CRND36	1						
16	M16	CRND36	1						
17	M17	CRECT72X36	1	4 #5			4 #5	4 #5	

Concrete Beam Bending Reinforcement (Continued)

	Member	Shape	Span	Left Top	Left Bot	Mid Top	Mid Bot	Right Top	Right Bot
18	M18	CRECT72X36	1	4 #5			4 #5	4 #5	
19	M19	CRECT36X48	1						
20	M20	CRECT72X36	1	4 #5			4 #5	4 #5	
21	M21	CRECT72X36	1	4 #5			4 #5	4 #5	
22	M22	CRECT72X36	1	4 #5			4 #5	4 #5	
23	M23	CRECT72X36	1	4 #5			4 #5	4 #5	
24	M24	CRECT72X36	1	4 #5			4 #5	4 #5	
25	M25	CRECT72X36	1	4 #5			4 #6	4 #5	

Concrete Beam Shear Reinforcement

	Member	Span	Region 1	Region 2	Region 3	Region 4
1	M1	1				
2	M2	1				
3	M3	1				
4	M4	1				
5	M5	1				
6	M6	1				
7	M7	1				
8	M8	1				
9	M9	1				
10	M10	1				
11	M11	1				
12	M12	1				
13	M13	1				
14	M14	1				
15	M15	1				
16	M16	1				
17	M17	1	2 #4 @13in			
18	M18	1	2 #4 @13in			
19	M19	1				
20	M20	1	3 #4 @13in			
21	M21	1	3 #4 @13in			
22	M22	1	3 #4 @13in			
23	M23	1	3 #4 @13in			1 #4 @13in
24	M24	1	7 #4 @13in			
25	M25	1	4 #4 @13in			

Concrete Column Bending Reinforcement

Column	Shape	Span	Perim Bars
No Data to Print ...			

Concrete Column Shear Reinforcement

Column	Span	Region 1	Region 2	Region 3	Region 4
No Data to Print ...					

Plate Principal Stresses

	LC	Plate Label	Loc	Sigma1[ksi]	Sigma2[ksi]	Tau Max[ksi]	Angle[rad]	Von Mises[ksi]
1	1	P1	T	-.005	-.251	.123	.108	.249
2			B	.061	-.008	.034	1.752	.065
3	1	P2	T	.006	-.195	.1	.07	.198
4			B	.043	-.02	.031	1.839	.056
5	1	P3	T	.005	-.183	.094	1.631	.186

Plate Principal Stresses (Continued)

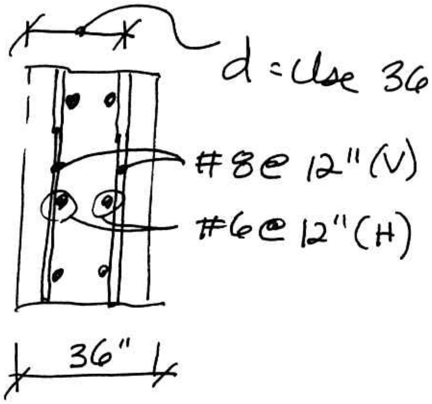
	LC	Plate Label	Loc	Sigma1[ksi]	Sigma2[ksi]	Tau Max[ksi]	Angle[rad]	Von Mises[ksi]
6			B	.03	-.018	.024	.257	.042
7	1	P4	T	.002	-.18	.091	.025	.181
8			B	.023	-.013	.018	1.667	.031
9	1	P5	T	.002	-.18	.091	-.012	.181
10			B	.023	-.014	.018	1.411	.032
11	1	P6	T	.005	-.182	.094	1.521	.185
12			B	.032	-.019	.025	-.29	.044
13	1	P7	T	.006	-.194	.1	-.068	.197
14			B	.044	-.019	.032	1.299	.056
15	1	P8	T	-.002	-.237	.117	-.134	.236
16			B	.075	-.005	.04	1.477	.078

Plate Forces (per ft) (By Combination)

	LC	Plate Label	Qx[k]	Qy[k]	Mx[k-ft]	My[k-ft]	Mxy[k-ft]	Fx[k]	Fy[k]	Fxy[k]
1	1	P1	.867	-5.506	-.245	-33.187	4.156	-2.772	-40.864	3.041
2	1	P2	.181	-3.48	2.157	-25.114	3.25	-2.244	-33.456	-.448
3	1	P3	-4.049	.039	-22.618	2.049	-2.49	-33.594	-2.33	.125
4	1	P4	-.035	-4.089	1.554	-21.825	.868	-2.383	-34.049	.265
5	1	P5	.035	-4.089	1.554	-21.825	-.868	-2.336	-33.974	.771
6	1	P6	-4.049	-.039	-22.618	2.049	2.49	-33.303	-2.19	-.992
7	1	P7	-.181	-3.48	2.157	-25.114	-3.25	-2.049	-33.072	.605
8	1	P8	-.867	-5.506	-.245	-33.187	-4.156	-2.225	-34.114	-5.101

• Wall Check / Bending: $M_o = 33.2$ k/Lf

Use $M_u = 33.2(1.6) = 53.2$ k-ft/Lf.



d = Use 36" − 3" − ½" = 32.5"

$$\rho = \frac{A_s}{bd} = \frac{0.79}{(12)(32.5")} = .002026$$

$$R_N = (60,4) = 117.1$$

$$\frac{M_u}{\phi b d^2} = 117.1 \text{ psi} ; \phi = 0.9, b = 12, d = 32.5"$$

$$M_u = \frac{(117.1)(0.9)(12)(32.5)^2 = 1.3358 \times E^6 \text{ \#-in}}{12000}$$

$$\boxed{M_u = 111.3 \text{ k-ft/Lf}}$$

KZF DESIGN Architecture | Engineering | Interiors | Planning

project name: SCI-823 TR 234 @ CSXT-RR

project #: #5355

date: 2/09

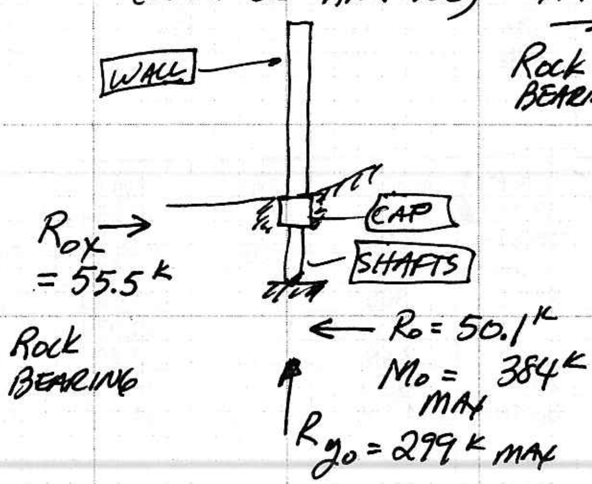
subject: DRILLED SHAFT FOOTING

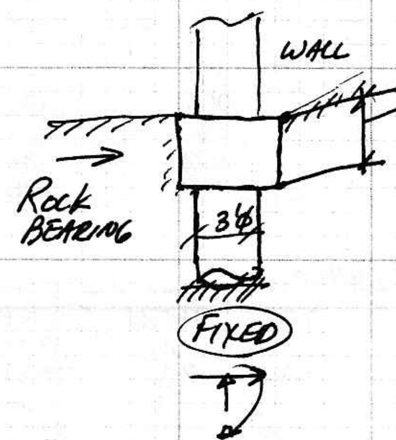
designed by: DGP

checked by: D.A.

SERVICE REACTIONS:

(RISA 3D ANALYSIS)





$$q_{Rock} = \frac{55.5^k}{(3\ FT)(7.5'\ \text{SHAFT SPACING})} = 2.467\ ksf$$

q_{Rock} Applied Bearing from CAP $= 2{,}467$ psf. OK

R_u Shear $= (50.1)(1.6) = 80.2$ kips ✓

M_u MAX $= 384\,(1.6) = 614.4$ k-ft ✓

$R_{yu} = 300^k(1.6) = 480$ k AXIAL ✓

FINAL ANALYSIS
3'Ø SHAFTS @ 7'-6" O.C. W/ (8) #14 BARS

{ALT. USE (12) #11 Bars}

Title Block Line 1
You can changes this area
using the "Settings" menu item
and then using the "Printing &
Title Block" selection.
Title Block Line 6

Title :
Dsgnr:
Project Desc.:

Project Notes :

Job #

Printed: 4 FEB 2009, 11:28AM

Concrete Column

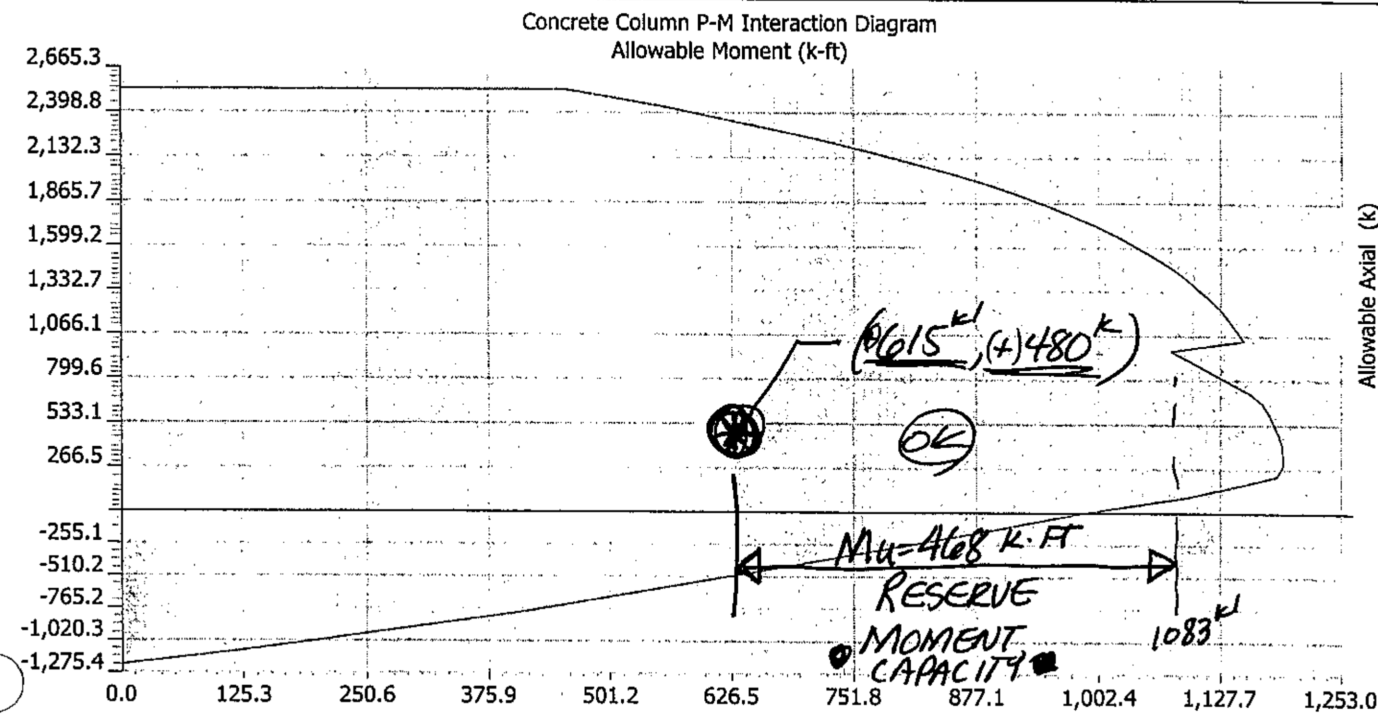
File: c:\Documents and Settings\dflynn.KZFDESIGN.000\My Documents\ENERCALC Data Files\flynns-files.ec6
ENERCALC, INC. 1983-2008, Ver: 6.0.21

Lic. # : KW-06004130

License Owner : kzf, inc.

Description : --None--

Interaction Diagram



36"Ø Conc. Col. w/ (8) #14 Bars

<Alt. 36"Ø w/ (12) #11 Bars>

$$\frac{\text{Mu Capacity}}{\text{Mu Applied}} = \frac{1083}{615} = 1.76 \text{ OK}$$

Title Block Line 1
You can changes this area
using the "Settings" menu item
and then using the "Printing &
Title Block" selection.
Title Block Line 6

Title :
Dsgnr:
Project Desc.:

Project Notes :

Job #

Printed: 4 FEB 2009, 11:28AM

Concrete Column

File: c:\Documents and Settings\dflynn.KZFDESIGN.000\My Documents\ENERCALC Data Files\flynns-files.ec
ENERCALC, INC. 1983-2008, Ver: 6.0.2

Lic. # : KW-06004130 **License Owner : kzf, inc.**

Description : --None--

General Information

Code Ref : 2006 IBC, ACI 318-05

f'c : Concrete 28 day strength	=	4.0 ksi
E =	=	3,122.0 ksi
Density	=	145.0 pcf
β	=	0.850
Fy - Main Rebar	=	60.0 ksi
E - Main Rebar	=	29,000.0 ksi
Allow. Reinforcing Limits	*ASTM A615 Bars Used*	
Min. Reinf.	=	1.0 %
Max. Reinf.	=	8.0 %

Load Combination :2006 IBC & ASCE 7-05

Overall Column Height = 10.0 ft
End Fixity Top Free, Bottom Fixed
ACI Code Year ACI 318-05

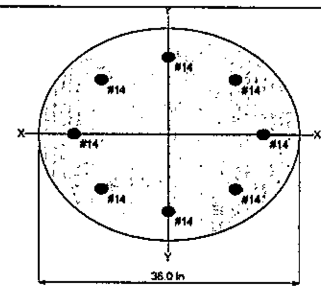
Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for X-X Axis buckling = 10ft, K = 2
Y-Y (depth) axis :Fully braced against buckling along Y-Y Axis

Type of Stirrups used : Spirals
Fy - Stirrups = 40.0 ksi
E - Stirrups = 29,000.0 ksi

Column Cross Section

Column Dimensions :36.0in Diameter, Column Edge to Rebar Edge
Cover = 4.0in

Column Reinforcing : 8 - #14 bars



Applied Loads

Entered loads are factored per load combinations specified by us

Column self weight included : 10,249.5 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 10.0 ft above base, Yecc = 3.50in, D = 300.0, L = 186.0 k
BENDING LOADS . . .
Lat. Point Load at 10.0 ft creating My-y, H = 38.40 k

DESIGN SUMMARY

Maximum Stress Ratio **0.55579 : 1**

Load Combination +1.20D+0.50Lr+1.60L+1.60H
Location of max.above base 0.0 ft
At Pn = Pu, Load Contour location values are . . .

Pu =	669.899 k	φ * Pn =	669.899 k
		Mux-Muy Angle =	73.0 deg
		Mu at Angle =	643.642 k-ft
		Phi*Mn at Angle =	1,158.06 k-ft
Mu-x =	191.80 k-ft	φ * Mn-x =	1,119.59 k-ft
Mu-y =	-614.40 k-ft	φ * Mn-y =	296.024 k-ft

Maximum SERVICE Load Reactions . .

Top along Y-Y	0.0 k	Bottom along Y-Y	38.40 k
Top along X-X	0.0 k	Bottom along X-X	0.0 k

Maximum SERVICE Load Deflections . . .

Along Y-Y -0.029174 in at 10.0 ft above base
for load combination : D Only
Along X-X 0.0 in at 0.0 ft above base
for load combination : H Only

Column Capacities . . .

Pnmax : Nominal Max. Compressive Axial Capacity	4,479.58 k
Pnmin : Nominal Min. Tension Axial Capacity	-1,080.0 k
φ Pn, max : Usable Compressive Axial Capacity	2,665.35 k
φ Pn, min : Usable Tension Axial Capacity	-756.0 k

General Section Information . φ = 0.70 β = 0.850 θ = 0.850

ρ : % Reinforcing 1.7684 % Rebar % Ok
Reinforcing Area 18.0 in^2
Concrete Area 1,017.88 in^2

Governing Load Combination Results

Governing Factored Load Combination	Dist. from base ft	Axial Load Analysis k Pu	φ * Pn	Pu/φPn	Dist. from base ft	Bending Analysis k-ft δx * Mux	φ Mnx	δy * Muy	φ Mny	Mu / φ *Mn at Mx-My Angle
+1.40D	0.00	434.35	434.35	**1.000**	0.00	122.50	1,191.43			**0.10**
+1.20D+0.50Lr+1.60L+1.60H	0.00	669.90	669.90	**1.000**	0.00	191.80	1,119.59	-614.40	296.02	**0.556**
+1.20D+0.50L+0.50S+1.60H	0.00	465.30	465.30	**1.000**	0.00	132.13	1,159.50	-614.40	249.38	**0.530**

Concrete Column

File: c:\Documents and Settings\dflynn.KZFDESIGN.000\My Documents\ENERCALC Data Files\flynns-files.ec6
ENERCALC, INC. 1983-2008, Ver: 6.0.21

Description : –None–

Governing Load Combination Results

Governing Factored Load Combination	Dist. from base ft	Axial Load Analysis k Pu	φ * Pn	Pu/ φPn	Dist. from base ft	Bending Analysis k-ft δx * Mux	φ Mnx	δy * Muy	φ Mny	Mu / φ *Mn at Mx-My Angle
+1.20D+1.60Lr+0.50L	0.00	465.30	465.30	**1.000**	0.00	132.13	1,190.01			**0.111**
+0.90D+1.60W+1.60H	0.00	279.22	279.22	**1.000**	0.00	78.75	1,184.48	-614.40	137.54	**0.519**
+0.90D+E+1.60H	0.00	279.22	279.22	**1.000**	0.00	78.75	1,184.48	-614.40	137.54	**0.519**

Maximum Reactions - Unfactored

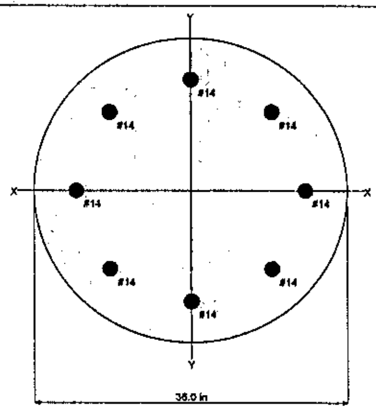
Note: Only non-zero reactions are listed.

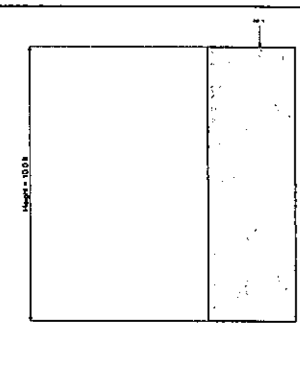
Load Combination	Reaction along X-X Axis @ Base	@ Top	Reaction along Y-Y Axis @ Base	@ Top
D Only				
L Only				
H Only			38.400	

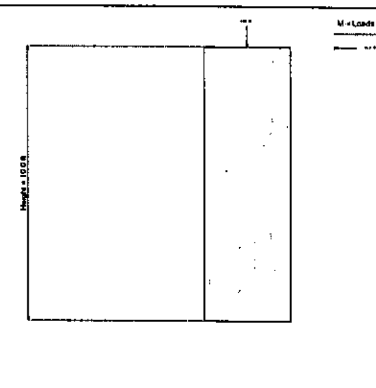
Maximum Deflections for Load Combinations - Unfactored Loads

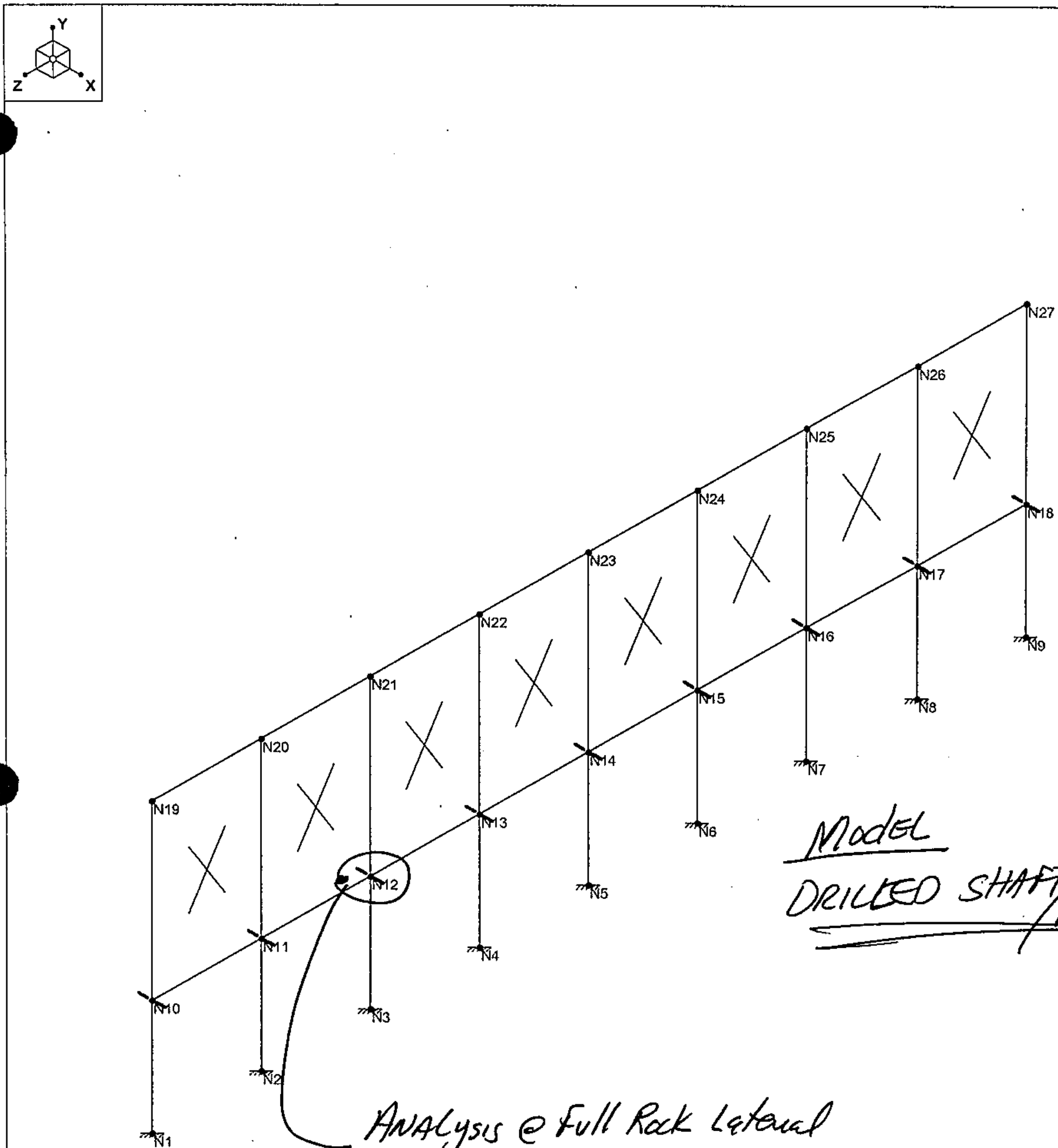
Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
D Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0846 in	9.933 ft	0.000 in	0.000 ft

Sketches









Model
DRILLED SHAFT/PIER WALL

Analysis @ Full Rock Lateral Support, R_x Now goes to 127.4^k & Column (36"φ Drilled Shaft) Decreases alot, to MAX $M_o = \underline{165.4^{k'}}$

Rock Bearing Lateral $= \dfrac{127.4^k}{3' \text{ High Cap} \times 7.75'} = 5.48$ ksf.

= 5500 psf Lateral Pressure. OK

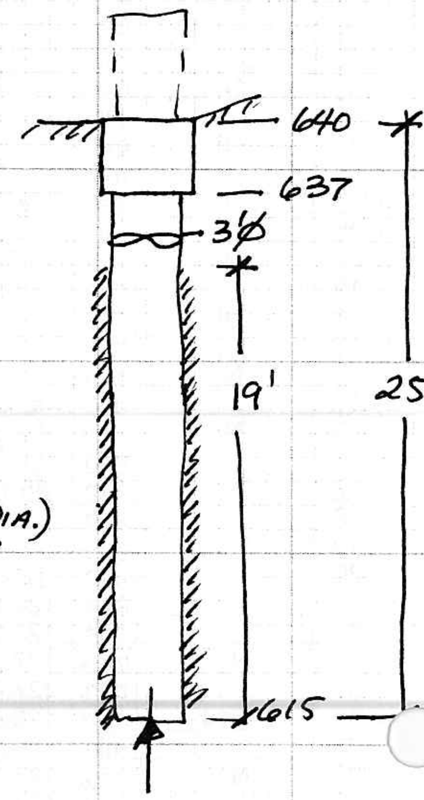
Solution: Envelope

Feb 4, 2009 at 1:38 PM

SCI-RR-Forward-Abutment.r3d

Envelope Joint Reactions

	Joint		X [k]	lc	Y [k]	lc	Z [k]	lc	MX [k-ft]	lc	MY [k-ft]	lc	MZ [k-ft]	lc
1	N1	max	-57.626	1	216.587	1	-2.824	1	-9.756	1	-1.704	1	146.199	1
2		min	-57.626	1	216.587	1	-2.824	1	-9.756	1	-1.704	1	146.199	1
3	N2	max	-62.653	1	278.941	1	-1.822	1	-6.646	1	.016	1	158.952	1
4		min	-62.653	1	278.941	1	-1.822	1	-6.646	1	.016	1	158.952	1
5	N3	max	-64.271	1	292.574	1	-.949	1	-3.804	1	-.347	1	163.058	1
6		min	-64.271	1	292.574	1	-.949	1	-3.804	1	-.347	1	163.058	1
7	N4	max	-64.976	1	297.641	1	-.561	1	-2.241	1	-.069	1	164.847	1
8		min	-64.976	1	297.641	1	-.561	1	-2.241	1	-.069	1	164.847	1
9	N5	max	-65.17	1	299.007	1	-.128	1	-.557	1	0	1	165.34	1
10		min	-65.17	1	299.007	1	-.128	1	-.557	1	0	1	165.34	1
11	N6	max	-64.976	1	295.448	1	.359	1	1.324	1	.069	1	164.847	1
12		min	-64.976	1	295.448	1	.359	1	1.324	1	.069	1	164.847	1
13	N7	max	-64.271	1	284.772	1	.982	1	3.656	1	.347	1	163.058	1
14		min	-64.271	1	284.772	1	.982	1	3.656	1	.347	1	163.058	1
15	N8	max	-62.653	1	256.361	1	2.014	1	7.137	1	-.016	1	158.952	1
16		min	-62.653	1	256.361	1	2.014	1	7.137	1	-.016	1	158.952	1
17	N10	max	85.132	1	0	1	0	1	0	1	0	1	0	1
18		min	85.132	1	0	1	0	1	0	1	0	1	0	1
19	N11	max	129.72	1	0	1	0	1	0	1	0	1	0	1
20		min	129.72	1	0	1	0	1	0	1	0	1	0	1
21	N12	max	123.895	1	0	1	0	1	0	1	0	1	0	1
22		min	123.895	1	0	1	0	1	0	1	0	1	0	1
23	N13	max	127.339	1	0	1	0	1	0	1	0	1	0	1
24		min	127.339	1	0	1	0	1	0	1	0	1	0	1
25	N14	max	126.512	1	0	1	0	1	0	1	0	1	0	1
26		min	126.512	1	0	1	0	1	0	1	0	1	0	1
27	N15	max	127.339	1	0	1	0	1	0	1	0	1	0	1
28		min	127.339	1	0	1	0	1	0	1	0	1	0	1
29	N16	max	123.895	1	0	1	0	1	0	1	0	1	0	1
30		min	123.895	1	0	1	0	1	0	1	0	1	0	1
31	N17	max	129.72	1	0	1	0	1	0	1	0	1	0	1
32		min	129.72	1	0	1	0	1	0	1	0	1	0	1
33	N18	max	85.132	1	0	1	0	1	0	1	0	1	0	1
34		min	85.132	1	0	1	0	1	0	1	0	1	0	1
35	N9	max	-57.626	1	186.666	1	2.93	1	10.085	1	1.704	1	146.199	1
36		min	-57.626	1	186.666	1	2.93	1	10.085	1	1.704	1	146.199	1
37	Totals:	max	494.46	1	2407.996	1	0	1						
38		min	494.46	1	2407.996	1	0	1						



DRILLED Shaft BEARING:

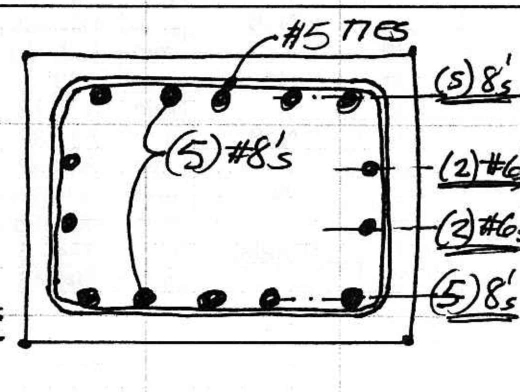
- THE 36"Ø DRILLED Shaft has Plenty of BEARING CAPACITY.

- BRG END POINT $= \pi R^2 (80\ ksf)$
  $= (3.1416)(1.5)^2(80) = \underline{565\ kips}$.

- SIDE WALL/SHEAR STRESS $= (7500\ psf)(2\pi r * h)$
  $= (7.5)(2 * 3.1416)(1.5)(\underbrace{25' - 3' - 3'\ DIA.}_{19'})$
  $= \underline{1343\ kips}$

- TOTAL BRG CAPACITY $= 565 + 1343 = 1908^{kips}$

OK



- CHECK CAP BM SPANING BETWEEN DRILLED SHAFTS @ 7.5' O.C.

$P_u \cong 460^k/\text{SHAFT}$, $w_u = \frac{460}{7.75'} \cong 60\ klf$.

$M_u = \frac{wL^2}{8} = \frac{60(7.75)^2}{8} = 451^{k'}$

$R_u = \frac{M_u}{\phi b d^2} = \frac{451(12000)}{(0.9)(48")(31.5)^2} = 127$

$d = 36" - 3" - \frac{1}{2}" - \frac{1}{2}" = 32"$

Use $d = 31.5"$

$\rho = \langle f'_c = 4,\ f_y = 60 \rangle = 0.0022$

$A_s = \rho b d = (.0022)(48)(31.5) = 3.33\ in^2$

(5) #8 Bars $= (5)(.79) = 3.95\ in^2$ OK

$V_u = 460^k$, $\phi V_c = (.85)(2)\sqrt{f'_c}(b)(d) = 162.5^k$

$S_{MIN} = \frac{(2)(\#5's)(f_y)}{(50)(b_w = 48)} = 15.5"$

$\phi V_c\ (WALL) = .85(2)\sqrt{f'_c}(b)(d)$

$= (.85)(2)\sqrt{4000}(36)(144)/1000 = 557^k$

$\underline{}$ 719.5 kips. OK

Use #5 Ties @ 12"

Title Block Line 1
You can changes this area
using the "Settings" menu item
and then using the "Printing &
Title Block" selection.
Title Block Line 6

Title :
Dsgnr:
Project Desc.:

Project Notes :

Job #

Printed: 4 FEB 2009, 2:34PM

Pole Embedded in Soil

File: c:\Documents and Settings\dflynn.KZFDESIGN.000\My Documents\ENERCALC Data Files\flynns-files.ec6
ENERCALC, INC. 1983-2008, Ver: 6.0.21

Lic. # : KW-06004130

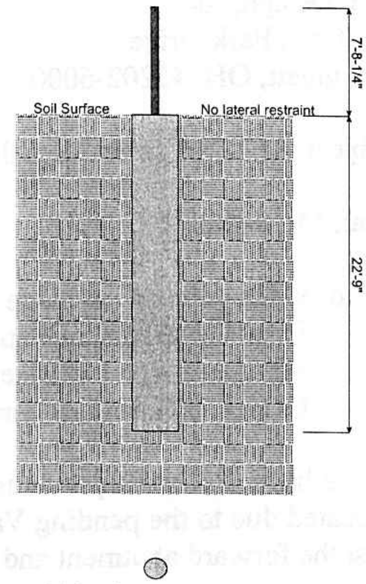
License Owner : kzf, inc.

Description : Preliminary Analysis: Drilled Shaft

General Information

Code References: 2006 IBC 1805.7.2, 1997 UBC 1806.8.2.1

Pole Shape	Circular
Pole Diameter	36 in
Calculate Min. Depth for Allowable Pressures	
No Lateral Restraint at Ground Surface	
Allow Passive	500.0 pcf
Max Passive	5,000.0 psf



Pole Cross Section, Diameter = 3'-0"

Controlling Values

Governing Load Combination :	+D+L+H
Lateral Load	80.0 k
Moment	615.20 k-ft
NO Ground Surface Restraint	
Pressures at 1/3 Depth	
Actual	**3,773.54** psf
Allowable	**3,775.75** psf
Minimum Required Depth	**22.750 ft**
Footing Base Area	7.0686 ft^2
Maximum Soil Pressure	0.0 ksf

Assumes pole is square

• Use 25' Per Soils Report. ✓

Applied Loads

Lateral Concentrated Load		Lateral Distributed Load	Applied Moment	Vertical Load
D : Dead Load	k	k/ft	k-ft	k
Lr : Roof Live	k	k/ft	k-ft	k
L : Live	k	k/ft	k-ft	k
S : Snow	k	k/ft	k-ft	k
W : Wind	k	k/ft	k-ft	k
E : Earthquake	k	k/ft	k-ft	k
H : Lateral Earth	80.0 k	k/ft	k-ft	k
Load distance above Base	7.690 ft	TOP of Load above ground ft		
		BOTTOM of Load above ground ft		

• Preliminary Calc. Lateral Embed Calc.

Load Combination Results

Load Combination	Forces @ Ground Surface Loads - (k)	Moments - (ft-k)	Required Depth - (ft)	Pressure at 1/3 Depth Actual - (psf)	Allow - (psf)
+D+L+H	80.0	615.2	22.75	3,773.5	3,775.8
+D+Lr+H	80.0	615.2	22.75	3,773.5	3,775.8
+D+S+H	80.0	615.2	22.75	3,773.5	3,775.8
+D+0.750Lr+0.750L+H	80.0	615.2	22.75	3,773.5	3,775.8
+D+0.750L+0.750S+H	80.0	615.2	22.75	3,773.5	3,775.8
+D+W+H	80.0	615.2	22.75	3,773.5	3,775.8
+D+0.70E+H	80.0	615.2	22.75	3,773.5	3,775.8
+D+0.750Lr+0.750L+0.750W+H	80.0	615.2	22.75	3,773.5	3,775.8
+D+0.750L+0.750S+0.750W+H	80.0	615.2	22.75	3,773.5	3,775.8
+D+0.750Lr+0.750L+0.5250E+H	80.0	615.2	22.75	3,773.5	3,775.8
+D+0.750L+0.750S+0.5250E+H	80.0	615.2	22.75	3,773.5	3,775.8
+0.60D+W+H	80.0	615.2	22.75	3,773.5	3,775.8
+0.60D+0.70E+H	80.0	615.2	22.75	3,773.5	3,775.8

HDR ONE COMPANY Many Solutions

December 19, 2007

David A. Tomley, P.E.
Associate Director & Chief Bridge Engineer
KZF Design, Inc.
655 Eden Park Drive
Cincinnati, OH 45202-6000

Subject: **SCI-283 (Portsmouth By-Pass) – Shumway Hollow Road Bridge over CSXT**

Dear, Mr. Tomley:

Enclosed for your use in Stage 2 Design of the subject structure are the following documents:

1. Final Geotechnical Report (10/1/07)
2. Revised Stage 1 Bridge Plans
3. ODOT OSE review comments regarding the Geotechnical Report (11/5/07)

As we have discussed previously, the right turn lane and soldier pile wall along SR 335 may be relocated due to the pending Value Engineering Report. We recommend suspending design of at least the forward abutment and approach until this item is resolved by ODOT.

Additionally, I have concerns regarding Item 4 in OSE's comments regarding the design of the forward abutment. Specifically, the recommended use of passive pressure from soil behind the rear abutment to restrain lateral translation of the drilled shafts in the forward abutment. With the forward abutment fixed, the rear abutment will push soil back during thermal cycles. During the thermal contraction cycles of the superstructure, there is potential for the pier to "ratchet" forward instead of the soil recovering behind the rear abutment. Further, during future rehabilitations the passive restraint will be removed temporarily. Eliminating the use of the rear abutment soil pressure, or reducing to active pressure may be a better solution.

Please feel free to contact me if you would like to discuss further.

Sincerely,

Christian H. Nyberg, P.E.
Senior Bridge Engineer

Enclosures

CC: B. Hyre, Project File (212-45878)

HDR Engineering, Inc.
9987 Carver Road
Suite 200
Cincinnati, OH 45242-5710
Phone: (513) 984-7500
Fax: (513) 984-7580
www.hdrinc.com

inter-office communication

to: District 9 - Gary Cochenour, District Production Administrator date: November 5, 2007

from: Tim Keller, P.E., Administrator, Office of Structural Engineering by: Peter Narsavage, P.E.

subject: SCI-823-6.81; PID 19415; SCI-TR 234-0122 Subsurface Exploration Report
Shumway Hollow Road over CSXT Railroad

We have reviewed the subsurface exploration report and updated TS&L plans submitted for the above referenced project and we offer the following comments.

1. We concur with the concept for the forward abutment of a drilled shaft retaining wall with an integral abutment above the shafts. However, the recommended size and depth of the drilled shafts seem excessive. The proposed shafts are 4-foot diameter spaced 6 feet apart (center-to-center) and are socketed 25 feet into rock. It appears three assumptions are responsible for these results: using a deflection criteria of 1/8-inch, neglecting any resistance from the top 10 feet of rock, and neglecting any resistance from the superstructure after the beams are placed.

2. The deflection criteria of 1/8-inch was based on typical construction tolerances for structures. However, using this criteria results in a very costly abutment. We believe that using a less restrictive deflection criteria (perhaps ½ to ¾ inch before setting the beams) will result in a more cost effective design. Allowing the abutment wall to deflect more than 1/8-inch will reduce the lateral forces from at-rest earth pressure to active earth pressure. We also believe that the abutment can be designed to accommodate the anticipated movement of the drilled shafts before the beams are set and the abutment backwall is poured.

3. The exposed bedrock at the forward abutment is highly weathered and highly fractured. For this reason, DLZ chose to ignore any lateral resistance in the top ten feet of bedrock. We believe totally ignoring any resistance from the rock is overly conservative. The borings behind the proposed abutment indicate the rock is only slightly weathered and slightly fractured away from the exposed face, with an RQD of at least 80 percent. Also, the rock at the exposed face still has sufficient strength to support a slope steeper than 50 degrees. Based on these observations, we believe it is appropriate to use a reduced strength for the bedrock in the upper ten feet, but not appropriate to totally neglect it.

4. The calculations by DLZ assume there is no lateral resistance from the superstructure after the beams are placed and the abutment backwall is poured. We believe there will be significant lateral resistance from the superstructure, provided by the passive earth pressure at the rear abutment and the weight of the superstructure because the bridge has a slight grade down towards the forward abutment.

5. The bottom of the footing for the forward abutment is shown at Elev. 636, about 4 feet below rock. To simplify construction and minimize costs, rock excavation should be kept to a minimum.

We have attached calculations to demonstrate the proof-of-concept for a drilled shaft wall consisting of 3-foot diameter shafts, spaced 8 feet apart (center-to-center), and socketed 13 feet into bedrock. Our assumptions include: active earth pressure on the wall, lateral resistance from the superstructure after the beams are placed, and modeling the ten feet of fractured rock as a stiff clay. We used the Hoek-Brown criteria to calculate an equivalent cohesion of 3,000 psf for the fractured rock, and we included the ground slope of 56 degrees in the LPile analysis. We used eight #14 bars for the longitudinal reinforcing in the drilled shaft. The shaft extends one shaft diameter (3 feet) into the intact bedrock below Elev. 630. The resulting deflection at the beam seat is 0.8 inches before placing the beams and 1.25 inches at completion. However, because the bridge is at a slight grade down towards the forward abutment, we believe that any deflection after the superstructure is in place will be insignificant.

We understand that a different designer will be performing the Stage 2 design of this structure. Please send our comments to the new designer and have them incorporate them into the Stage 2 design.

Nothing in these comments is to be construed as authorizing extra work for which additional compensation may be claimed. If you believe that these comments require work outside the limits of the Scope of Services for this project, please contact this office before proceeding.

If you should have any questions regarding these comments, please contact our office.

TJK:JS: pan

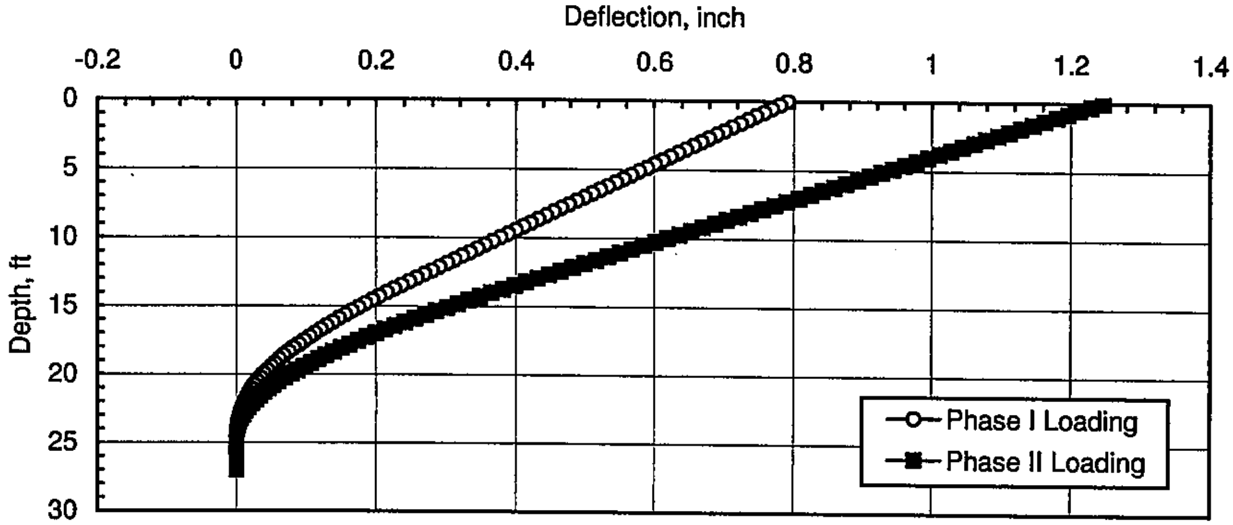
c: Tom Barnitz, P.E., District 9
Larry Wills, P.E., District 9
Chad Mitten, P.E., District 9 Geotechnical Engineer
Tim Keller, P.E., Office of Structural Engineering
Jawdat Siddiqi, P.E., Office of Structural Engineering
file

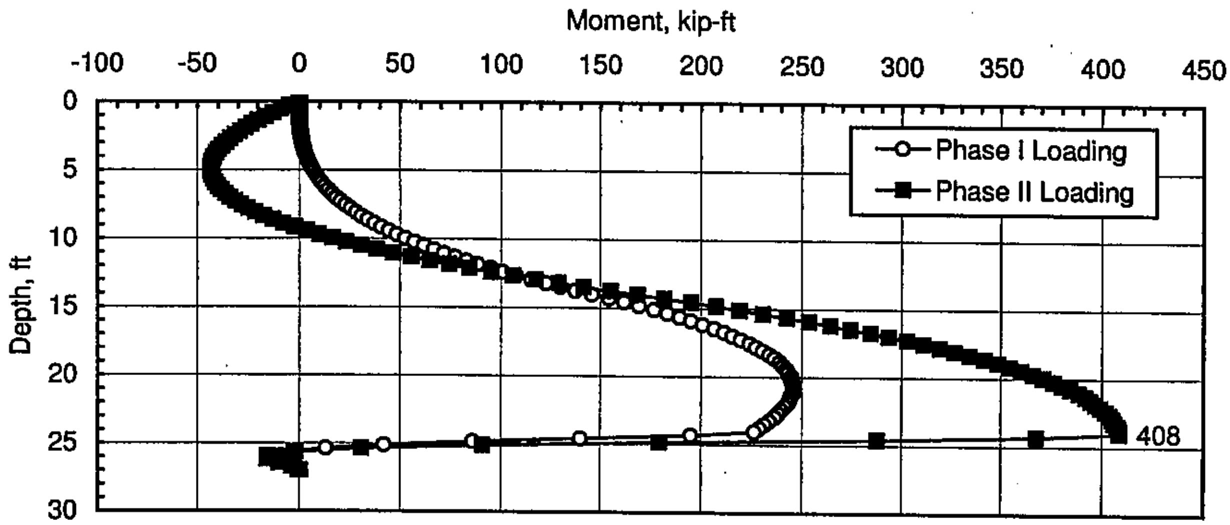
Area (3' ϕ) Shaft = $\pi(18'')^2 = 1017 \text{ in}^2$

$* .01 = 10.2 \text{ in}^2$

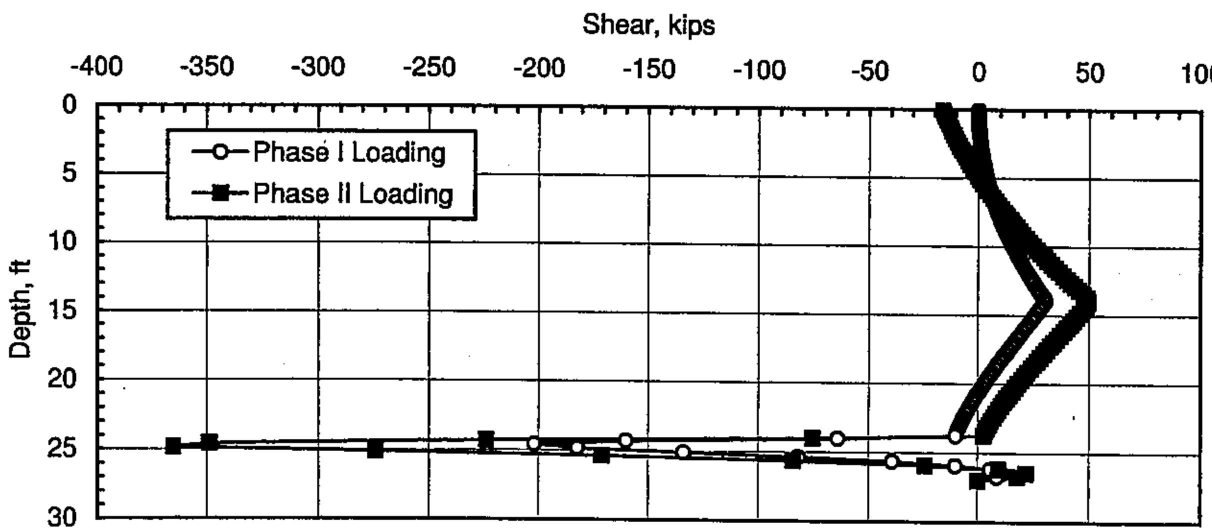
(8) #14 Bars = (8)(2.25) = 18 in^2 OK

SCI-TR 234-0122 Shumway Hollow Road
Forward Abutment Retaining Wall
3-ft Diameter Shafts, 8-ft Spacing
Preliminary Lateral Load Analysis





408 kip-ft x 1.3 x 1.3 = 690 kip-ft (Factored)



Ohio Dept. of Transportation 1980 West Broad St. Columbus, OH 43223	Project:	SCI-TR 234-0122 PID 19415	Engineer:	PAN	Project #
	Subject:	Drilled Shaft Interaction Diagram	Date:	11/5/07	Page: 1
			Checker:		
			Date:		

Capacity Of Eccentrically Loaded Short Column

Material Properties

f'_c = 4.00 ksi E_c = 3,605 ksi
f_y = 60.00 ksi E_s = 29,000 ksi
ϕ_b = 0.9
ϕ_c = 0.75

Section Shape: ○ Rectangular ◉ Circular
Units: ◉ US ○ Metric
Design Code: ○ ACI 318-99 ◉ ACI 318-02

Diameter = 36.00 in

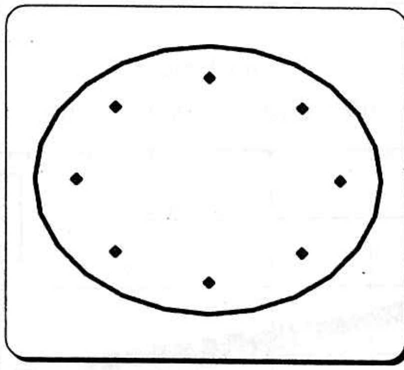
Cover = 4.25 in
No.bars = 8
Bar Area = 2.25 in^2
ρ_s = 1.77%

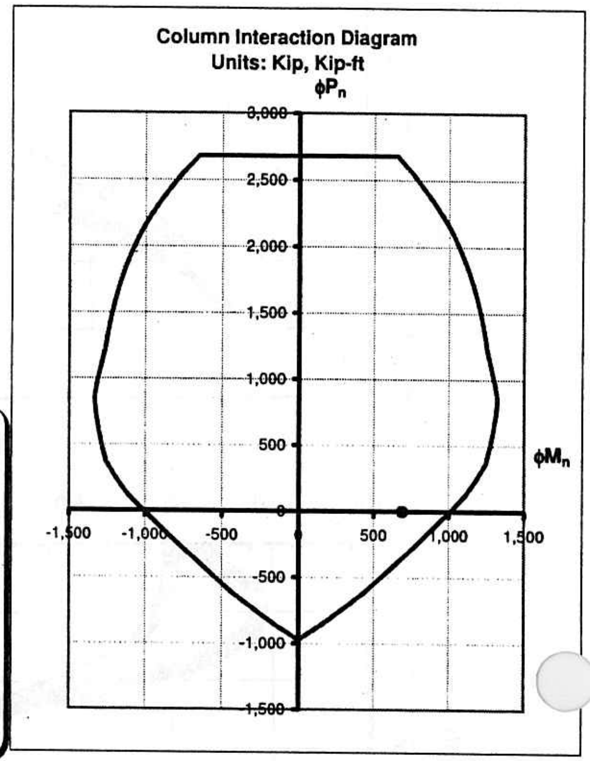
Maximum Force: $\phi P_u(max) = 0.8\phi[0.85f'_cbh+(f_y-f'_c)\Sigma A_s]$ = 2,681 kip

Maximum calculated capacities:

ϕM_b = 1329 kip-ft ϕM_b = -1329 kip-ft
ϕP_b = 855 kip ϕP_b = 855 kip

Reinforcing Bar No.	Steel Area A_s in^2	Dist. from Center X in	Dist. from Bottom Y in
1	2.25	13.75	18.00
2	2.25	9.72	27.72
3	2.25	0.00	31.75
4	2.25	-9.72	27.72
5	2.25	-13.75	18.00
6	2.25	-9.72	8.28
7	2.25	0.00	4.25
8	2.25	9.72	8.28
9			
10			
11			
12			
13			
14			
15			
16			
17			
18			
19			
20			
21			
22			
23			
24			
25			
26			
27			
28			
29			
30			
31			
32			
33			
34			
35			
36			
37			
38			
39			
40			





Interaction Diagram ϕP_n kip	ϕM_n kip-ft	ϕ
-972	0	0.900
-850	160	0.900
-728	306	0.900
-607	443	0.900
-485	560	0.900
-363	676	0.900
-241	791	0.900
-120	903	0.900
2	1012	0.900
124	1113	0.900
246	1189	0.900
368	1252	0.900
489	1279	0.873
611	1302	0.845
733	1318	0.818
855	1329	0.793
976	1311	0.775
1098	1285	0.757
1220	1261	0.750
1342	1243	0.750
1464	1222	0.750
1585	1197	0.750
1707	1167	0.750
1829	1131	0.750
1951	1090	0.750
2072	1041	0.750
2194	983	0.750
2316	914	0.750
2438	835	0.750
2559	750	0.750
2681	652	0.750
2681	0	0.750

	Factored Loads Pu kip	Mu kip-ft	Capacity ϕM_n kip-ft	D/C Ratio %
1	0	690	1010	68%
2				
3				
4				
5				
6				
7				
8				
9				
10				
11				
12				
13				
14				
15				
16				
17				
18				
19				
20				

DR-21

COMPUTATION SHEET

OHIO DEPARTMENT OF TRANSPORTATION - OFFICE OF PRODUCTION - COLUMBUS, OHIO

DESIGNER PAN DATE 9/12/07 CHECKER ________ DATE ________ NO ___ OF ___ SHEETS

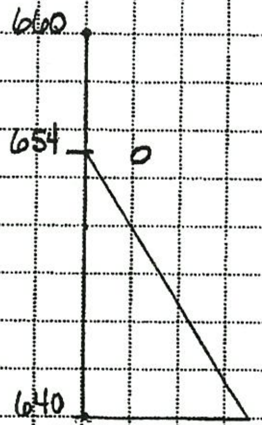
COUNTY SCI - TR 234-0122 SECTION ________

BRIDGE NO ________ OVER ________

SUBJECT OF THIS SHEET FWD ABUTMENT LPILE ANALYSES

Before beam placement

Phase 1

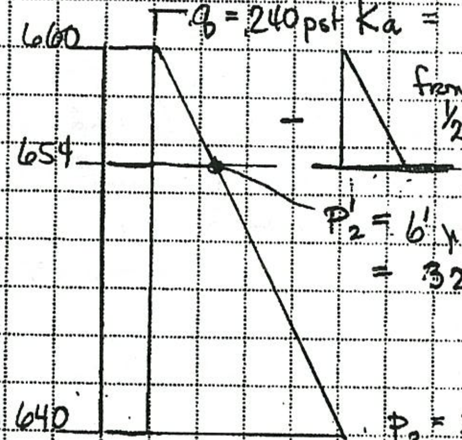


$P_1 = H \cdot \gamma \cdot K_a$
$= 14'(120)(0.33)$
$= 554$ psf

Final

Phase II

$q = 240$ psf $K_a = 80$ psf

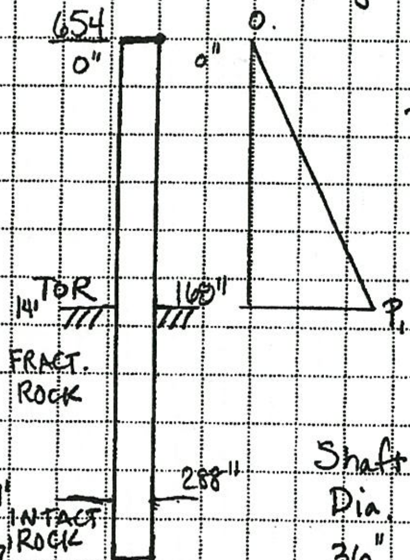


from rear abut.
$\frac{1}{2} K_p ; 6' \gamma = 1080$ psf

$P_2' = 6' \gamma K_a + 80$
$= 320$ psf

$P_2 = 20 \cdot \gamma \cdot K_a + 80$
$= 1880$

Phase I Loading

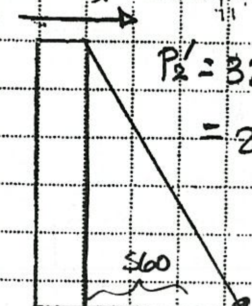


$P_1 = 554 \frac{lb}{ft^2} \times S \times \frac{1 ft}{12"}$

$= 46.2$ lb/in $\times$ Spacing (ft)

Phase II Loading

$(80+320)/2 \times 6 - \frac{1080}{2} \times 6 = -2040$ lb/ft



$P_2' = 320 \frac{lb}{ft^2} \times S \times \frac{1 ft}{12"}$

$= 26.7$ lb/in $\times S$

$P_2 = 1880 \times S \times \frac{1 ft}{12"}$

$= 173.3 \times S$ lb/in

Shaft

Dia.	Area	I
36"	1018 in²	82448
~~42"~~	~~1385~~	~~152745~~
~~48"~~	~~1810~~	~~260576~~

E = 5000 ksi

Shaft Spacing

$320 \times 14' = 600$ plf @ $7' \times 7.5 =$
3920 @ $4.67' \times 7.5 =$
$169^{k'}$

BR-21

COMPUTATION SHEET

OHIO DEPARTMENT OF TRANSPORTATION - OFFICE OF PRODUCTION - COLUMBUS, OHIO

DESIGNER_______DATE _______CHECKER________DATE_________NO___OF___SHEETS

COUNTY______________________________SECTION______________

BRIDGE NO_______________ OVER______________________________

SUBJECT OF THIS SHEET______________________________

Fractured rock

Hoek - Brown criteria

$q_u = 10$ ksi

GSI = 30 (Poor rock)

$m_i = 17$ (sandstone)

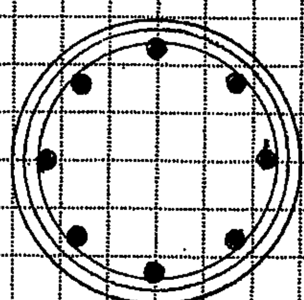
for slope ≥ 20' high 140 pcf

$C = 21$ psi $\phi = 63.79°$

≈ 3000 psf

model as clay with $s_u = 3000$ psf

ground surface slopes 1.5V:1H 56°



8 - #14 bars

clearance: $3" + \frac{1}{2}" + \frac{1}{2} - \frac{14}{8} = 4.375"$

$36" - 2(4.375) = 27.25"$

clearance between longitudinal bars

$\pi(27.25")/8 - 14/8 = 8.95"$ OK

===

LPILE Plus for Windows, Version 5.0 (5.0.9)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

===

This program is licensed to:

ODOT Engineer
Ohio Department of Transportation

```
Path to file locations:      X:\Bridge Design
Resources\Projects\SCI\SCI-823-0.00\SCI-TR234-0122\Lpile\
Name of input data file:     3ft_dia-8ft_spa-Ph1.lpd
Name of output file:         3ft_dia-8ft_spa-Ph1.lpo
Name of plot output file:    3ft_dia-8ft_spa-Ph1.lpp
Name of runtime file:        3ft_dia-8ft_spa-Ph1.lpr
```

Time and Date of Analysis

Date: November 5, 2007   Time: 14:11:42

Problem Title

SCI-Shumway Hollow 3-ft Dia 8-ft spacing Phase 1

Program Options

Units Used in Computations - US Customary Units, inches, pounds

Basic Program Options:

Analysis Type 3:
- Computation of Nonlinear Bending Stiffness and Ultimate Bending Moment Capacity with Pile Response Computed Using Nonlinear EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
```
- Number of pile increments              =        100
- Maximum number of iterations allowed   =        100
- Deflection tolerance for convergence   =   1.0000E-04 in
- Maximum allowable deflection           =   1.0000E+02 in
```

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

```
Pile Length                              =     324.00 in
```

3ft_dia-8ft_spa-Ph1.lpo

Depth of ground surface below top of pile = 168.00 in
Slope angle of ground surface = 56.00 deg.

Structural properties of pile defined using 2 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	36.00000000	82448.0000	1018.0000	5000000.
2	324.0000	36.00000000	82448.0000	1018.0000	5000000.

Please note that because this analysis makes computations of ultimate moment capacity and pile response using nonlinear bending stiffness that the above values of moment of inertia and modulus of are not used for any computations other than total stress due to combined axial loading and bending.

Soil and Rock Layering Information

The soil profile is modelled using 2 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 168.000 in
Distance from top of pile to bottom of layer = 288.000 in

Layer 2 is weak rock, p-y criteria by Reese, 1997
Distance from top of pile to top of layer = 288.000 in
Distance from top of pile to bottom of layer = 620.000 in
Initial modulus of rock at top of layer = 2.5000E+06 lbs/in**2
Initial modulus of rock at bottom of layer = 2.5000E+06 lbs/in**2

(Depth of lowest layer extends 296.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth is defined using 4 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	168.00	.08100
2	288.00	.08100
3	288.00	.08100
4	620.00	.08100

Shear Strength of Soils

Distribution of shear strength parameters with depth defined using 4 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	168.000	21.00000	.00	.00500	.0
2	288.000	21.00000	.00	.00500	.0
3	288.000	10000.00000	.00	.00050	80.0
4	620.000	10000.00000	.00	.00050	80.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves

Distributed Lateral Loading

Distributed lateral load intensity defined using 2 points

Point No.	Depth X in	Dist. Load lbs/in
1	.000	.00000
2	168.000	369.60000

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = .000 lbs
Bending moment at pile head = .000 in-lbs
Axial load at pile head = .000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Computations of Ultimate Moment Capacity and Nonlinear Bending Stiffness

Number of pile sections = 1

Pile Section No. 1

The sectional shape is a circular drilled shaft (bored pile).

Outside Diameter = 36.0000 In

Material Properties:

Compressive Strength of Concrete = 4.000 Kip/In**2
Yield Stress of Reinforcement = 60. Kip/In**2
Modulus of Elasticity of Reinforcement = 29000. Kip/In**2
Number of Reinforcing Bars = 8
Area of Single Bar = 2.25000 In**2
Number of Rows of Reinforcing Bars = 5
Cover Thickness (edge to bar center) = 4.250 In

Unfactored Axial Squash Load Capacity = 4479.58 Kip

Distribution and Area of Steel Reinforcement

Row Number	Area of Reinforcement In**2	Distance to Centroidal Axis In
1	2.250000	13.7500
2	4.500000	9.7227
3	4.500000	.0000
4	4.500000	-9.7227
5	2.250000	-13.7500

Axial Thrust Force = .00 lbs

Bending Moment in-lbs	Bending Stiffness lb-in2	Bending Curvature rad/in	Maximum Strain in/in	Neutral Axis Position inches
348971.48316	3.489715E+11	.00000100	.00001806	18.05534363
1725183.	3.450365E+11	.00000500	.00009028	18.05506897
1725183.	1.916870E+11	.00000900	.00009596	10.66209412
1725183.	1.327064E+11	.00001300	.00013885	10.68077087
1747125.	1.027721E+11	.00001700	.00018190	10.69972229
2153862.	1.025648E+11	.00002100	.00022510	10.71894836
2558892.	1.023557E+11	.00002500	.00026846	10.73844910
2962188.	1.021444E+11	.00002900	.00031199	10.75822449
3363723.	1.019310E+11	.00003300	.00035568	10.77827454
3763463.	1.017152E+11	.00003700	.00039955	10.79859924
4161717.	1.015053E+11	.00004100	.00044290	10.80244446
4557804.	1.012845E+11	.00004500	.00048699	10.82194519
4952014.	1.010615E+11	.00004900	.00053124	10.84172058
5344451.	1.008387E+11	.00005300	.00057569	10.86204529
5734815.	1.006108E+11	.00005700	.00062030	10.88237000
6123348.	1.003828E+11	.00006100	.00066510	10.90324402
6509712.	1.001494E+11	.00006500	.00071007	10.92411804
6894009.	9.991318E+10	.00006900	.00075522	10.94526672
7276368.	9.967627E+10	.00007300	.00080059	10.96696472
7656580.	9.943610E+10	.00007700	.00084615	10.98893738
8034595.	9.919253E+10	.00008100	.00089191	11.01118469
8410359.	9.894540E+10	.00008500	.00093787	11.03370667
8784015.	9.869680E+10	.00008900	.00098405	11.05677795
9155317.	9.844427E+10	.00009300	.00103045	11.08012390
9524204.	9.818767E+10	.00009700	.00107706	11.10374451
9871485.	9.773748E+10	.00010100	.00112311	11.11994934
11441437.	8.733922E+10	.00013100	.00143020	10.91752625
11980896.	7.441551E+10	.00016100	.00169210	10.50993347
12472458.	6.530083E+10	.00019100	.00195620	10.24186707
12921918.	5.847022E+10	.00022100	.00223195	10.09931946
13332455.	5.311735E+10	.00025100	.00251025	10.00099182
13451155.	4.786888E+10	.00028100	.00274676	9.77494812
13490300.	4.337717E+10	.00031100	.00298338	9.59284973
13508890.	3.961551E+10	.00034100	.00320242	9.39125061
13524880.	3.645520E+10	.00037100	.00342383	9.22865295
13540155.	3.376597E+10	.00040100	.00364661	9.09379578
13540155.	3.141567E+10	.00043100	.00387894	8.99986267

Unfactored (Nominal) Moment Capacity at Concrete Strain of 0.003 = 13491.71100 In-Kip

***** WARNING *****

An unreasonable input value for unconfined compressive strength has been specified for a soil defined using the weak rock criteria. The input value is greater than 1000 psi. You should check your input data for correctness.

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = .000 lbs
Specified moment at pile head = .000 in-lbs
Specified axial load at pile head = .000 lbs

(Zero moment for this load indicates free-head conditions)

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Flx. Rig. EI lbs-in**2	Soil Res p lbs/in
0.000	.791870	-3.32E-05	0.000	-.003463	7.25E-09	3.49E+11	0.000
3.240	.780650	4.677	12.991	-.003463	.001021	3.49E+11	0.000
6.480	.769431	84.180	47.633	-.003463	.018378	3.49E+11	0.000
9.720	.758212	313.338	105.370	-.003463	.068408	3.49E+11	0.000
12.960	.746992	766.976	186.201	-.003463	.167446	3.49E+11	0.000

16.200	.735773	1519.921	290.127	-.003463	.331828	3.49E+11	0.000
19.440	.724553	2647.001	417.148	-.003463	.577892	3.49E+11	0.000
22.680	.713334	4223.043	567.264	-.003463	.921972	3.49E+11	0.000
25.920	.702115	6322.872	740.474	-.003463	1.380	3.49E+11	0.000
29.160	.690896	9021.317	936.780	-.003463	1.970	3.49E+11	0.000
32.400	.679677	12393.	1156.179	-.003463	2.706	3.49E+11	0.000
35.640	.668459	16513.	1398.674	-.003462	3.605	3.49E+11	0.000
38.880	.657241	21457.	1664.263	-.003462	4.684	3.49E+11	0.000
42.120	.646023	27298.	1952.947	-.003462	5.960	3.49E+11	0.000
45.360	.634807	34112.	2264.726	-.003462	7.447	3.49E+11	0.000
48.600	.623592	41973.	2599.599	-.003461	9.164	3.49E+11	0.000
51.840	.612378	50957.	2957.568	-.003461	11.125	3.49E+11	0.000
55.080	.601165	61138.	3338.630	-.003460	13.348	3.49E+11	0.000
58.320	.589954	72591.	3742.788	-.003460	15.848	3.49E+11	0.000
61.560	.578746	85392.	4170.040	-.003459	18.643	3.49E+11	0.000
64.800	.567540	99613.	4620.387	-.003458	21.748	3.49E+11	0.000
68.040	.556337	1.15E+05	5093.829	-.003457	25.179	3.49E+11	0.000
71.280	.545137	1.33E+05	5590.366	-.003456	28.954	3.49E+11	0.000
74.520	.533942	1.52E+05	6109.997	-.003455	33.088	3.49E+11	0.000
77.760	.522751	1.72E+05	6652.723	-.003453	37.598	3.49E+11	0.000
81.000	.511565	1.95E+05	7218.543	-.003452	42.500	3.49E+11	0.000
84.240	.500385	2.19E+05	7807.459	-.003450	47.810	3.49E+11	0.000
87.480	.489212	2.45E+05	8419.469	-.003447	53.545	3.49E+11	0.000
90.720	.478046	2.74E+05	9054.574	-.003445	59.721	3.49E+11	0.000
93.960	.466888	3.04E+05	9712.773	-.003442	66.354	3.49E+11	0.000
97.200	.455739	3.36E+05	10394.	-.003439	73.462	3.49E+11	0.000
100.440	.444601	3.71E+05	11098.	-.003436	81.059	3.49E+11	0.000
103.680	.433473	4.08E+05	11826.	-.003432	89.163	3.48E+11	0.000
106.920	.422358	4.48E+05	12577.	-.003428	97.789	3.48E+11	0.000
110.160	.411257	4.90E+05	13350.	-.003424	106.955	3.48E+11	0.000
113.400	.400170	5.34E+05	14147.	-.003419	116.676	3.47E+11	0.000
116.640	.389100	5.82E+05	14967.	-.003414	126.969	3.47E+11	0.000
119.880	.378047	6.31E+05	15810.	-.003408	137.850	3.47E+11	0.000
123.120	.367013	6.84E+05	16676.	-.003402	149.335	3.47E+11	0.000
126.360	.356000	7.39E+05	17565.	-.003396	161.441	3.46E+11	0.000
129.600	.345009	7.98E+05	18477.	-.003388	174.184	3.46E+11	0.000
132.840	.334043	8.59E+05	19413.	-.003381	187.581	3.46E+11	0.000
136.080	.323102	9.24E+05	20371.	-.003372	201.647	3.46E+11	0.000
139.320	.312190	9.91E+05	21353.	-.003363	216.400	3.46E+11	0.000
142.560	.301308	1.06E+06	22357.	-.003354	231.855	3.46E+11	0.000
145.800	.290458	1.14E+06	23385.	-.003343	248.029	3.46E+11	0.000
149.040	.279642	1.21E+06	24436.	-.003332	264.938	3.45E+11	0.000
152.280	.268863	1.29E+06	25510.	-.003321	282.598	3.45E+11	0.000
155.520	.258124	1.38E+06	26607.	-.003308	301.026	3.45E+11	0.000
158.760	.247427	1.47E+06	27727.	-.003295	320.239	3.45E+11	0.000
162.000	.236774	1.56E+06	28870.	-.003281	340.252	3.45E+11	0.000
165.240	.226169	1.65E+06	30036.	-.003266	361.081	3.45E+11	0.000
168.480	.215613	1.75E+06	30218.	-.003247	382.744	2.53E+11	-381.119
171.720	.205131	1.85E+06	29189.	-.003206	403.830	1.03E+11	-383.593
174.960	.194838	1.94E+06	27942.	-.003146	424.038	1.03E+11	-385.790
178.200	.184743	2.03E+06	26689.	-.003084	443.361	1.03E+11	-387.700
181.440	.174856	2.12E+06	25430.	-.003018	461.795	1.03E+11	-389.318
184.680	.165186	2.20E+06	24167.	-.002950	479.337	1.03E+11	-390.634
187.920	.155740	2.27E+06	22900.	-.002879	495.984	1.02E+11	-391.640
191.160	.146527	2.34E+06	21630.	-.002806	511.734	1.02E+11	-392.329
194.400	.137554	2.41E+06	20358.	-.002731	526.584	1.02E+11	-392.689
197.640	.128829	2.48E+06	19086.	-.002654	540.534	1.02E+11	-392.711
200.880	.120357	2.54E+06	17814.	-.002575	553.585	1.02E+11	-392.385
204.120	.112145	2.59E+06	16544.	-.002493	565.736	1.02E+11	-391.700
207.360	.104199	2.64E+06	15276.	-.002411	576.989	1.02E+11	-390.642
210.600	.096524	2.69E+06	14013.	-.002326	587.347	1.02E+11	-389.199
213.840	.089126	2.73E+06	12755.	-.002240	596.813	1.02E+11	-387.356
217.080	.082008	2.77E+06	11503.	-.002153	605.391	1.02E+11	-385.099
220.320	.075174	2.81E+06	10260.	-.002065	613.087	1.02E+11	-382.411
223.560	.068629	2.84E+06	9026.077	-.001975	619.906	1.02E+11	-379.272
226.800	.062376	2.87E+06	7803.084	-.001885	625.856	1.02E+11	-375.662
230.040	.056417	2.89E+06	6592.585	-.001793	630.945	1.02E+11	-371.559
233.280	.050756	2.91E+06	5396.220	-.001701	635.183	1.02E+11	-366.938
236.520	.045393	2.92E+06	4215.714	-.001609	638.579	1.02E+11	-361.769
239.760	.040330	2.94E+06	3052.893	-.001516	641.147	1.02E+11	-356.021
243.000	.035570	2.94E+06	1909.694	-.001423	642.898	1.02E+11	-349.657
246.240	.031112	2.95E+06	788.181	-.001329	643.848	1.02E+11	-342.635
249.480	.026957	2.95E+06	-309.432	-.001236	644.013	1.02E+11	-334.904
252.720	.023105	2.95E+06	-1380.759	-.001142	643.411	1.02E+11	-326.408
255.960	.019556	2.94E+06	-2423.206	-.001049	642.060	1.02E+11	-317.077
259.200	.016310	2.93E+06	-3433.932	-.000956	639.983	1.02E+11	-306.828
262.440	.013364	2.92E+06	-4409.799	-.000863	637.202	1.02E+11	-295.558
265.680	.010719	2.90E+06	-5347.290	-.000770	633.744	1.02E+11	-283.140
268.920	.008371	2.88E+06	-6242.415	-.000679	629.637	1.02E+11	-269.407

272.160	.006320	2.86E+06	-7090.563	-.000588	624.913	1.02E+11	-254.142
275.400	.004563	2.84E+06	-7886.280	-.000497	619.606	1.02E+11	-237.046
278.640	.003098	2.81E+06	-8622.962	-.000408	613.756	1.02E+11	-217.691
281.880	.001921	2.78E+06	-9292.202	-.000319	607.407	1.02E+11	-195.420
285.120	.001030	2.75E+06	-9882.792	-.000231	600.610	1.02E+11	-169.142
288.360	.000422	2.72E+06	-64139.	-.000145	593.426	1.02E+11	-33322.
291.600	9.21E-05	2.34E+06	-1.60E+05	-6.48E-05	509.872	1.02E+11	-25609.
294.840	1.77E-06	1.68E+06	-2.02E+05	-2.00E-05	367.628	3.45E+11	-553.134
298.080	-3.73E-05	1.03E+06	-1.82E+05	-7.25E-06	224.116	3.46E+11	12807.
301.320	-4.52E-05	5.04E+05	-1.34E+05	-8.83E-08	109.954	3.47E+11	16873.
304.560	-3.79E-05	1.58E+05	-81977.	2.99E-06	34.463	3.49E+11	15276.
307.800	-2.58E-05	-27574.	-39118.	3.60E-06	6.020	3.49E+11	11181.
311.040	-1.46E-05	-95629.	-10075.	3.03E-06	20.878	3.49E+11	6746.690
314.280	-6.20E-06	-92860.	5807.111	2.15E-06	20.273	3.49E+11	3057.004
317.520	-6.25E-07	-57999.	11290.	1.45E-06	12.662	3.49E+11	327.303
320.760	3.20E-06	-19702.	8950.461	1.09E-06	4.301	3.49E+11	-1771.271
324.000	6.43E-06	0.000	0.000	9.98E-07	0.000	3.49E+11	-3753.705

Please note that because this analysis makes computations of ultimate moment capacity and pile response using nonlinear bending stiffness that the above values of total stress due to combined axial stress and bending may not be representative of actual conditions.

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

Pile-head deflection	=	.79186998 in
Computed slope at pile head	=	-.00346281
Maximum bending moment	=	2949867. lbs-in
Maximum shear force	=	-201989.50026 lbs
Depth of maximum bending moment	=	249.48000 in
Depth of maximum shear force	=	294.84000 in
Number of iterations	=	31
Number of zero deflection points	=	2

Summary of Pile-Head Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment,	y = pile-head displacment in
Type 2 = Shear and Slope,	M = pile-head moment lbs-in
Type 3 = Shear and Rot. Stiffness,	V = pile-head shear force lbs
Type 4 = Deflection and Moment,	S = pile-head slope, radians
Type 5 = Deflection and Slope,	R = rotational stiffness of pile-headin-lbs/rad

Load Type	Boundary Condition 1	Boundary Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= 0.000	M= 0.000	0.0000	.7918700	2949867.	-201990.

===

LPILE Plus for Windows, Version 5.0 (5.0.9)

Analysis of Individual Piles and Drilled Shafts
Subjected to Lateral Loading Using the p-y Method

===

This program is licensed to:

ODOT Engineer
Ohio Department of Transportation

Path to file locations: X:\Bridge Design Resources\Projects\SCI\SCI-823-0.00\SCI-TR234-0122\Lpile\
Name of input data file: 3ft_dia-8ft_spa-Ph2.lpd
Name of output file: 3ft_dia-8ft_spa-Ph2.lpo
Name of plot output file: 3ft_dia-8ft_spa-Ph2.lpp
Name of runtime file: 3ft_dia-8ft_spa-Ph2.lpr

Time and Date of Analysis

Date: November 5, 2007 Time: 12:58:41

Problem Title

SCI-Shumway Hollow 3-ft Dia 8-ft spacing Phase 2

Program Options

Units Used in Computations - US Customary Units, inches, pounds

Basic Program Options:

Analysis Type 3:
- Computation of Nonlinear Bending Stiffness and Ultimate Bending Moment Capacity with Pile Response Computed Using Nonlinear EI

Computation Options:
- Only internally-generated p-y curves used in analysis
- Analysis does not use p-y multipliers (individual pile or shaft action only)
- Analysis assumes no shear resistance at pile tip
- Analysis for fixed-length pile or shaft only
- No computation of foundation stiffness matrix elements
- Output pile response for full length of pile
- Analysis assumes no soil movements acting on pile
- No additional p-y curves to be computed at user-specified depths

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 100
- Deflection tolerance for convergence = 1.0000E-05 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 324.00 in

3ft_dia-8ft_spa-Ph2.lpo

Depth of ground surface below top of pile = 168.00 in
Slope angle of ground surface = 56.00 deg.

Structural properties of pile defined using 2 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	36.00000000	82448.0000	1018.0000	5000000.
2	324.0000	36.00000000	82448.0000	1018.0000	5000000.

Please note that because this analysis makes computations of ultimate moment capacity and pile response using nonlinear bending stiffness that the above values of moment of inertia and modulus of are not used for any computations other than total stress due to combined axial loading and bending.

Soil and Rock Layering Information

The soil profile is modelled using 2 layers

Layer 1 is stiff clay without free water
Distance from top of pile to top of layer = 168.000 in
Distance from top of pile to bottom of layer = 288.000 in

Layer 2 is weak rock, p-y criteria by Reese, 1997
Distance from top of pile to top of layer = 288.000 in
Distance from top of pile to bottom of layer = 620.000 in
Initial modulus of rock at top of layer = 2.5000E+06 lbs/in**2
Initial modulus of rock at bottom of layer = 2.5000E+06 lbs/in**2

(Depth of lowest layer extends 296.00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth is defined using 4 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	168.00	.08100
2	288.00	.08100
3	288.00	.08100
4	620.00	.08100

Shear Strength of Soils

Distribution of shear strength parameters with depth defined using 4 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	168.000	21.00000	.00	.00500	.0
2	288.000	21.00000	.00	.00500	.0
3	288.000	10000.00000	.00	.00050	80.0
4	620.000	10000.00000	.00	.00050	80.0

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves

Distributed Lateral Loading

Distributed lateral load intensity defined using 2 points

Point No.	Depth X in	Dist. Load lbs/in
1	.000	213.60000
2	168.000	586.40000

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = -16320.000 lbs
Bending moment at pile head = .000 in-lbs
Axial load at pile head = .000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Computations of Ultimate Moment Capacity and Nonlinear Bending Stiffness

Number of pile sections = 1

Pile Section No. 1

The sectional shape is a circular drilled shaft (bored pile).

Outside Diameter = 36.0000 In

Material Properties:

Compressive Strength of Concrete = 4.000 Kip/In**2
Yield Stress of Reinforcement = 60. Kip/In**2
Modulus of Elasticity of Reinforcement = 29000. Kip/In**2
Number of Reinforcing Bars = 8
Area of Single Bar = 2.25000 In**2
Number of Rows of Reinforcing Bars = 5
Cover Thickness (edge to bar center) = 4.250 In

Unfactored Axial Squash Load Capacity = 4479.58 Kip

Distribution and Area of Steel Reinforcement

Row Number	Area of Reinforcement In**2	Distance to Centroidal Axis In
1	2.250000	13.7500
2	4.500000	9.7227
3	4.500000	.0000
4	4.500000	-9.7227
5	2.250000	-13.7500

Axial Thrust Force = .00 lbs

Bending Moment in-lbs	Bending Stiffness lb-in2	Bending Curvature rad/in	Maximum Strain in/in	Neutral Axis Position inches
348971.48316	3.489715E+11	.00000100	.00001806	18.05534363
1725183.	3.450365E+11	.00000500	.00009028	18.05506897
1725183.	1.916870E+11	.00000900	.00009596	10.66209412
1725183.	1.327064E+11	.00001300	.00013885	10.68077087
1747125.	1.027721E+11	.00001700	.00018190	10.69972229
2153862.	1.025648E+11	.00002100	.00022510	10.71894836
2558892.	1.023557E+11	.00002500	.00026846	10.73844910
2962188.	1.021444E+11	.00002900	.00031199	10.75822449
3363723.	1.019310E+11	.00003300	.00035568	10.77827454
3763463.	1.017152E+11	.00003700	.00039955	10.79859924
4161717.	1.015053E+11	.00004100	.00044290	10.80244446
4557804.	1.012845E+11	.00004500	.00048699	10.82194519
4952014.	1.010615E+11	.00004900	.00053124	10.84172058
5344451.	1.008387E+11	.00005300	.00057569	10.86204529
5734815.	1.006108E+11	.00005700	.00062030	10.88237000
6123348.	1.003828E+11	.00006100	.00066510	10.90324402
6509712.	1.001494E+11	.00006500	.00071007	10.92411804
6894009.	9.991318E+10	.00006900	.00075522	10.94526672
7276368.	9.967627E+10	.00007300	.00080059	10.96696472
7656580.	9.943610E+10	.00007700	.00084615	10.98893738
8034595.	9.919253E+10	.00008100	.00089191	11.01118469
8410359.	9.894540E+10	.00008500	.00093787	11.03370667
8784015.	9.869680E+10	.00008900	.00098405	11.05677795
9155317.	9.844427E+10	.00009300	.00103045	11.08012390
9524204.	9.818767E+10	.00009700	.00107706	11.10374451
9871485.	9.773748E+10	.00010100	.00112311	11.11994934
11441437.	8.733922E+10	.00013100	.00143020	10.91752625
11980896.	7.441551E+10	.00016100	.00169210	10.50993347
12472458.	6.530083E+10	.00019100	.00195620	10.24186707
12921918.	5.847022E+10	.00022100	.00223195	10.09931946
13332455.	5.311735E+10	.00025100	.00251025	10.00099182
13451155.	4.786888E+10	.00028100	.00274676	9.77494812
13490300.	4.337717E+10	.00031100	.00298338	9.59284973
13508890.	3.961551E+10	.00034100	.00320242	9.39125061
13524880.	3.645520E+10	.00037100	.00342383	9.22865295
13540155.	3.376597E+10	.00040100	.00364661	9.09379578
13540155.	3.141567E+10	.00043100	.00387894	8.99986267

Unfactored (Nominal) Moment Capacity at Concrete Strain of 0.003 = 13491.71100 In-Kip

**** WARNING ****

An unreasonable input value for unconfined compressive strength has been specified for a soil defined using the weak rock criteria. The input value is greater than 1000 psi. You should check your input data for correctness.

Computed Values of Load Distribution and Deflection for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = -16320.000 lbs
Specified moment at pile head = .000 in-lbs
Specified axial load at pile head = .000 lbs

(Zero moment for this load indicates free-head conditions)

Depth X in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Flx. Rig. EI lbs-in**2	Soil Res p lbs/in
0.000	1.247	-.000273	-16320.	-.005176	5.96E-08	3.49E+11	0.000
3.240	1.230	-52312.	-15788.	-.005177	11.421	3.49E+11	0.000
6.480	1.213	-1.02E+05	-15061.	-.005177	22.335	3.49E+11	0.000
9.720	1.196	-1.50E+05	-14311.	-.005179	32.727	3.49E+11	0.000
12.960	1.180	-1.95E+05	-13537.	-.005180	42.580	3.49E+11	0.000

16.200	1.163	-2.38E+05	-12740.	-.005182	51.878	3.49E+11	0.000
19.440	1.146	-2.78E+05	-11920.	-.005185	60.604	3.49E+11	0.000
22.680	1.129	-3.15E+05	-11076.	-.005187	68.741	3.49E+11	0.000
25.920	1.112	-3.49E+05	-10210.	-.005190	76.274	3.49E+11	0.000
29.160	1.096	-3.81E+05	-9319.550	-.005194	83.185	3.49E+11	0.000
32.400	1.079	-4.10E+05	-8406.186	-.005198	89.458	3.48E+11	0.000
35.640	1.062	-4.35E+05	-7469.528	-.005201	95.077	3.48E+11	0.000
38.880	1.045	-4.58E+05	-6509.576	-.005206	100.026	3.48E+11	0.000
42.120	1.028	-4.78E+05	-5526.328	-.005210	104.286	3.48E+11	0.000
45.360	1.011	-4.94E+05	-4519.786	-.005215	107.844	3.48E+11	0.000
48.600	.994369	-5.07E+05	-3489.949	-.005219	110.681	3.47E+11	0.000
51.840	.977451	-5.17E+05	-2436.818	-.005224	112.781	3.47E+11	0.000
55.080	.960518	-5.23E+05	-1360.392	-.005229	114.128	3.47E+11	0.000
58.320	.943568	-5.25E+05	-260.671	-.005234	114.706	3.47E+11	0.000
61.560	.926603	-5.24E+05	862.345	-.005239	114.497	3.47E+11	0.000
64.800	.909623	-5.20E+05	2008.655	-.005243	113.486	3.47E+11	0.000
68.040	.892626	-5.11E+05	3178.260	-.005248	111.655	3.47E+11	0.000
71.280	.875614	-4.99E+05	4371.159	-.005253	108.989	3.47E+11	0.000
74.520	.858587	-4.83E+05	5587.353	-.005258	105.471	3.47E+11	0.000
77.760	.841545	-4.63E+05	6826.842	-.005262	101.085	3.48E+11	0.000
81.000	.824489	-4.39E+05	8089.626	-.005266	95.813	3.48E+11	0.000
84.240	.807420	-4.11E+05	9375.704	-.005270	89.640	3.48E+11	0.000
87.480	.790339	-3.78E+05	10685.	-.005274	82.549	3.49E+11	0.000
90.720	.773246	-3.41E+05	12018.	-.005277	74.524	3.49E+11	0.000
93.960	.756143	-3.00E+05	13374.	-.005280	65.548	3.49E+11	0.000
97.200	.739031	-2.55E+05	14753.	-.005283	55.604	3.49E+11	0.000
100.440	.721911	-2.05E+05	16156.	-.005285	44.677	3.49E+11	0.000
103.680	.704785	-1.50E+05	17581.	-.005286	32.749	3.49E+11	0.000
106.920	.687655	-90711.	19031.	-.005288	19.804	3.49E+11	0.000
110.160	.670522	-26686.	20503.	-.005288	5.826	3.49E+11	0.000
113.400	.653388	42148.	21999.	-.005288	9.202	3.49E+11	0.000
116.640	.636255	1.16E+05	23518.	-.005287	25.296	3.49E+11	0.000
119.880	.619126	1.95E+05	25060.	-.005286	42.472	3.49E+11	0.000
123.120	.602003	2.78E+05	26626.	-.005284	60.748	3.49E+11	0.000
126.360	.584888	3.67E+05	28215.	-.005281	80.140	3.49E+11	0.000
129.600	.567784	4.61E+05	29827.	-.005277	100.664	3.48E+11	0.000
132.840	.550694	5.60E+05	31462.	-.005272	122.336	3.47E+11	0.000
136.080	.533621	6.65E+05	33121.	-.005266	145.174	3.47E+11	0.000
139.320	.516568	7.75E+05	34803.	-.005260	169.193	3.46E+11	0.000
142.560	.499539	8.90E+05	36509.	-.005252	194.410	3.46E+11	0.000
145.800	.482537	1.01E+06	38237.	-.005243	220.842	3.46E+11	0.000
149.040	.465565	1.14E+06	39989.	-.005233	248.504	3.46E+11	0.000
152.280	.448628	1.27E+06	41764.	-.005222	277.415	3.45E+11	0.000
155.520	.431730	1.41E+06	43563.	-.005209	307.589	3.45E+11	0.000
158.760	.414874	1.55E+06	45385.	-.005195	339.044	3.45E+11	0.000
162.000	.398066	1.70E+06	47230.	-.005180	371.795	3.45E+11	0.000
165.240	.381309	1.86E+06	49098.	-.005142	405.860	1.03E+11	0.000
168.480	.364742	2.02E+06	49668.	-.005081	441.255	1.03E+11	-434.651
171.720	.348383	2.18E+06	48588.	-.005015	476.126	1.03E+11	-437.905
174.960	.332246	2.34E+06	47164.	-.004943	509.993	1.02E+11	-440.859
178.200	.316349	2.49E+06	45732.	-.004867	542.850	1.02E+11	-443.504
181.440	.300707	2.63E+06	44291.	-.004786	574.690	1.02E+11	-445.833
184.680	.285334	2.77E+06	42843.	-.004701	605.509	1.02E+11	-447.835
187.920	.270247	2.91E+06	41389.	-.004610	635.301	1.02E+11	-449.500
191.160	.255459	3.04E+06	39931.	-.004516	664.063	1.02E+11	-450.820
194.400	.240983	3.17E+06	38469.	-.004417	691.791	1.02E+11	-451.782
197.640	.226833	3.29E+06	37004.	-.004315	718.485	1.02E+11	-452.376
200.880	.213022	3.41E+06	35538.	-.004208	744.141	1.02E+11	-452.589
204.120	.199563	3.52E+06	34072.	-.004098	768.760	1.02E+11	-452.409
207.360	.186466	3.63E+06	32607.	-.003984	792.343	1.02E+11	-451.820
210.600	.173744	3.73E+06	31145.	-.003867	814.890	1.02E+11	-450.809
213.840	.161406	3.83E+06	29686.	-.003747	836.404	1.02E+11	-449.359
217.080	.149465	3.92E+06	28234.	-.003623	856.888	1.02E+11	-447.453
220.320	.137928	4.01E+06	26788.	-.003497	876.346	1.02E+11	-445.071
223.560	.126807	4.10E+06	25350.	-.003367	894.784	1.02E+11	-442.193
226.800	.116109	4.18E+06	23923.	-.003235	912.209	1.01E+11	-438.796
230.040	.105844	4.25E+06	22508.	-.003100	928.629	1.01E+11	-434.855
233.280	.096018	4.32E+06	21106.	-.002963	944.051	1.01E+11	-430.342
236.520	.086640	4.39E+06	19720.	-.002824	958.488	1.01E+11	-425.226
239.760	.077717	4.45E+06	18352.	-.002683	971.950	1.01E+11	-419.471
243.000	.069255	4.51E+06	17003.	-.002540	984.450	1.01E+11	-413.038
246.240	.061261	4.56E+06	15676.	-.002395	996.004	1.01E+11	-405.882
249.480	.053739	4.61E+06	14374.	-.002248	1006.628	1.01E+11	-397.952
252.720	.046695	4.66E+06	13099.	-.002100	1016.340	1.01E+11	-389.185
255.960	.040134	4.70E+06	11854.	-.001950	1025.159	1.01E+11	-379.513
259.200	.034060	4.73E+06	10641.	-.001799	1033.109	1.01E+11	-368.851
262.440	.028477	4.76E+06	9465.417	-.001647	1040.214	1.01E+11	-357.098
265.680	.023388	4.79E+06	8329.430	-.001494	1046.500	1.01E+11	-344.129
268.920	.018797	4.82E+06	7237.678	-.001340	1051.998	1.01E+11	-329.791

272.160	.014706	4.84E+06	6194.919	-.001185	1056.739	1.01E+11	-313.887
275.400	.011118	4.86E+06	5206.647	-.001030	1060.762	1.01E+11	-296.158
278.640	.008033	4.87E+06	4279.338	-.000874	1064.105	1.01E+11	-276.255
281.880	.005456	4.89E+06	3420.832	-.000717	1066.816	1.01E+11	-253.687
285.120	.003385	4.90E+06	2640.928	-.000561	1068.945	1.01E+11	-227.735
288.360	.001823	4.90E+06	-75574.	-.000404	1070.552	1.01E+11	-48053.
291.600	.000770	4.41E+06	-2.24E+05	-.000255	962.029	1.01E+11	-43556.
294.840	.000174	3.45E+06	-3.49E+05	-.000129	753.682	1.02E+11	-33335.
298.080	-6.72E-05	2.15E+06	-3.65E+05	-4.04E-05	468.938	1.03E+11	23077.
301.320	-8.82E-05	1.09E+06	-2.74E+05	-1.38E-06	237.083	3.46E+11	32917.
304.560	-7.61E-05	3.69E+05	-1.71E+05	5.43E-06	80.668	3.49E+11	30711.
307.800	-5.30E-05	-24567.	-84425.	7.03E-06	5.363	3.49E+11	22962.
311.040	-3.06E-05	-1.78E+05	-24275.	6.09E-06	38.769	3.49E+11	14167.
314.280	-1.35E-05	-1.82E+05	9469.228	4.42E-06	39.706	3.49E+11	6662.599
317.520	-1.90E-06	-1.16E+05	21877.	3.04E-06	25.373	3.49E+11	996.823
320.760	6.20E-06	-40103.	17935.	2.32E-06	8.755	3.49E+11	-3430.516
324.000	1.31E-05	0.000	0.000	2.13E-06	0.000	3.49E+11	-7640.415

Please note that because this analysis makes computations of ultimate moment capacity and pile response using nonlinear bending stiffness that the above values of total stress due to combined axial stress and bending may not be representative of actual conditions.

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

Pile-head deflection	=	1.24669107 in
Computed slope at pile head	=	-.00517648
Maximum bending moment	=	4903604. lbs-in
Maximum shear force	=	-365163.07218 lbs
Depth of maximum bending moment	=	288.36000 in
Depth of maximum shear force	=	298.08000 in
Number of iterations	=	33
Number of zero deflection points	=	2

Summary of Pile-Head Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

Type 1 = Shear and Moment,	y = pile-head displacment in
Type 2 = Shear and Slope,	M = pile-head moment lbs-in
Type 3 = Shear and Rot. Stiffness,	V = pile-head shear force lbs
Type 4 = Deflection and Moment,	S = pile-head slope, radians
Type 5 = Deflection and Slope,	R = rotational stiffness of pile-headin-lbs/rad

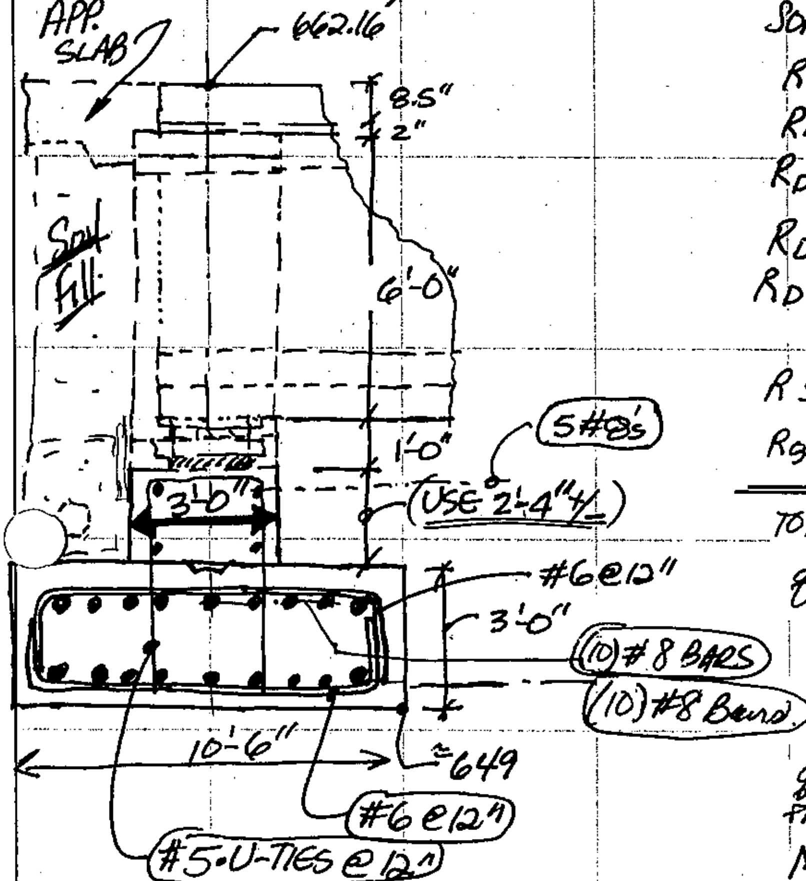
Load Type	Boundary Condition 1	Boundary Condition 2	Axial Load lbs	Pile-Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1	V= -16320.	M= 0.000	0.0000	1.2467	4903604.	-365163.

$\mathcal{C}$ BRG = 36+70.50 @ $\mathcal{C}$ RW

ELEV = 662.155' = 662.16'

FOOTING ON MSE FILL:

SLOPES = .016 FT/FT × 28.5' = 0.456' GRADE SLOPE



Soil Allowable BRG PRESSURE = 4 ksf

R_D BM = 149^k/BM

R_{LL+I} BM = 71^k/BM

$R_{DIAPHRAGM}$ = 7'×3'×0.15 = 3.15 klf × 7' = 22.10^k/BM.

R_D CAP = ± 2.33'×3'×0.15 = 1.05 klf × 7.75' = 8.2^k/BM

R_D FTG = TRIAL 10.50'×0.15×3' = 4.73 klf × 7.75' = 36.7

R SLAB = Use 1'×3.75'×7.75'×0.15 = 4.36^k/BM = 0.563 klf

R_{SOIL} = 9.34'×3.75'×7.75×0.12 = 32.6^k/BM. = 4.2 klf

TOTAL DL + LL_I SERVICE LOAD = 324 kips.

$$q = \frac{A}{q} = \frac{324 \text{ KIPS}}{(10.5' \times 7.75')} = 3.98 \text{ ksf.}$$

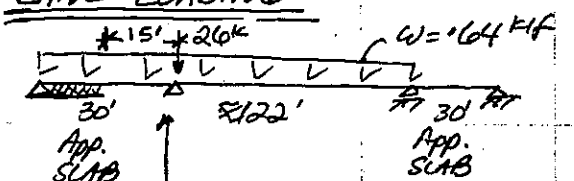
$$q_{service} = \frac{(8 \text{ bms})(104+149) + (4 \text{ lanes})(0.75)(86.3)}{(10.5)(62)} = 3.51 \text{ ksf}$$

$$q_{Factored} = \frac{1.3(2024) + 1.3(1.67)(258.9)}{(10.5)(62)} = 4.91 \text{ ksf}$$

$M_u = 232^{k'}$

$$\frac{M_u}{\phi b d^2} = \frac{232(12{,}000)}{(.9)(126)(\text{Use } 31.5")^2} = 24.74$$

LANE LOADING:



$\rho_{MIN.}$ (4, 60) = .0018

$A_s = \rho b d = (.0018)(126)(31.5) = 7.15 \text{ in}^2$

MIN. #8 Bars @ 12" O.C. = A_s = (10)(.79) = 7.9 (OK)
(MAIN. LONG. STEEL)

$$R_{LANE} = \left[0.64\left(\frac{15'+119.5}{2}\right) + 26^k\right](1.25) = 86.3^k$$

R_{BM} = (86.3)^k (1.41) (1.21) (0.5 wheel)
(S, I, Wheel)

LANE = 73.6^k ≈ 71^k ⇒ TRUCK/LANE About Equal w/ Approach Slab ⇒ $\boxed{\text{SURCHARGE loading OVER footing "tip" CAN BE NEG.}}$ ⇒ Safety factor ON Soil Brg, = Min. 2.5 @ 10 ksf

Project: **SCI-823-TR234/CSXT-RR Bridge**

Footing Analysis: **Strip Footing Analysis for Bearing Pressure**

Dean Load/Bm =	**149 kips**		
Live Load/Bm =	**71 kips**	Dist off Diap.=	3.75 feet
Soil Unit Wt=	**120 pcf**	Wt of Soil =	4.20 KLF
App slab Thk=	**15 inches**	Wt of App Slab =	0.53 KLF
Imposed LL =	**0 psf**	Wt of Sucharge=	0.00 KLF
(Truck Controls at 71 kips)			
Ftg Trial Width=	**10.5 feet**	Wt of Footing=	4.725 KLF
Ftg Trial Height=	**36 inches**		
Bm Cap Width	**36 inches**	Wt of Beam Cap=	1.05 KLF
Bm Cap Hight=	**28 inches**		
Bm Diap. Width=	**3 feet**	Wt of Diaphragm=	3.15 KLF
Bm Diap. Height=	**7 feet**		
		Total Uniform DL=	13.65 KLF
Beam Spacing=	**7.75 feet**		
Dist. To Ftg=	**9.34 feet**		
Total DL+LL =	**325.81 Kips**		
Soil Bearing Pres=	**4.00 KSF**		

Allowable Bearg @ 4 ksf.
10'-6" x 36" deep Step Ftg.

Title Block Line 1
You can changes this area
using the "Settings" menu item
and then using the "Printing &
Title Block" selection.
Title Block Line 6

Title :
Dsgnr:
Project Desc.:

Project Notes :

Job #

Printed: 2 FEB 2009, 12:01PM

Concrete Beam Design

File: c:\Documents and Settings\dflynn.KZFDESIGN.000\My Documents\ENERCALC Data Files\flynns-files.ec6
ENERCALC, INC. 1983-2008, Ver: 6.0.20

Lic. # : KW-06004130 **License Owner : kzf, inc.**

Description : SCI-823-TR234/CSXT-RR

Material Properties

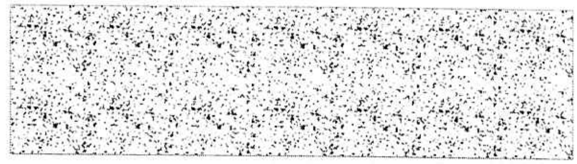
Calculations per IBC 2006, CBC 2007, ACI 318-05

f'c = 4.0 ksi
fr = $f'c^{1/2}$ * 7.50 = 0.0 psi
ψ Density = 150.0 pcf
Elastic Modulus = 3,122.0 ksi
Load Combination 2006 IBC & ASCE 7-05

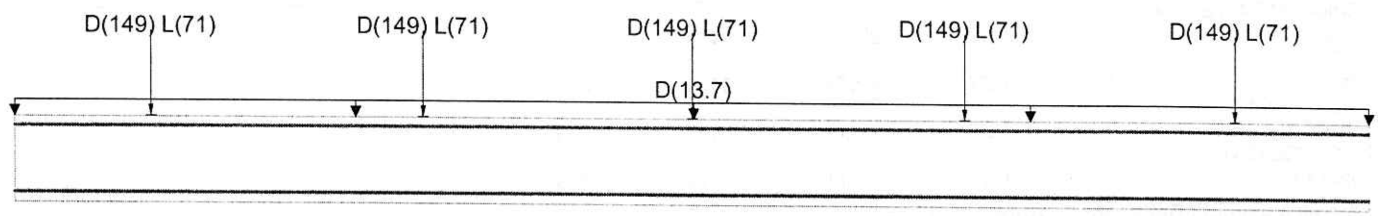
ϕ Phi Values Flexure : 0.90
Shear : 0.750
β_1 = 0.850

Fy - Main Rebar = 60.0 ksi
E - Main Rebar = 29,000.0 ksi

Fy - Stirrups = 40.0 ksi
E - Stirrups = 29,000.0 ksi
Stirrup Bar Size # # 3
Num of Bars Crossing Inclined Crack = 2



Beam is supported on an elastic foundation.



Cross Section & Reinforcing Details

Rectangular Section, Width = 126.0 in, Height = 36.0 in
Span #1 Reinforcing....
10-#8 at 4.0 in from Bottom, from 0.0 to 38.750 ft in this span
10-#8 at 32.0 in from Bottom, from 0.0 to 38.750 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Load for Span Number 1
Point Load : D = 149.0, L = 71.0 k @ 3.8750 ft
Point Load : D = 149.0, L = 71.0 k @ 11.6250 ft
Point Load : D = 149.0, L = 71.0 k @ 19.3750 ft
Point Load : D = 149.0, L = 71.0 k @ 27.1250 ft
Point Load : D = 149.0, L = 71.0 k @ 34.8750 ft
Uniform Load : D = 13.70 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio =		**0.194** : 1
Section used for this span		**Typical Section**
	Mu : Applied	231.720 k-ft
	Mn * Phi : Allowable	1,194.44 k-ft
Load Combination		+1.20D+0.50Lr+1.60L+1.60
Location of maximum on span		3.577 ft
Span # where maximum occurs		Span # 1

Maximum Deflection	
Max Downward L+Lr+S Deflection	0.031 in
Max Upward L+Lr+S Deflection	0.000 in
Max Downward Total Deflection	0.141 in
Max Upward Total Deflection	0.000 in

Maximum Soil Pressure = **4.064** ksf at 19.38 ft

Cross Section Strength & Inertia

		Phi*Mn (k-ft)		Moment of Inertia (in^4)		
Cross Section	Bar Layout Description	Btm Tension	Top Tension	I gross	Icr - Btm Tension	Icr - Top Tension
Section 1	10- #8 @ d=32",10- #8 @ d=4",	1,194.43	1,194.43	489888.00	58,663.66	58,663.66

Maximum Vertical Reactions - Unfactored

Support & Load Combinati	Support Reaction
Support 1, (D+L+Lr+S)	12.114 k

Maximum Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2
Overall MAXimum	12.114	12.114
D Only	9.521	9.521
Lr Only		

Concrete Beam Design

Description : SCI-823-TR234/CSXT-RR

Maximum Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2
L Only	2.592	2.592
S Only		
W Only		
-W Only		
E Only		
-E Only		
H Only		
D+L+Lr+S	12.114	12.114

Shear Stirrup Requirements

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	3.577	231.72	1,194.43	0.19
+1.40D					
Span # 1	1	3.577	165.29	1,194.43	0.14
+1.20D+0.50Lr+1.60L+1.60H					
Span # 1	1	3.577	231.72	1,194.43	0.19
+1.20D+1.60Lr+0.50L					
Span # 1	1	3.577	169.81	1,194.43	0.14

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L+Lr+S	1	0.1411	19.375		0.0000	0.000

Maximum Deflections for Load Combinations - Unfactored Loads

Load Combination	Span	Max. Downward Defl	Location in Span	Max. Upward Defl	Location in Span
D Only	1	0.1102	19.375	0.0000	0.000
Lr Only	1	0.0000	0.000	0.0000	0.000
L Only	1	0.0309	19.375	0.0000	0.000
S Only	1	0.0000	0.000	0.0000	0.000
Lr+L+S	1	0.0309	19.375	0.0000	0.000
W Only	1	0.0000	0.000	0.0000	0.000
-W Only	1	0.0000	0.000	0.0000	0.000
E Only	1	0.0000	0.000	0.0000	0.000
-E Only	1	0.0000	0.000	0.0000	0.000
H Only	1	0.0000	0.000	0.0000	0.000
D+L+Lr+S	1	0.1411	19.375	0.0000	0.000

Title Block Line 1
You can changes this area
using the "Settings" menu item
and then using the "Printing &
Title Block" selection.
Title Block Line 6

Title :
Dsgnr:
Project Desc.:

Project Notes :

Job #

Printed: 5 FEB 2009, 11:06AM

Concrete Beam Design

File: c:\Documents and Settings\dflynn.KZFDESIGN.000\My Documents\ENERCALC Data Files\flynns-files.ec6
ENERCALC, INC. 1983-2008, Ver: 6.0.21

Lic. # : KW-06004130

License Owner : kzf, Inc.

Description : SCI-823: TR234/CSXT-RR Forward Abutment Cap for the Drilled columns

Material Properties

Calculations per IBC 2006, CBC 2007, ACI 318-05

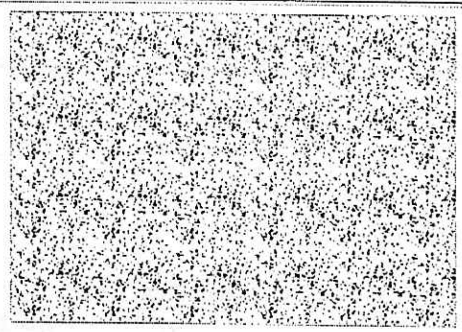
f'c = 4.0 ksi
$fr = f'c^{1/2} * 7.50$ = 474.342 psi
ψ Density = 145.0 pcf
Elastic Modulus = 3,122.0 ksi

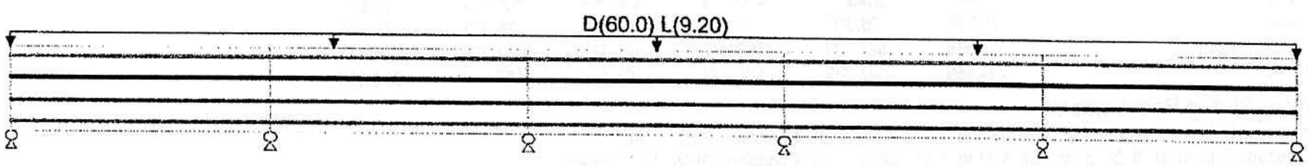
φ Phi Values Flexure : 0.90
Shear : 0.750
β_1 = 0.850

Load Combination 2006 IBC & ASCE 7-05

Fy - Main Rebar = 60.0 ksi
E - Main Rebar = 29,000.0 ksi

Fy - Stirrups = 60.0 ksi
E - Stirrups = 29,000.0 ksi
Stirrup Bar Size # = # 5
Num of Bars Crossing Inclined Crack = 2





Cross Section & Reinforcing Details

Rectangular Section, Width = 48.0 in, Height = 36.0 in

Span #1 Reinforcing....
5-#8 at 4.0 in from Bottom, from 0.0 to 7.50 ft in this span
2-#6 at 13.0 in from Bottom, from 0.0 to 7.50 ft in this span
5-#8 at 4.0 in from Top, from 0.0 to 7.50 ft in this span
2-#6 at 13.0 in from Top, from 0.0 to 7.50 ft in this span

Span #2 Reinforcing....
5-#8 at 4.0 in from Bottom, from 0.0 to 7.50 ft in this span
2-#6 at 13.0 in from Bottom, from 0.0 to 7.50 ft in this span
5-#8 at 4.0 in from Top, from 0.0 to 7.50 ft in this span
2-#6 at 13.0 in from Top, from 0.0 to 7.50 ft in this span

Span #3 Reinforcing....
5-#8 at 4.0 in from Bottom, from 0.0 to 7.50 ft in this span
2-#6 at 13.0 in from Bottom, from 0.0 to 7.50 ft in this span
5-#8 at 4.0 in from Top, from 0.0 to 7.50 ft in this span
2-#6 at 13.0 in from Top, from 0.0 to 7.50 ft in this span

Span #4 Reinforcing....
5-#8 at 4.0 in from Bottom, from 0.0 to 7.50 ft in this span
2-#6 at 13.0 in from Bottom, from 0.0 to 7.50 ft in this span
5-#8 at 4.0 in from Top, from 0.0 to 7.50 ft in this span
2-#6 at 13.0 in from Top, from 0.0 to 7.50 ft in this span

Span #5 Reinforcing....
5-#8 at 4.0 in from Bottom, from 0.0 to 7.50 ft in this span
2-#6 at 13.0 in from Bottom, from 0.0 to 7.50 ft in this span
5-#8 at 4.0 in from Top, from 0.0 to 7.50 ft in this span
2-#6 at 13.0 in from Top, from 0.0 to 7.50 ft in this span

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Loads on all spans...

Uniform Load on ALL spans : D = 60.0, L = 9.20 k/ft, Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio = **0.738 : 1**
Section used for this span **Typical Section**
Mu : Applied -513.474 k-ft
Mn * Phi : Allowable 695.715 k-ft

Load Combination +1.20D+0.50Lr+1.60L+1.60H
Location of maximum on span 0.000 ft
Span # where maximum occurs Span # 5

Maximum Deflection
Max Downward L+Lr+S Deflection 0.004 in
Max Upward L+Lr+S Deflection 0.000 in
Live Load Deflection Ratio 21409
Max Downward Total Deflection 0.004 in
Max Upward Total Deflection 0.000 in
Total Deflection Ratio 21409

Cross Section Strength & Inertia

Cross Section	Bar Layout Description	Phi*Mn (k-ft) Btm Tension	Phi*Mn (k-ft) Top Tension	Moment of Inertia (in^4) I gross	Icr - Btm Tension	Icr - Top Tension
Section 1	5- #8 @ d=32",5- #8 @ d=4",2- #6 @ d=23",2- #6 @ d=13",	695.71	695.71	186624.00	31,025.97	31,025.97
Section 2	5- #8 @ d=32",5- #8 @ d=4",2- #6 @ d=23",2- #6 @ d=13",	695.71	695.71	186624.00	31,025.97	31,025.97
Section 3	5- #8 @ d=32",5- #8 @ d=4",2- #6 @ d=23",2- #6 @ d=13",	695.71	695.71	186624.00	31,025.97	31,025.97
Section 4	5- #8 @ d=32",5- #8 @ d=4",2- #6 @ d=23",2- #6 @ d=13",	695.71	695.71	186624.00	31,025.97	31,025.97

Title Block Line 1
You can changes this area
using the "Settings" menu item
and then using the "Printing &
Title Block" selection.
Title Block Line 6

Title :
Dsgnr:
Project Desc.:

Project Notes :

Job #

Printed: 5 FEB 2009, 11:06AM

Concrete Beam Design

File: c:\Documents and Settings\dflynn.KZFDESIGN.000\My Documents\ENERCALC Data Files\flynns-files.ec6
ENERCALC, INC. 1983-2008, Ver: 6.0.21

Lic. # : KW-06004130 **License Owner : kzf, Inc.**

Description : SCI-823: TR234/CSXT-RR Forward Abutment Cap for the Drilled columns

Cross Section Strength & Inertia

Cross Section	Bar Layout Description	Phi*Mn (k-ft) Btm Tension	Top Tension	Moment of Inertia (in^4) I gross	Icr - Btm Tension	Icr - Top Tension
Section 5	5- #8 @ d=32",5- #8 @ d=4",2- #6 @ d=23",2- #6 @ d=13",	695.71	695.71	186624.00	31,025.97	31,025.97

Maximum Vertical Reactions - Unfactored

Support & Load Combinati	Support Reaction
Support 1, (D+L+Lr)	204.868 k
Support 2, (D+L+Lr)	587.289 k
Support 3, (D+L+Lr)	505.342 k
Support 4, (D+L+Lr)	505.342 k
Support 5, (D+L+Lr)	587.289 k

Maximum Vertical Reactions - Unfactored

Support notation : Far left is #1

Load Combination	Support 1	Support 2	Support 3	Support 4	Support 5	Support 6
Overall MAXimum	204.868	587.289	505.342	505.342	587.289	204.868
D Only	177.632	509.211	438.158	438.158	509.211	177.632
L Only	27.237	78.079	67.184	67.184	78.079	27.237
D+L+S	204.868	587.289	505.342	505.342	587.289	204.868
D+L+Lr	204.868	587.289	505.342	505.342	587.289	204.868

Shear Stirrup Requirements

#5 stirrups (2 legs) at 12.00 in o/c from 0.00 to 1.13 ft along span, Condition : PhiVc < Vu
#5 stirrups (2 legs) at 15.50 in o/c from 1.31 to 1.88 ft along span, Condition : PhiVc/2 < Vu <= PhiVc
No Stirrups Required from 2.06 to 3.75 ft along span, Condition : Vu < PhiVc/2
#5 stirrups (2 legs) at 15.50 in o/c from 3.94 to 4.69 ft along span, Condition : PhiVc/2 < Vu <= PhiVc
#5 stirrups (2 legs) at 5.50 in o/c from 4.88 to 9.56 ft along span, Condition : PhiVc < Vu
#5 stirrups (2 legs) at 15.50 in o/c from 9.75 to 10.50 ft along span, Condition : PhiVc/2 < Vu <= PhiVc
No Stirrups Required from 10.69 to 12.19 ft along span, Condition : Vu < PhiVc/2
#5 stirrups (2 legs) at 15.50 in o/c from 12.38 to 13.13 ft along span, Condition : PhiVc/2 < Vu <= PhiVc
#5 stirrups (2 legs) at 7.00 in o/c from 13.31 to 16.88 ft along span, Condition : PhiVc < Vu
#5 stirrups (2 legs) at 15.50 in o/c from 17.06 to 17.81 ft along span, Condition : PhiVc/2 < Vu <= PhiVc
No Stirrups Required from 18.00 to 19.50 ft along span, Condition : Vu < PhiVc/2
#5 stirrups (2 legs) at 15.50 in o/c from 19.69 to 20.44 ft along span, Condition : PhiVc/2 < Vu <= PhiVc
#5 stirrups (2 legs) at 7.75 in o/c from 20.63 to 24.19 ft along span, Condition : PhiVc < Vu
#5 stirrups (2 legs) at 15.50 in o/c from 24.38 to 25.13 ft along span, Condition : PhiVc/2 < Vu <= PhiVc
No Stirrups Required from 25.31 to 26.81 ft along span, Condition : Vu < PhiVc/2
#5 stirrups (2 legs) at 15.50 in o/c from 27.00 to 27.75 ft along span, Condition : PhiVc/2 < Vu <= PhiVc
#5 stirrups (2 legs) at 5.00 in o/c from 27.94 to 32.63 ft along span, Condition : PhiVc < Vu
#5 stirrups (2 legs) at 15.50 in o/c from 32.81 to 33.56 ft along span, Condition : PhiVc/2 < Vu <= PhiVc
No Stirrups Required from 33.75 to 35.44 ft along span, Condition : Vu < PhiVc/2
#5 stirrups (2 legs) at 15.50 in o/c from 35.63 to 36.19 ft along span, Condition : PhiVc/2 < Vu <= PhiVc
#5 stirrups (2 legs) at 12.00 in o/c from 36.38 to 37.50 ft along span, Condition : PhiVc < Vu

Maximum Forces & Stresses for Load Combinations

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
MAXimum BENDING Envelope					
Span # 1	1	7.313	-441.19	695.71	0.63
Span # 2	2	7.500	-513.47	695.71	0.74
Span # 3	3	15.000	-385.11	695.71	0.55
Span # 4	4	29.813	-450.81	695.71	0.65
Span # 5	5	30.000	-513.47	695.71	0.74
+1.40D					
Span # 1	1	7.313	-427.35	695.71	0.61
Span # 2	2	7.500	-497.37	695.71	0.71
Span # 3	3	15.000	-373.03	695.71	0.54
Span # 4	4	29.813	-436.67	695.71	0.63
Span # 5	5	30.000	-497.37	695.71	0.71
+1.20D+0.50Lr+1.60L+1.60H					
Span # 1	1	7.313	-441.19	695.71	0.63
Span # 2	2	7.500	-513.47	695.71	0.74
Span # 3	3	15.000	-385.11	695.71	0.55
Span # 4	4	29.813	-450.81	695.71	0.65
Span # 5	5	30.000	-513.47	695.71	0.74
+1.20D+1.60Lr+0.50L					
Span # 1	1	7.313	-389.70	695.71	0.56
Span # 2	2	7.500	-453.55	695.71	0.65
Span # 3	3	15.000	-340.16	695.71	0.49
Span # 4	4	29.813	-398.20	695.71	0.57

Title Block Line 1
You can changes this area
using the "Settings" menu item
and then using the "Printing &
Title Block" selection.
Title Block Line 6

Title :
Dsgnr:
Project Desc.:

Project Notes :

Job #

Printed 5 FEB 2009, 11:06AM

Concrete Beam Design

File: c:\Documents and Settings\dflynn.KZFDESIGN.000\My Documents\ENERCALC Data Files\flynns-files.ec6
ENERCALC, INC. 1983-2008, Ver: 6.0.21

Lic. # : KW-06004130

License Owner : kzf, inc.

Description : SCI-823: TR234/CSXT-RR Forward Abutment Cap for the Drilled columns

Load Combination / Segment Length	Span #	Location (ft) in Span	Bending Stress Results (k-ft) Mu : Max	Phi*Mnx	Stress Ratio
Span # 5	5	30.000	-453.55	695.71	0.65

Overall Maximum Deflections - Unfactored Loads

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
D+L+Lr	1	0.0042	3.375	D+L+Lr	-0.0001	7.875
D+L+Lr	2	0.0010	4.125	D+L+Lr	-0.0000	1.125
D+L+Lr	3	0.0020	4.125		0.0000	1.125
D+L+Lr	4	0.0010	3.375	D+L+Lr	-0.0001	7.125
D+L+Lr	5	0.0042	4.125		0.0000	7.125

Maximum Deflections for Load Combinations - Unfactored Loads

Load Combination	Span	Max. Downward Defl	Location in Span	Max. Upward Defl	Location in Span
D Only	1	0.0036	3.375	-0.0001	7.875
D Only	2	0.0008	4.125	-0.0000	1.125
D Only	3	0.0017	4.125	0.0000	0.000
D Only	4	0.0008	3.375	-0.0001	7.125
D Only	5	0.0036	4.125	0.0000	0.000
L Only	1	0.0006	3.375	-0.0000	7.875
L Only	2	0.0001	4.125	-0.0000	1.125
L Only	3	0.0003	4.125	0.0000	0.000
L Only	4	0.0001	3.375	-0.0000	7.125
L Only	5	0.0006	4.125	0.0000	0.000
D+L+S	1	0.0042	3.375	-0.0001	7.875
D+L+S	2	0.0010	4.125	-0.0000	1.125
D+L+S	3	0.0020	4.125	0.0000	0.000
D+L+S	4	0.0010	3.375	-0.0001	7.125
D+L+S	5	0.0042	4.125	0.0000	0.000
D+L+Lr	1	0.0042	3.375	-0.0001	7.875
D+L+Lr	2	0.0010	4.125	-0.0000	1.125
D+L+Lr	3	0.0020	4.125	0.0000	0.000
D+L+Lr	4	0.0010	3.375	-0.0001	7.125
D+L+Lr	5	0.0042	4.125	0.0000	0.000

KZF DESIGN Architecture | Engineering | Interiors | Planning	SCI-823 (project name)	sheet ___ of ___
		project # #5355
	BRIDGE @ TR 234/SXT-RR (subject)	date 1/09
		designed by BEf
		checked by D.A.T.

LONGITUDINAL TEMP EXP.

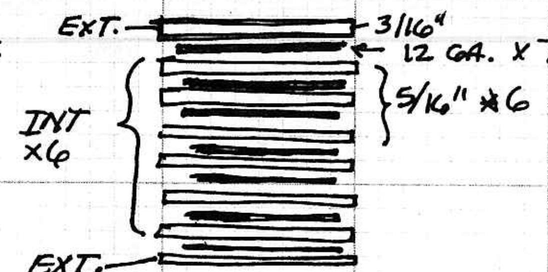
(A) $\Delta_S = 0.000006 \times 120\,FT \times 80 = .0576\,FT = 0.7"$, ANTISIPATED WALL $= 3/4"$ MAX

$\boxed{1/2 = 0.35}$ $\boxed{2/3\,m = 0.5"}$ (B) MIN $= 0.5"$

$T\,MIN = (2)(0.7 + WALL\,@\,0.75) = 1.45 \times 2 = 2.9"$

TRIAL 3" PAD.

TRIAL PAD 24" x 16" x 3" THICK



EXT = .1875" x 2
PL's = .105 x 7
INT. = .3125 x 6

$\boxed{T = 2.985" \simeq 3"}$

ELASTOMER Thickness:

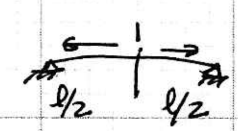
$= 2(3/16") + 6(5/16") = 2.25"$

AREA $= 24 \times 16 = 384\,in^2$, DUROMETER = Use 60 = G = 180 psi AVE.

F_S MAX ALLOWABLE $= \frac{GA\Delta_S}{h_{rt}} = \frac{180(384)(1.5)}{2.25\,(1000)} = 46.08^k$ KIPS.

(C) Δ_Z CS (CREEP) $= \pm .0005\,(L/_2\,(1/2\,SPAN))\,(12"/ft) = 0.36'$ / 2/3 movement $= 0.48"$

60'



Δ_T ANTISIPATED (MIN.) $= 0.35 + 0.5 + 0.36" = 1.21"$

$\Delta T\,MAX = .5" + .75" + .48" = 1.73"$

ΔT Ave $= (1.21 + 1.73)/2 = 1.47"$ AVE.

⇒ $\boxed{\text{TRIAL: Use 4" x 24x16" PAD}}$

EXT LAYER $= 2 \times 1/4"$, Int Layer $= 7 \times 3/8"$, PL's $= 8 \times 0.105"$ (12ga.) ⇒ $\boxed{t = 3.96 \simeq 4"}$

ROTATION θ max : δ DECK = 3.17", DL = .486", Diap = .073, COMP = .296, LL = .721 $\}\simeq 4.75"$; L/2 = 717"

$\theta = \tan^{-1}\left(\frac{4.75"}{717}\right) = 0.3796°$
$= .0066$ Rads

t_i (Elastomer) $= 3.125" \geq [2 \times 1.47"$ ANTISIPATED MOVEMENT $= 2.94]$ — OK

$t_i = 2(.25) + 7(.375) = 3.125"$

$\boxed{\text{Pad } 24 \times 16 \times 4"}$

ELASTOMERIC BEARING PAD DESIGN (PIER)
Project: SCI-823: TR234@CSXT-RR

Input Data:

width =	24.00	in
length =	16.00	in
interior layer thick. =	0.3750	in
exterior layer thick. =	0.2500	in
number of interior layers =	7	
exterior shim thick. =	0.1050	in
interior shim thick. =	0.1050	in
effective rubber thick. =	3.13	in
shear mod. "G" =	180	psi
maximum stress =	1000	psi
creep factor =	1.35	
beta factor =	1.00	

12 GAUGE

$t_{total} = (7)(.375") + (2)(0.25") + (8)(0.10$

$= 3.965" \approx \underline{4"}$

$t_i = (2)(.25") + 7(.375") = \underline{3.125"}$ ✓

Output Data:

shape factor "S" L*W/2*T(L+W) =	12.80		
allowable stress G*S/beta =	2304	psi	design based on 1000 psi maximum
compressive deflection "delta"			
compressive strain =	0.039		from AASHTO 14.4.1.2B
"delta" elastic =	0.122	in	
long term =	0.165	in	
max. movement =	1.56	in	
max. rotation 2*delta/L =	0.01523	radians	
bearing thickness =	3.97	in	
minimum plan dimension =	11.90	in	
bearing capacity =	384.00	K	based on max. stress of 1000 psi
	884.74	K	based on allowable stress
horizontal force at maximum movement =	34.56	K	
spring constants horizontal =	265	K/ft	
vertical =	37809	K/ft	

Unit Conversions and Constants:

$kip := 1000 \cdot lbf$ and $ksi := 1000 \cdot psi$

Modifying Factor for Internal Layers of Reinforced Bearings, $\beta_i := 1.0$

Modifying Factor for Cover Layers of Reinforced Bearings, $\beta_c := 1.4$

Design Parameters:

Dead Load on Pad, $P_{dl} := 176.0 \cdot kip$ and Live Load on Pad, $P_{ll} := 71.0 \cdot kip$

Total Load on Pad, $P_{tl} := P_{dl} + P_{ll}$ or $P_{tl} = 247\,kip$

Yield Strength of the Steel Reinforcement, $F_y := 50 \cdot ksi$

Allowable Stress Range, $F_{sr} := 30 \cdot ksi$ (from AASHTO Table 10.3.1A for a Nonredundant Load Path Structure)

Shear Deformation of Bearing Pad in one direction, $\Delta_s := 1.50 \cdot in$

Relative Rotation of top and bottom surfaces of Bearing due to Total Load $\theta_{tl} := 0.38 \cdot deg$ or $\theta_{tl} = 0.00663\,rad$

Elastomer Properties (from AASHTO Table 14.6.5.2-1): Durometer Hardness = 60......................

Minimum Shear Modulus, $G_{min} := 130 \cdot psi$ and Maximum Shear Modulus, $G_{max} := 200 \cdot psi$

Constant dependent on elastomer hardness, $k := 0.75$

Ratio of Creep Deflection at 25 years to Instantaneous Deflection $Creep\_to\_Instant := 0.35$

Geometry:

Gross Dimension of Rectangle Bearing Pad Parallel to the Longitudinal Axis, $L := 16 \cdot in$

Gross Dimension of Rectangle Bearing Pad Parallel to the Transverse Axis $W := 24 \cdot in$

Total Number of Elastomer Layers (2 cover layers + internal layers $N_{elastomer} := 9$

Total Number of Steel Layers (2 cover layer + internal layers) $N_{steel} := N_{elastomer} - 1$ or $N_{steel} = 8$

Thickness of Elastomer Cover Layer, $h_{rc} := 0.25 \cdot in$

Thickness of Elastomer Internal Layer, $h_{ri} := 0.375 \cdot in$

Total Elastomer Thickness, $h_{rt} := 2 \cdot h_{rc} + (N_{elastomer} - 2) \cdot h_{ri}$ or $h_{rt} = 3.125\,in$

Thickness of Steel Cover Layer, $h_{sc} := 0.105 \cdot in$

Thickness of Steel Internal Layer, $h_{si} := 0.105 \cdot in$

Total Steel Thickness, $h_{st} := 2 \cdot h_{sc} + (N_{steel} - 2) \cdot h_{si}$ or $h_{st} = 0.84\,in$

Total Pad Thickness, $h_t := h_{rt} + h_{st}$ or $h_t = 3.965\,in$

Design Pad = 24" Wide x 16" Long x 4" Thick: w/ (6) Elastomer Layers;
- (2) 1/4" Ext. Elastomeric Layers,
- (6) 3/8" Int. Elastomeric Layers,
with Stl Plates::
-(2) 12 ga Ext. Steel Plates, and,
-(6) 12ga Interior Steel Plates,
Durometer Hardness=60.

Shape Factor for one Internal Layer of a Bearing, $S_i := \frac{L \cdot W}{2 \cdot h_{ri} \cdot (L + W)}$ or $S_i = 12.8$ (AASHTO 14.2)

Shape Factor for one Cover Layer of a Bearing, $S_c := \frac{L \cdot W}{2 \cdot h_{rc} \cdot (L + W)}$ or $S_c = 19.2$ (AASHTO 14.2)

Compressive Stress (AASHTO 14.6.5.3):

$\sigma_{ll} := \frac{P_{ll}}{W \cdot L}$ or $\sigma_{ll} = 184.9\,\text{psi}$ and $\sigma_{dl} := \frac{P_{dl}}{W \cdot L}$ or $\sigma_{dl} = 458.3\,\text{psi}$ and $\sigma_{tl} := \frac{P_{tl}}{W \cdot L}$ or $\sigma_{tl} = 643.2\,\text{psi}$

Compressive Stress Limits for Internal Layers Subject to Shear Deformations-

σ_{tl_max} = the minimum of $1660 \cdot \text{psi}$, $\frac{1.66 \cdot G_{min} \cdot S_i}{\beta_i} = 2762.2\,\text{psi}$ or $\sigma_{tl\_max} := \text{if}\left(1660 \cdot \text{psi} < \frac{1.66 \cdot G_{min} \cdot S_i}{\beta_i}, 1660 \cdot \text{psi}, \frac{1.66 \cdot G_{min} \cdot S_i}{\beta_i}\right)$

or $\sigma_{tl\_max} = 1660\,\text{psi}$ $\geq$ $\sigma_{tl} = 643.2\,\text{psi}$

$\sigma_{ll\_max} := \frac{0.66 \cdot G_{min} \cdot S_i}{\beta_i}$ or $\sigma_{ll\_max} = 1098.2\,\text{psi}$ $\geq$ $\sigma_{ll} = 184.9\,\text{psi}$

Compressive Stress Limits for Cover Layers Subject to Shear Deformations-

σ_{tl_max} = the minimum of $1660 \cdot \text{psi}$, $\frac{1.66 \cdot G_{min} \cdot S_c}{\beta_c} = 2959.5\,\text{psi}$ or $\sigma_{tl\_max} := \text{if}\left(1660 \cdot \text{psi} < \frac{1.66 \cdot G_{min} \cdot S_c}{\beta_c}, 1660 \cdot \text{psi}, \frac{1.66 \cdot G_{min} \cdot S_c}{\beta_c}\right)$

or $\sigma_{tl\_max} = 1660\,\text{psi}$ $\geq$ $\sigma_{tl} = 643.2\,\text{psi}$

$\sigma_{ll\_max} := \frac{0.66 \cdot G_{min} \cdot S_c}{\beta_c}$ or $\sigma_{ll\_max} = 1176.7\,\text{psi}$ $\geq$ $\sigma_{ll} = 184.9\,\text{psi}$

Compressive Stress Limits for Internal Layers Fixed against Shear Deformations-

σ_{tl_max} = the minimum of $1660 \cdot \text{psi}$, $\frac{2.0 \cdot G_{min} \cdot S_i}{\beta_i} = 3328\,\text{psi}$ or $\sigma_{tl\_max} := \text{if}\left(1660 \cdot \text{psi} < \frac{2.0 \cdot G_{min} \cdot S_i}{\beta_i}, 1660 \cdot \text{psi}, \frac{2.0 \cdot G_{min} \cdot S_i}{\beta_i}\right)$

or $\sigma_{tl\_max} = 1660\,\text{psi}$ $\geq$ $\sigma_{tl} = 643.2\,\text{psi}$

$\sigma_{ll\_max} := \frac{1.0 \cdot G_{min} \cdot S_i}{\beta_i}$ or $\sigma_{ll\_max} = 1664\,\text{psi}$ $\geq$ $\sigma_{ll} = 184.9\,\text{psi}$

Compressive Stress Limits for Internal Layers Fixed against Shear Deformations-

σ_{tl_max} = the minimum of $1660 \cdot \text{psi}$, $\frac{2.0 \cdot G_{min} \cdot S_c}{\beta_c} = 3565.7\,\text{psi}$ or $\sigma_{tl\_max} := \text{if}\left(1660 \cdot \text{psi} < \frac{2.0 \cdot G_{min} \cdot S_c}{\beta_c}, 1660 \cdot \text{psi}, \frac{2.0 \cdot G_{min} \cdot S_c}{\beta_c}\right)$

or $\sigma_{tl\_max} = 1660\,\text{psi}$ $\geq$ $\sigma_{tl} = 643.2\,\text{psi}$

$\sigma_{ll\_max} := \frac{1.0 \cdot G_{min} \cdot S_c}{\beta_c}$ or $\sigma_{ll\_max} = 1782.9\,\text{psi}$ $\geq$ $\sigma_{ll} = 184.9\,\text{psi}$

Compressive Deflection (AASHTO 14):

Effective Compressive Modulus of Internal Elastomer Layers, $E_i := 3 \cdot G_{max} \cdot \left(1 + 2 \cdot k \cdot S_i^{\,2}\right)$ or $E_i = 148.1\,\text{ksi}$

Effective Compressive Modulus of Cover Elastomer Layers, $E_c := 3 \cdot G_{max} \cdot \left(1 + 2 \cdot k \cdot S_c^{\,2}\right)$ or $E_c = 332.4\,\text{ksi}$

Instantaneous deflection due to Live Load, $\Delta_{ll} := 2 \cdot h_{rc} \cdot \frac{\sigma_{ll}}{E_c} + \left(N_{elastomer} - 2\right) \cdot h_{ri} \cdot \frac{\sigma_{ll}}{E_i}$ or $\Delta_{ll} = 0.00356\,\text{in}$

Long-Term deflection due to Live Load, $\Delta_{ll\_long} := \Delta_{ll} \cdot \text{Creep\_to\_Instant}$ or $\Delta_{ll\_long} = 0.00124\,\text{in}$

Instantaneous deflection due to Dead Load, $\Delta_{dl} := 2 \cdot h_{rc} \cdot \frac{\sigma_{dl}}{E_c} + \left(N_{elastomer} - 2\right) \cdot h_{ri} \cdot \frac{\sigma_{dl}}{E_i}$ or $\Delta_{dl} = 0.00882\,\text{in}$

Long-Term deflection due to Dead Load, $\Delta_{dl\_long} := \Delta_{dl} \cdot \text{Creep\_to\_Instant}$ or $\Delta_{dl\_long} = 0.00309\,\text{in}$

Total Instantaneous deflection, $\Delta_c := \Delta_{ll} + \Delta_{dl}$ or $\Delta_c = 0.01237\,\text{in}$

Total Long-Term deflection, $\Delta_{c\_long} := \Delta_{ll\_long} + \Delta_{dl\_long}$ or $\Delta_{c\_long} = 0.00433\,\text{in}$

Shear (AASHTO 14.6.5.3.4):

Total Elastomer Thickness of the Bearing, $h_{rt} = 3.125\,\text{in}$

$\geq \quad 2 \cdot \Delta_s = 3\,\text{in}$

Rotation and Combined Compression and Rotation (AASHTO 14.6.5.3.5):

Relative Rotation of Top and Bottom Surfaces of Bearing, $\theta_{tl} = 0.00663\,\text{rad} \le \dfrac{2\cdot\Delta_s}{L} = 0.1875\,\text{rad}$

and

$\theta_{tl} = 0.00663\,\text{rad} \le \dfrac{2\cdot\Delta_s}{W} = 0.125\,\text{rad}$

Compressive Stress Limits for Internal Layers Subject to Shear Deformations-

$$\sigma_{tl\_max} := \frac{1.66\cdot G_{min}\cdot S_i}{\beta_i\cdot\left(1+\frac{L\cdot\theta_{tl}}{4\cdot\Delta_c}\right)} \quad \text{or} \quad \sigma_{tl\_max} = 878.5\,\text{psi} \quad \ge \quad \sigma_{tl} = 643.2\,\text{psi}$$

Compressive Stress Limits for Cover Layers Subject to Shear Deformations-

$$\sigma_{tl\_max} := \frac{1.66\cdot G_{min}\cdot S_c}{\beta_c\cdot\left(1+\frac{L\cdot\theta_{tl}}{4\cdot\Delta_c}\right)} \quad \text{or} \quad \sigma_{tl\_max} = 941.2\,\text{psi} \quad \ge \quad \sigma_{tl} = 643.2\,\text{psi}$$

Compressive Stress Limits for Internal Layers Fixed against Shear Deformations-

$$\sigma_{tl\_max} := \frac{2.0\cdot G_{min}\cdot S_i}{\beta_i\cdot\left(1+\frac{L\cdot\theta_{tl}}{4\cdot\Delta_c}\right)} \quad \text{or} \quad \sigma_{tl\_max} = 1058.4\,\text{psi} \quad \ge \quad \sigma_{tl} = 643.2\,\text{psi}$$

Compressive Stress Limits for Cover Layers Fixed against Shear Deformations-

$$\sigma_{tl\_max} := \frac{2.0\cdot G_{min}\cdot S_c}{\beta_c\cdot\left(1+\frac{L\cdot\theta_{tl}}{4\cdot\Delta_c}\right)} \quad \text{or} \quad \sigma_{tl\_max} = 1134\,\text{psi} \quad \ge \quad \sigma_{tl} = 643.2\,\text{psi}$$

Stability (AASHTO 14.6.5.3.6):

Compressive Stress Limits for Internal Layer if Bridge Deck is free to Translate Horizontally-

$$\sigma_{tl\_max} := \frac{G_{min}}{\dfrac{3.84\cdot\dfrac{h_{rt}}{L}}{S_i\cdot\sqrt{1+2\cdot\dfrac{L}{W}}} - \dfrac{2.67}{S_i\cdot(S_i+2)\cdot\left(1+\dfrac{L}{4\cdot W}\right)}}$$

or $\sigma_{tl\_max} = 4947.1\,psi$ use $\sigma_{tl\_max} := |\sigma_{tl\_max}|$

$\sigma_{tl\_max} = 4947.1\,psi$ $\geq$ $\sigma_{tl} = 643.2\,psi$

Compressive Stress Limits for Cover Layer if Bridge Deck is free to Translate Horizontally-

$$\sigma_{tl\_max} := \frac{G_{min}}{\dfrac{3.84\cdot\dfrac{h_{rt}}{L}}{S_c\cdot\sqrt{1+2\cdot\dfrac{L}{W}}} - \dfrac{2.67}{S_c\cdot(S_c+2)\cdot\left(1+\dfrac{L}{4\cdot W}\right)}}$$

or $\sigma_{tl\_max} = 6516.3\,psi$ use $\sigma_{tl\_max} := |\sigma_{tl\_max}|$

$\sigma_{tl\_max} = 6516.3\,psi$ $\geq$ $\sigma_{tl} = 643.2\,psi$

Compressive Stress Limits for Internal Layer if Bridge Deck is not free to Translate Horizontally-

$$\sigma_{tl\_max} := \frac{G_{min}}{\dfrac{1.92\cdot\dfrac{h_{rt}}{L}}{S_i\cdot\sqrt{1+2\cdot\dfrac{L}{W}}} - \dfrac{2.67}{S_i\cdot(S_i+2)\cdot\left(1+\dfrac{L}{4\cdot W}\right)}}$$

or $\sigma_{tl\_max} = 18313.5\,psi$ use $\sigma_{tl\_max} := |\sigma_{tl\_max}|$

$\sigma_{tl\_max} = 18313.5\,psi$ $\geq$ $\sigma_{tl} = 643.2\,psi$

Compressive Stress Limits for Cover Layer if Bridge Deck is free to Translate Horizontally-

$$\sigma_{tl\_max} := \frac{G_{min}}{\dfrac{1.92\cdot\dfrac{h_{rt}}{L}}{S_c\cdot\sqrt{1+2\cdot\dfrac{L}{W}}} - \dfrac{2.67}{S_c\cdot(S_c+2)\cdot\left(1+\dfrac{L}{4\cdot W}\right)}}$$

or $\sigma_{tl\_max} = 18147\,psi$ use $\sigma_{tl\_max} := |\sigma_{tl\_max}|$

or

$\sigma_{tl\_max} = 18147\,psi$ $\geq$ $\sigma_{tl} = 643.2\,psi$

Reinforcement (AASHTO 14):

Minimum Steel Thickness for Internal Layers-

h_{s_min} = maxmum of

$$\frac{1.5\cdot(h_{ri}+h_{ri})\cdot\sigma_{tl}}{F_y} = 0.01447\,\text{in}$$

$$\frac{1.5\cdot(h_{ri}+h_{ri})\cdot\sigma_{ll}}{F_{sr}} = 0.00693\,\text{in}$$

or $$h_{s\_min} := \text{if}\left[\frac{1.5\cdot(h_{ri}+h_{ri})\cdot\sigma_{tl}}{F_y} > \frac{1.5\cdot(h_{ri}+h_{ri})\cdot\sigma_{ll}}{F_{sr}}, \frac{1.5\cdot(h_{ri}+h_{ri})\cdot\sigma_{tl}}{F_y}, \frac{1.5\cdot(h_{ri}+h_{ri})\cdot\sigma_{ll}}{F_{sr}}\right]$$

$h_{s\_min} = 0.01447\,\text{in}$

$\leq \quad h_{si} = 0.105\,\text{in}$

Minimum Steel Thickness for Cover Layers-

h_{s_min} = maxmum of

$$\frac{1.5\cdot(h_{rc}+h_{ri})\cdot\sigma_{tl}}{F_y} = 0.01206\,\text{in}$$

$$\frac{1.5\cdot(h_{rc}+h_{ri})\cdot\sigma_{ll}}{F_{sr}} = 0.00578\,\text{in}$$

or $$h_{s\_min} := \text{if}\left[\frac{1.5\cdot(h_{rc}+h_{ri})\cdot\sigma_{tl}}{F_y} > \frac{1.5\cdot(h_{rc}+h_{ri})\cdot\sigma_{ll}}{F_{sr}}, \frac{1.5\cdot(h_{rc}+h_{ri})\cdot\sigma_{tl}}{F_y}, \frac{1.5\cdot(h_{rc}+h_{ri})\cdot\sigma_{ll}}{F_{sr}}\right]$$

$h_{s\_min} = 0.01206\,\text{in}$

$\leq \quad h_{sc} = 0.105\,\text{in}$

Design Forces for Supporting Structure (AASHTO 14.6):

Shear Force assuming no Positive Slip Apparatus (AASHTO 14.6.1):

$H := \frac{G_{max} \cdot (W \cdot L) \cdot \Delta_s}{h_{rt}}$ or $H = 36.9\,\mathrm{kip}$

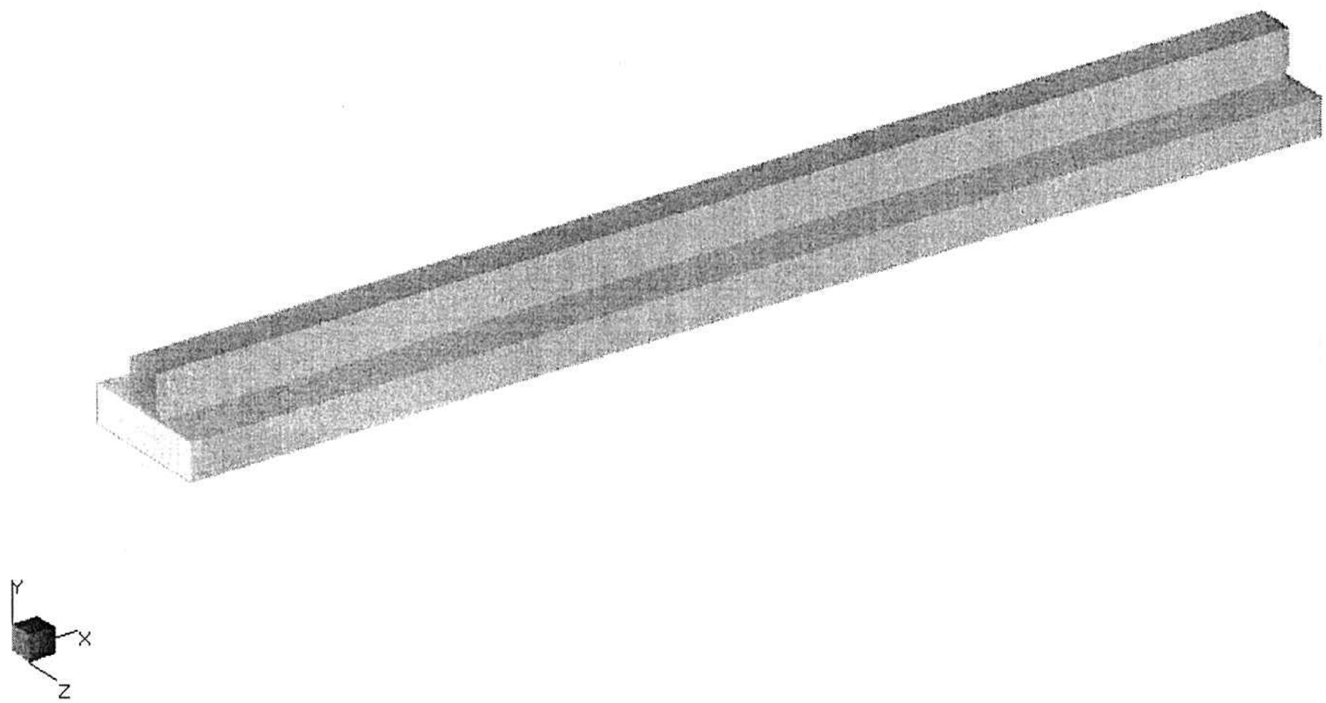
Moment (AASHTO 14.6.2):

$I := \frac{W \cdot L^3}{12}$ or $I = 8192\,\mathrm{in}^4$

$M := \frac{(0.5 \cdot E_c) \cdot I \cdot \theta_{tl}}{h_{rt}}$ or $M = 240.8\,\mathrm{kip \cdot ft}$

PROJECT: SCI-823-TR234 at CSXT RR

FULL IMAGE:



PROJECT DATA

PROJECT DATA

Project:	SCI-823-TR234 at CSXT RR
User Job No.:	5355.02
State:	OHIO
State Job No.:	PID 19415/SFN 7336934
Pier View:	Upstation.
Code:	AASHTO STANDARD (17th Edition 2002)
Comments:	Rear Abutment Pier on Spread Footing

PIER GEOMETRY

Pier Info:

Pier View: Upstation.
Pier Type: Hammer Head

Column Shape: Rectangular Non Tapered

Length(X) = 90.00 ft Height max(Y) = 3.00 ft Height min(Y) = 2.75 ft
Bottom length(X) = 89.50 ft Depth(Z) = 3.00 ft Skew angle = 0.00 Reduction of I = 1.000

SUPERSTRUCTURE INFO

Superstructure info:

Total number of spans:	1
Span number rear to current pier:	1
Number of traffic lanes:	4
Barrier height :	42.00 in
Depth of slab :	8.50 in
Curb to curb distance: 0.000	59.000 ft

Beam info:

Height in	Section area in^2	Inertia (Ixx) in^4	Inertia (Iyy) in^4	Beam CG in
72.00	956.00	616018.00	50000.00	34.43

Span #	Span length ft	Bridge Width ft
1	121.000	59.000
		59.000

BEARING POINTS

Number of bearing lines: 1

First bearing line **Eccentricity = 0.00 ft**

Point	Distance ft
1	17.88
2	25.63
3	33.38
4	41.13
5	48.88
6	56.63
7	64.38
8	72.13

MATERIAL PROPERTIES

MATERIAL PROPERTIES

	Cap	Column	Footing
Concrete Type	normal	normal	normal
Concrete Strength (psi)	4000.00	4000.00	4000.00
Concrete Density (lb/ft3)	150.00	150.00	150.00
Concrete Modulus Ec (ksi)	3834.30	3834.30	3834.30
Steel Strength Fy (ksi)	60.00	60.00	60.00

DESIGN PARAMETERS

DESIGN PARAMETERS

AASHTO STANDARD Code

Strength Reduction factors for reinf. concrete

Flexure and tension	0.90
Shear and torsion (normal)	0.85
Shear and torsion (lightweight)	0.85
Axial compression (ties)	0.70
Axial compression (spiral)	0.75

Multi presence factors for live load

1 Lane	1.00
2 Lanes	1.00
3 Lanes	0.90
more than 3 Lanes	0.75

	Crack control factor kip/ft	Clear cover in	Clear side cover in	Impact factors (auto calculation)
Cap	170.00	2.00	3.00	1.27
Column	170.00	2.00		1.27
Footing	130.00	3.00	3.00	1.00

Degree of fixity in foundations for Moment Magnify Method: Ga = 5.00

SEISMIC DESIGN PARAMETERS

Strength Reduction factors for reinf. Concrete Seismic Design

Flexure and tension	0.90
Shear and torsion (normal)	0.85
Shear and torsion (lightweight)	0.85
Axial compression (ties)	0.70
Axial compression (spiral)	0.75

Seismic Overstrength

Flexure and tension	1.30
Axial compression (ties)	1.30
Axial compression (spiral)	1.30

Response Modification Factor 3.00

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: Upstation.

Load Cases: 50

Loadcase ID: D1 Name:

Multiplier = 1.000

Column loads

Col #	Type	Dir	Mag1	y1/L	Mag2	y2/L
1	UDL	Y	-3.150 klf	0.00		1.00
1	UDL	Y	-2.400 klf	0.00		0.23
1	UDL	Y	-2.400 klf	0.78		1.00

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	-149.00
1	2	Y	-149.00
1	3	Y	-149.00
1	4	Y	-149.00
1	5	Y	-149.00
1	6	Y	-149.00
1	7	Y	-149.00
1	8	Y	-149.00

Auto generation details:

Generated Dead Load

Slab weight = 150.00 pcf
Girder weight = 150.00 pcf
Wearing weight = 3360.00 plf
Barrier load = 1060.00 plf

Loadcase ID: (L+In)1 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	-71.00
1	2	Y	-71.00
1	3	Y	-71.00
1	4	Y	-71.00
1	5	Y	-71.00
1	6	Y	-71.00
1	7	Y	-71.00
1	8	Y	-71.00

Auto generation details:

Generated Live Load

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:
HS25 truck
H25/HS25 Lane Load
Military

Transverse Positioning

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Not Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 42
Total number of Possible Combination = 6263

Loadcase ID: (L+In)2 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	-50.91
1	2	Y	-58.94
1	3	Y	-50.91
1	4	Y	-5.36
1	5	Y	0.00
1	6	Y	0.00
1	7	Y	0.00
1	8	Y	0.00

Auto generation details:

Generated Live Load

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:
HS25 truck
H25/HS25 Lane Load
Military

Transverse Positioning

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Not Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 42
Total number of Possible Combination = 6263

Loadcase ID: (L+In)3 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	0.00
1	2	Y	0.00
1	3	Y	0.00
1	4	Y	0.00
1	5	Y	-5.36
1	6	Y	-50.91
1	7	Y	-54.93
1	8	Y	-54.93

Auto generation details:

Generated Live Load

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:
HS25 truck
H25/HS25 Lane Load
Military

Transverse Positioning

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Not Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 42
Total number of Possible Combination = 6263

Loadcase ID: (L+In)4 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	0.00
1	2	Y	0.00
1	3	Y	0.00
1	4	Y	0.00
1	5	Y	0.00
1	6	Y	0.00
1	7	Y	-28.13
1	8	Y	-54.93

Auto generation details:

Generated Live Load

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:
HS25 truck
H25/HS25 Lane Load
Military

Transverse Positioning

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Not Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 42
Total number of Possible Combination = 6263

Loadcase ID: (L+In)5 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	0.00
1	2	Y	0.00
1	3	Y	0.00
1	4	Y	-8.04
1	5	Y	-50.91
1	6	Y	-61.62
1	7	Y	-45.55
1	8	Y	0.00

Auto generation details:

Generated Live Load

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:
HS25 truck
H25/HS25 Lane Load
Military

Transverse Positioning

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Not Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 42
Total number of Possible Combination = 6263

Loadcase ID: (L+In)6 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	0.00
1	2	Y	0.00
1	3	Y	0.00
1	4	Y	0.00
1	5	Y	-32.15
1	6	Y	-50.91
1	7	Y	0.00
1	8	Y	0.00

Auto generation details:

Generated Live Load

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:
HS25 truck
H25/HS25 Lane Load
Military

Transverse Positioning

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Not Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 42
Total number of Possible Combination = 6263

Loadcase ID: (L+In)7 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	0.00
1	2	Y	0.00
1	3	Y	0.00
1	4	Y	-18.09
1	5	Y	-50.64
1	6	Y	-56.67
1	7	Y	-49.43
1	8	Y	-49.43

Auto generation details:

Generated Live Load

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:

HS25 truck
H25/HS25 Lane Load
Military

Transverse Positioning

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Not Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 42
Total number of Possible Combination = 6263

Loadcase ID: (L+In)8 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	-45.82
1	2	Y	-53.05
1	3	Y	-56.67
1	4	Y	-50.64
1	5	Y	-18.09
1	6	Y	0.00
1	7	Y	0.00
1	8	Y	0.00

Auto generation details:

Generated Live Load

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:
HS25 truck
H25/HS25 Lane Load
Military

Transverse Positioning

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Not Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 42
Total number of Possible Combination = 6263

Loadcase ID: (L+In)9 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	0.00
1	2	Y	0.00
1	3	Y	0.00
1	4	Y	-48.23
1	5	Y	-34.83
1	6	Y	0.00
1	7	Y	0.00
1	8	Y	0.00

Auto generation details:

Generated Live Load

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:
HS25 truck
H25/HS25 Lane Load
Military

Transverse Positioning

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Not Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 42
Total number of Possible Combination = 6263

Loadcase ID: (L+In)10 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	0.00
1	2	Y	0.00
1	3	Y	-50.91
1	4	Y	-32.15
1	5	Y	0.00
1	6	Y	0.00
1	7	Y	0.00
1	8	Y	0.00

Auto generation details:

Generated Live Load

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:

HS25 truck
H25/HS25 Lane Load
Military

Transverse Positioning

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Not Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 42
Total number of Possible Combination = 6263

Loadcase ID: (L+In)11 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	0.00
1	2	Y	-45.55
1	3	Y	-61.62
1	4	Y	-50.91
1	5	Y	-8.04
1	6	Y	0.00
1	7	Y	0.00
1	8	Y	0.00

Auto generation details:

Generated Live Load

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:
HS25 truck
H25/HS25 Lane Load
Military

Transverse Positioning

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Not Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 42
Total number of Possible Combination = 6263

Loadcase ID: (L+In)12 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	-50.91
1	2	Y	-32.15
1	3	Y	0.00
1	4	Y	0.00
1	5	Y	0.00
1	6	Y	0.00
1	7	Y	0.00
1	8	Y	0.00

Auto generation details:

Generated Live Load

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:
HS25 truck
H25/HS25 Lane Load
Military

Transverse Positioning

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Not Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 42
Total number of Possible Combination = 6263

Loadcase ID: (L+In)13 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	-28.13
1	2	Y	-50.91
1	3	Y	-4.02
1	4	Y	0.00
1	5	Y	0.00
1	6	Y	0.00
1	7	Y	0.00
1	8	Y	0.00

Auto generation details:

Generated Live Load

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:
HS25 truck
H25/HS25 Lane Load
Military

Transverse Positioning

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Not Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 42
Total number of Possible Combination = 6263

Loadcase ID: W1   Name: Angle: 75

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-2.72 kips	0.50		
UDL	Z		0.11 klf	0.00		1.00

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.85
1	1	Y	-0.64
1	1	Z	1.70
1	2	X	-0.85
1	2	Y	-0.00
1	2	Z	1.70
1	3	X	-0.85
1	3	Y	-0.00
1	3	Z	1.70
1	4	X	-0.85
1	4	Y	-0.00
1	4	Z	1.70
1	5	X	-0.85
1	5	Y	-0.00
1	5	Z	1.70
1	6	X	-0.85
1	6	Y	-0.00
1	6	Z	1.70
1	7	X	-0.85
1	7	Y	-0.00
1	7	Z	1.70
1	8	X	-0.85
1	8	Y	0.64
1	8	Z	1.70

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 75.00 deg Elevation above which wind load acts = 17.50 ft

Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 11.000 psf Cap 40.000 psf
Longitudinal 22.000 psf Column 40.000 psf
Overturning not considered

Loadcase ID: W2 Name: Angle: 60

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-4.77 kips	0.50		
UDL	Z		0.09 klf	0.00		1.00

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-1.31
1	1	Y	-0.99
1	1	Z	1.47
1	2	X	-1.31
1	2	Y	-0.00
1	2	Z	1.47
1	3	X	-1.31
1	3	Y	-0.00
1	3	Z	1.47
1	4	X	-1.31
1	4	Y	-0.00
1	4	Z	1.47
1	5	X	-1.31
1	5	Y	-0.00
1	5	Z	1.47
1	6	X	-1.31
1	6	Y	-0.00
1	6	Z	1.47
1	7	X	-1.31
1	7	Y	-0.00
1	7	Z	1.47
1	8	X	-1.31
1	8	Y	0.99
1	8	Z	1.47

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 60.00 deg Elevation above which wind load acts = 17.50 ft
Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 17.000 psf Cap 40.000 psf
Longitudinal 19.000 psf Column 40.000 psf
Overturning not considered

Loadcase ID: W3 Name: Angle: 45

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-5.58 kips	0.50		
UDL	Z		0.06 klf	0.00		1.00

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-2.55
1	1	Y	-1.92
1	1	Z	1.24
1	2	X	-2.55
1	2	Y	-0.00
1	2	Z	1.24
1	3	X	-2.55
1	3	Y	-0.00
1	3	Z	1.24
1	4	X	-2.55
1	4	Y	-0.00
1	4	Z	1.24
1	5	X	-2.55
1	5	Y	-0.00
1	5	Z	1.24
1	6	X	-2.55
1	6	Y	-0.00
1	6	Z	1.24
1	7	X	-2.55

Line #	Bearing #	Dir	Load kips
1	7	Y	-0.00
1	7	Z	1.24
1	8	X	-2.55
1	8	Y	1.92
1	8	Z	1.24

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 45.00 deg Elevation above which wind load acts = 17.50 ft
Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 33.000 psf Cap 40.000 psf
Longitudinal 16.000 psf Column 40.000 psf
Overturning not considered

Loadcase ID: W4 Name: Angle: 30

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-4.95 kips	0.50		
UDL	Z		0.03 klf	0.00		1.00

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-3.17
1	1	Y	-2.38
1	1	Z	0.93
1	2	X	-3.17
1	2	Y	-0.00
1	2	Z	0.93
1	3	X	-3.17
1	3	Y	-0.00
1	3	Z	0.93

Line #	Bearing #	Dir	Load kips
1	4	X	-3.17
1	4	Y	-0.00
1	4	Z	0.93
1	5	X	-3.17
1	5	Y	-0.00
1	5	Z	0.93
1	6	X	-3.17
1	6	Y	-0.00
1	6	Z	0.93
1	7	X	-3.17
1	7	Y	-0.00
1	7	Z	0.93
1	8	X	-3.17
1	8	Y	2.38
1	8	Z	0.93

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 30.00 deg Elevation above which wind load acts = 17.50 ft
Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 41.000 psf Cap 40.000 psf
Longitudinal 12.000 psf Column 40.000 psf
Overturning not considered

Loadcase ID: W5 Name: Angle: 15

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-3.04 kips	0.50		
UDL	Z		0.01 klf	0.00		1.00

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-3.40
1	1	Y	-2.56
1	1	Z	0.46
1	2	X	-3.40
1	2	Y	-0.00
1	2	Z	0.46
1	3	X	-3.40
1	3	Y	-0.00
1	3	Z	0.46
1	4	X	-3.40
1	4	Y	-0.00
1	4	Z	0.46
1	5	X	-3.40
1	5	Y	-0.00
1	5	Z	0.46
1	6	X	-3.40
1	6	Y	-0.00
1	6	Z	0.46
1	7	X	-3.40
1	7	Y	-0.00
1	7	Z	0.46
1	8	X	-3.40
1	8	Y	2.56
1	8	Z	0.46

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 15.00 deg Elevation above which wind load acts = 17.50 ft
Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 44.000 psf Cap 40.000 psf
Longitudinal 6.000 psf Column 40.000 psf
Overturning not considered

Loadcase ID: W6 Name: Angle: 0

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-0.36 kips	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-3.86
1	1	Y	-13.39
1	1	Z	0.00
1	2	X	-3.86
1	2	Y	8.92
1	2	Z	0.00
1	3	X	-3.86
1	3	Y	8.92
1	3	Z	0.00
1	4	X	-3.86
1	4	Y	8.92
1	4	Z	0.00
1	5	X	-3.86
1	5	Y	8.92
1	5	Z	0.00
1	6	X	-3.86
1	6	Y	8.92
1	6	Z	0.00
1	7	X	-3.86
1	7	Y	8.92
1	7	Z	0.00
1	8	X	-3.86
1	8	Y	31.24
1	8	Z	0.00

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 0.00 deg Elevation above which wind load acts = 17.50 ft
Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 50.000 psf Cap 40.000 psf
Longitudinal 0.000 psf Column 40.000 psf
Overturning 20.000 psf

Loadcase ID: W7 Name: Angle: -15

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-3.04 kips	0.50		
UDL	Z		-0.01 klf	0.00		1.00

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-3.40
1	1	Y	-2.56
1	1	Z	-0.46
1	2	X	-3.40
1	2	Y	-0.00
1	2	Z	-0.46
1	3	X	-3.40
1	3	Y	-0.00
1	3	Z	-0.46
1	4	X	-3.40
1	4	Y	-0.00
1	4	Z	-0.46
1	5	X	-3.40
1	5	Y	-0.00
1	5	Z	-0.46
1	6	X	-3.40
1	6	Y	-0.00
1	6	Z	-0.46
1	7	X	-3.40
1	7	Y	-0.00
1	7	Z	-0.46
1	8	X	-3.40
1	8	Y	2.56
1	8	Z	-0.46

Auto generation details:

Generated Wind Load on Structure

Angle of wind = -15.00 deg Elevation above which wind load acts = 17.50 ft
Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 44.000 psf Cap 40.000 psf
Longitudinal 6.000 psf Column 40.000 psf
Overturning not considered

Loadcase ID: W8 Name: Angle: -30

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-4.95 kips	0.50		
UDL	Z		-0.03 klf	0.00		1.00

Bearing loads

Line #	Bearing #	Dir.	Load kips
1	1	X	-3.17
1	1	Y	-2.38
1	1	Z	-0.93
1	2	X	-3.17
1	2	Y	-0.00
1	2	Z	-0.93
1	3	X	-3.17
1	3	Y	-0.00
1	3	Z	-0.93
1	4	X	-3.17
1	4	Y	-0.00
1	4	Z	-0.93
1	5	X	-3.17
1	5	Y	-0.00
1	5	Z	-0.93
1	6	X	-3.17
1	6	Y	-0.00
1	6	Z	-0.93
1	7	X	-3.17
1	7	Y	-0.00
1	7	Z	-0.93
1	8	X	-3.17
1	8	Y	2.38
1	8	Z	-0.93

Auto generation details:

Generated Wind Load on Structure

Angle of wind = -30.00 deg Elevation above which wind load acts = 17.50 ft

Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 41.000 psf Cap 40.000 psf
Longitudinal 12.000 psf Column 40.000 psf
Overturning not considered

Loadcase ID: W9 Name: Angle: -45

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-5.58 kips	0.50		
UDL	Z		-0.06 klf	0.00		1.00

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-2.55
1	1	Y	-1.92
1	1	Z	-1.24
1	2	X	-2.55
1	2	Y	-0.00
1	2	Z	-1.24
1	3	X	-2.55
1	3	Y	-0.00
1	3	Z	-1.24
1	4	X	-2.55
1	4	Y	-0.00
1	4	Z	-1.24
1	5	X	-2.55
1	5	Y	-0.00
1	5	Z	-1.24
1	6	X	-2.55
1	6	Y	-0.00
1	6	Z	-1.24
1	7	X	-2.55
1	7	Y	-0.00
1	7	Z	-1.24
1	8	X	-2.55
1	8	Y	1.92
1	8	Z	-1.24

Auto generation details:

Generated Wind Load on Structure

Angle of wind = -45.00 deg Elevation above which wind load acts = 17.50 ft
Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 33.000 psf Cap 40.000 psf
Longitudinal 16.000 psf Column 40.000 psf
Overturning not considered

Loadcase ID: W10 Name: Angle: -60

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-4.77 kips	0.50		
UDL	Z		-0.09 klf	0.00		1.00

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-1.31
1	1	Y	-0.99
1	1	Z	-1.47
1	2	X	-1.31
1	2	Y	-0.00
1	2	Z	-1.47
1	3	X	-1.31
1	3	Y	-0.00
1	3	Z	-1.47
1	4	X	-1.31
1	4	Y	-0.00
1	4	Z	-1.47
1	5	X	-1.31
1	5	Y	-0.00
1	5	Z	-1.47
1	6	X	-1.31
1	6	Y	-0.00
1	6	Z	-1.47
1	7	X	-1.31

Line #	Bearing #	Dir	Load kips
1	7	Y	-0.00
1	7	Z	-1.47
1	8	X	-1.31
1	8	Y	0.99
1	8	Z	-1.47

Auto generation details:

Generated Wind Load on Structure

Angle of wind = -60.00 deg Elevation above which wind load acts = 17.50 ft
Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 17.000 psf Cap 40.000 psf
Longitudinal 19.000 psf Column 40.000 psf
Overturning not considered

Loadcase ID: W11 Name: Angle: -75

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-2.72 kips	0.50		
UDL	Z		-0.11 klf	0.00		1.00

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.85
1	1	Y	-0.64
1	1	Z	-1.70
1	2	X	-0.85
1	2	Y	-0.00
1	2	Z	-1.70
1	3	X	-0.85
1	3	Y	-0.00
1	3	Z	-1.70

Line #	Bearing #	Dir	Load kips
1	4	X	-0.85
1	4	Y	-0.00
1	4	Z	-1.70
1	5	X	-0.85
1	5	Y	-0.00
1	5	Z	-1.70
1	6	X	-0.85
1	6	Y	-0.00
1	6	Z	-1.70
1	7	X	-0.85
1	7	Y	-0.00
1	7	Z	-1.70
1	8	X	-0.85
1	8	Y	0.64
1	8	Z	-1.70

Auto generation details:

Generated Wind Load on Structure

Angle of wind = -75.00 deg Elevation above which wind load acts = 17.50 ft
Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 11.000 psf Cap 40.000 psf
Longitudinal 22.000 psf Column 40.000 psf
Overturning not considered

Loadcase ID: WL1 Name: Angle: 75

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.11
1	1	Y	-0.20
1	1	Z	0.32
1	2	X	-0.11
1	2	Y	-0.00
1	2	Z	0.32

Line #	Bearing #	Dir	Load kips
1	3	X	-0.11
1	3	Y	-0.00
1	3	Z	0.32
1	4	X	-0.11
1	4	Y	-0.00
1	4	Z	0.32
1	5	X	-0.11
1	5	Y	-0.00
1	5	Z	0.32
1	6	X	-0.11
1	6	Y	-0.00
1	6	Z	0.32
1	7	X	-0.11
1	7	Y	-0.00
1	7	Z	0.32
1	8	X	-0.11
1	8	Y	0.20
1	8	Z	0.32

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = 75.00 deg Live load length = 60.50 ft

Loadcase ID: WL2 Name: Angle: 60

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.26
1	1	Y	-0.48
1	1	Z	0.29
1	2	X	-0.26
1	2	Y	-0.00
1	2	Z	0.29
1	3	X	-0.26
1	3	Y	-0.00
1	3	Z	0.29

Line #	Bearing #	Dir	Load kips
1	4	X	-0.26
1	4	Y	-0.00
1	4	Z	0.29
1	5	X	-0.26
1	5	Y	-0.00
1	5	Z	0.29
1	6	X	-0.26
1	6	Y	-0.00
1	6	Z	0.29
1	7	X	-0.26
1	7	Y	-0.00
1	7	Z	0.29
1	8	X	-0.26
1	8	Y	0.48
1	8	Z	0.29

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = 60.00 deg Live load length = 60.50 ft

Loadcase ID: WL3 Name: Angle: 45

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.50
1	1	Y	-0.94
1	1	Z	0.24
1	2	X	-0.50
1	2	Y	-0.00
1	2	Z	0.24
1	3	X	-0.50
1	3	Y	-0.00
1	3	Z	0.24
1	4	X	-0.50
1	4	Y	-0.00
1	4	Z	0.24

Line #	Bearing #	Dir	Load kips
1	5	X	-0.50
1	5	Y	-0.00
1	5	Z	0.24
1	6	X	-0.50
1	6	Y	-0.00
1	6	Z	0.24
1	7	X	-0.50
1	7	Y	-0.00
1	7	Z	0.24
1	8	X	-0.50
1	8	Y	0.94
1	8	Z	0.24

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = 45.00 deg Live load length = 60.50 ft

Loadcase ID: WL4 Name: Angle: 30

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.62
1	1	Y	-1.16
1	1	Z	0.18
1	2	X	-0.62
1	2	Y	-0.00
1	2	Z	0.18
1	3	X	-0.62
1	3	Y	-0.00
1	3	Z	0.18
1	4	X	-0.62
1	4	Y	-0.00
1	4	Z	0.18
1	5	X	-0.62
1	5	Y	-0.00
1	5	Z	0.18

Line #	Bearing #	Dir	Load kips
1	6	X	-0.62
1	6	Y	-0.00
1	6	Z	0.18
1	7	X	-0.62
1	7	Y	-0.00
1	7	Z	0.18
1	8	X	-0.62
1	8	Y	1.16
1	8	Z	0.18

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = 30.00 deg Live load length = 60.50 ft

Loadcase ID: WL5 Name: Angle: 15

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.67
1	1	Y	-1.25
1	1	Z	0.09
1	2	X	-0.67
1	2	Y	-0.00
1	2	Z	0.09
1	3	X	-0.67
1	3	Y	-0.00
1	3	Z	0.09
1	4	X	-0.67
1	4	Y	-0.00
1	4	Z	0.09
1	5	X	-0.67
1	5	Y	-0.00
1	5	Z	0.09
1	6	X	-0.67
1	6	Y	-0.00
1	6	Z	0.09

Line #	Bearing #	Dir	Load kips
1	7	X	-0.67
1	7	Y	-0.00
1	7	Z	0.09
1	8	X	-0.67
1	8	Y	1.25
1	8	Z	0.09

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = 15.00 deg Live load length = 60.50 ft

Loadcase ID: WL6 Name: Angle: 0

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.76
1	1	Y	-1.42
1	1	Z	0.00
1	2	X	-0.76
1	2	Y	-0.00
1	2	Z	0.00
1	3	X	-0.76
1	3	Y	-0.00
1	3	Z	0.00
1	4	X	-0.76
1	4	Y	-0.00
1	4	Z	0.00
1	5	X	-0.76
1	5	Y	-0.00
1	5	Z	0.00
1	6	X	-0.76
1	6	Y	-0.00
1	6	Z	0.00
1	7	X	-0.76
1	7	Y	-0.00
1	7	Z	0.00

Line #	Bearing #	Dir	Load kips
1	8	X	-0.76
1	8	Y	1.42
1	8	Z	0.00

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = 0.00 deg Live load length = 60.50 ft

Loadcase ID: WL7 Name: Angle: -15

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.67
1	1	Y	-1.25
1	1	Z	-0.09
1	2	X	-0.67
1	2	Y	-0.00
1	2	Z	-0.09
1	3	X	-0.67
1	3	Y	-0.00
1	3	Z	-0.09
1	4	X	-0.67
1	4	Y	-0.00
1	4	Z	-0.09
1	5	X	-0.67
1	5	Y	-0.00
1	5	Z	-0.09
1	6	X	-0.67
1	6	Y	-0.00
1	6	Z	-0.09
1	7	X	-0.67
1	7	Y	-0.00
1	7	Z	-0.09
1	8	X	-0.67
1	8	Y	1.25
1	8	Z	-0.09

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = -15.00 deg Live load length = 60.50 ft

Loadcase ID: WL8 Name: Angle: -30

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.62
1	1	Y	-1.16
1	1	Z	-0.18
1	2	X	-0.62
1	2	Y	-0.00
1	2	Z	-0.18
1	3	X	-0.62
1	3	Y	-0.00
1	3	Z	-0.18
1	4	X	-0.62
1	4	Y	-0.00
1	4	Z	-0.18
1	5	X	-0.62
1	5	Y	-0.00
1	5	Z	-0.18
1	6	X	-0.62
1	6	Y	-0.00
1	6	Z	-0.18
1	7	X	-0.62
1	7	Y	-0.00
1	7	Z	-0.18
1	8	X	-0.62
1	8	Y	1.16
1	8	Z	-0.18

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = -30.00 deg Live load length = 60.50 ft

Loadcase ID: WL9 Name: Angle: -45

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.50
1	1	Y	-0.94
1	1	Z	-0.24
1	2	X	-0.50
1	2	Y	-0.00
1	2	Z	-0.24
1	3	X	-0.50
1	3	Y	-0.00
1	3	Z	-0.24
1	4	X	-0.50
1	4	Y	-0.00
1	4	Z	-0.24
1	5	X	-0.50
1	5	Y	-0.00
1	5	Z	-0.24
1	6	X	-0.50
1	6	Y	-0.00
1	6	Z	-0.24
1	7	X	-0.50
1	7	Y	-0.00
1	7	Z	-0.24
1	8	X	-0.50
1	8	Y	0.94
1	8	Z	-0.24

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = -45.00 deg Live load length = 60.50 ft

Loadcase ID: WL10 Name: Angle: -60

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir.	Load kips
1	1	X	-0.26
1	1	Y	-0.48
1	1	Z	-0.29
1	2	X	-0.26
1	2	Y	-0.00
1	2	Z	-0.29
1	3	X	-0.26
1	3	Y	-0.00
1	3	Z	-0.29
1	4	X	-0.26
1	4	Y	-0.00
1	4	Z	-0.29
1	5	X	-0.26
1	5	Y	-0.00
1	5	Z	-0.29
1	6	X	-0.26
1	6	Y	-0.00
1	6	Z	-0.29
1	7	X	-0.26
1	7	Y	-0.00
1	7	Z	-0.29
1	8	X	-0.26
1	8	Y	0.48
1	8	Z	-0.29

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = -60.00 deg Live load length = 60.50 ft

Loadcase ID: WL11 Name: Angle: -75

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.11
1	1	Y	-0.20
1	1	Z	-0.32
1	2	X	-0.11
1	2	Y	-0.00
1	2	Z	-0.32
1	3	X	-0.11
1	3	Y	-0.00
1	3	Z	-0.32
1	4	X	-0.11
1	4	Y	-0.00
1	4	Z	-0.32
1	5	X	-0.11
1	5	Y	-0.00
1	5	Z	-0.32
1	6	X	-0.11
1	6	Y	-0.00
1	6	Z	-0.32
1	7	X	-0.11
1	7	Y	-0.00
1	7	Z	-0.32
1	8	X	-0.11
1	8	Y	0.20
1	8	Z	-0.32

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = -75.00 deg Live load length = 60.50 ft

Loadcase ID: LF1 Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Moment	X		-135.15 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Z	-1.33
1	2	Z	-1.33
1	3	Z	-1.33
1	4	Z	-1.33
1	5	Z	-1.33
1	6	Z	-1.33
1	7	Z	-1.33
1	8	Z	-1.33

Auto generation details:

Selected Live Load for LF generation

Load: H25/HS25 Lane Load
Number of loaded lanes = 4
Contributing longitudinal length = 60.50 ft

Loadcase ID: LF2 Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Moment	X		-135.15 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Z	-1.33
1	2	Z	-1.33
1	3	Z	-1.33
1	4	Z	-1.33

Line #	Bearing #	Dir	Load kips
1	5	Z	-1.33
1	6	Z	-1.33
1	7	Z	-1.33
1	8	Z	-1.33

Auto generation details:

Selected Live Load for LF generation

Number of loaded lanes = all combinations
Contributing longitudinal length = 0.00 ft

Loadcase ID: LF3 Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Moment	X		-135.15 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Z	-1.33
1	2	Z	-1.33
1	3	Z	-1.33
1	4	Z	-1.33
1	5	Z	-1.33
1	6	Z	-1.33
1	7	Z	-1.33
1	8	Z	-1.33

Auto generation details:

Selected Live Load for LF generation

Number of loaded lanes = all combinations
Contributing longitudinal length = 0.00 ft

Loadcase ID: LF4 Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Moment	X		-135.15 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Z	-1.33
1	2	Z	-1.33
1	3	Z	-1.33
1	4	Z	-1.33
1	5	Z	-1.33
1	6	Z	-1.33
1	7	Z	-1.33
1	8	Z	-1.33

Auto generation details:

Selected Live Load for LF generation

Number of loaded lanes = all combinations
Contributing longitudinal length = 0.00 ft

Loadcase ID: LF5 Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Moment	X		-135.15 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Z	-1.33
1	2	Z	-1.33
1	3	Z	-1.33
1	4	Z	-1.33
1	5	Z	-1.33
1	6	Z	-1.33
1	7	Z	-1.33
1	8	Z	-1.33

Auto generation details:

Selected Live Load for LF generation

Number of loaded lanes = all combinations
Contributing longitudinal length = 0.00 ft

Loadcase ID: LF6 Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Moment	X		-135.15 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips

Line #	Bearing #	Dir	Load kips
1	1	Z	-1.33
1	2	Z	-1.33
1	3	Z	-1.33
1	4	Z	-1.33
1	5	Z	-1.33
1	6	Z	-1.33
1	7	Z	-1.33
1	8	Z	-1.33

Auto generation details:

Selected Live Load for LF generation

Number of loaded lanes = all combinations
Contributing longitudinal length = 0.00 ft

Loadcase ID: LF7 Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Moment	X		-135.15 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Z	-1.33
1	2	Z	-1.33
1	3	Z	-1.33
1	4	Z	-1.33
1	5	Z	-1.33
1	6	Z	-1.33
1	7	Z	-1.33
1	8	Z	-1.33

Auto generation details:

Selected Live Load for LF generation

Number of loaded lanes = all combinations
Contributing longitudinal length = 0.00 ft

Loadcase ID: LF8 Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Moment	X		-135.15 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Z	-1.33
1	2	Z	-1.33
1	3	Z	-1.33
1	4	Z	-1.33
1	5	Z	-1.33
1	6	Z	-1.33
1	7	Z	-1.33
1	8	Z	-1.33

Auto generation details:

Selected Live Load for LF generation

Number of loaded lanes = all combinations
Contributing longitudinal length = 0.00 ft

Loadcase ID: LF9 Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Moment	X		-135.15 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Z	-1.33
1	2	Z	-1.33
1	3	Z	-1.33
1	4	Z	-1.33
1	5	Z	-1.33
1	6	Z	-1.33
1	7	Z	-1.33
1	8	Z	-1.33

Auto generation details:

Selected Live Load for LF generation

Number of loaded lanes = all combinations
Contributing longitudinal length = 0.00 ft

Loadcase ID: LF10 Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Moment	X		-135.15 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Z	-1.33
1	2	Z	-1.33
1	3	Z	-1.33
1	4	Z	-1.33
1	5	Z	-1.33
1	6	Z	-1.33
1	7	Z	-1.33
1	8	Z	-1.33

Auto generation details:

Selected Live Load for LF generation

Number of loaded lanes = all combinations
Contributing longitudinal length = 0.00 ft

Loadcase ID: LF11 Name:

Multiplier = 1.000

Cap loads

Type	Dir.	Arm ft	Mag1	x1/L	Mag2	x2/L
Moment	X		-135.15 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Z	-1.33
1	2	Z	-1.33
1	3	Z	-1.33
1	4	Z	-1.33
1	5	Z	-1.33
1	6	Z	-1.33
1	7	Z	-1.33
1	8	Z	-1.33

Auto generation details:

Selected Live Load for LF generation

Number of loaded lanes = all combinations
Contributing longitudinal length = 0.00 ft

Loadcase ID: LF12 Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Moment	X		-135.15 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Z	-1.33
1	2	Z	-1.33
1	3	Z	-1.33
1	4	Z	-1.33
1	5	Z	-1.33
1	6	Z	-1.33
1	7	Z	-1.33
1	8	Z	-1.33

Auto generation details:

Selected Live Load for LF generation

Number of loaded lanes = all combinations
Contributing longitudinal length = 0.00 ft

Loadcase ID: LF13 Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Moment	X		-135.15 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Z	-1.33
1	2	Z	-1.33
1	3	Z	-1.33
1	4	Z	-1.33
1	5	Z	-1.33
1	6	Z	-1.33
1	7	Z	-1.33
1	8	Z	-1.33

Auto generation details:

Selected Live Load for LF generation

Number of loaded lanes = all combinations
Contributing longitudinal length = 0.00 ft

Loadcase ID: T1 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	0.00
1	1	Z	-5.89
1	2	X	0.00
1	2	Z	-5.89
1	3	X	0.00

Line #	Bearing #	Dir	Load kips
1	3	Z	-5.89
1	4	X	0.00
1	4	Z	-5.89
1	5	X	0.00
1	5	Z	-5.89
1	6	X	0.00
1	6	Z	-5.89
1	7	X	0.00
1	7	Z	-5.89
1	8	X	0.00
1	8	Z	-5.89

Auto generation details:

Bearing type

Elastomeric Bearings
Direction of thermal force: +(Z)
Length of Superstructure Contributing, L: 82.000 ft
Change in temperature: 80.000 °F
Coefficient of thermal expansion: 6.0e-006 ft/°F
Area of bearing: 384.00 in^2
Shear modulus of Elastomer: 0.13 kips
Total Elastomer Thickness: 4.00 in

Selected load groups

SERVICE GROUP I
SERVICE GROUP II
SERVICE GROUP III
SERVICE GROUP IV
SERVICE GROUP V
SERVICE GROUP VI
SERVICE GROUP VII
SERVICE GROUP VIII
SERVICE GROUP IX
SERVICE GROUP X
LOAD FACTOR GROUP I
LOAD FACTOR GROUP II
LOAD FACTOR GROUP III
LOAD FACTOR GROUP IV
LOAD FACTOR GROUP V
LOAD FACTOR GROUP VI
LOAD FACTOR GROUP VII
LOAD FACTOR GROUP VIII
LOAD FACTOR GROUP IX
LOAD FACTOR GROUP X

CAP DESIGN

CAP DESIGN

Code: AASHTO STANDARD (17th Edition 2002) - Ultimate Strength Design
Units: US
Pier View: Upstation.

DESIGN PARAMETERS

f'c = 4000.0 psi
Fy flex = 60000.0 psi — Fy shear = 60000.0 psi
phi flex = 0.90 — phi shear = 0.85
Ec = 3834.3 ksi — Es = 29000.0 ksi
crack control factor z = 170.00 kips / in
Concrete Type : Normal Weight.
Design of cap at face of column.

CAP GEOMETRY

Hammer Head Cap : Length(X) = 90.00 ft Depth(Z) = 36.00 in

MAIN REINFORCEMENT

	Bar size	Quantity	Bar dist. in	As total in^2	From ft	To ft	Hook
TOP							
	# 8	5	3.00	3.950	0.50	89.50	None

STIRRUPS

From ft	To ft	Stirrup Size	n legs	Spacing in	Aprv/s in^2 / ft
0.50	89.50	# 4	2	24.00	0.20

Clear Cover on Sides = 2.00 in

FLEXURE DESIGN

Span 1: From 0.00 ft To 45.00 ft

Loc ft	AbsLoc ft	H in	Mmax Mmin kips-ft	pMn kips-ft	Comb	Asb-req in^2	Asb-prv in^2	Asb-eff in^2	Ast-req in^2	Ast-prv in^2	Ast-eff in^2
0.3	0.3	36	0.0	0.0	0	0.00	0.00	0.00	0.00	0.00	0.00
			-0.1	-0.1	1264	0.00	0.00	0.00	0.00	0.00*	0.00*

Span 2: From 45.00 ft To 90.00 ft

Loc ft	AbsLoc ft	H in	Mmax Mmin kips-ft	pMn kips-ft	Comb	Asb-req in^2	Asb-prv in^2	Asb-eff in^2	Ast-req in^2	Ast-prv in^2	Ast-eff in^2
44.8	89.8	36	0.0	0.0	0	0.00	0.00	0.00	0.00	0.00	0.00
			-0.1	-0.1	1264	0.00	0.00	0.00	0.00	0.00	0.00

Flexure Design : Notes

* The provided reinforcement is not adequate, either less than required or larger than maximum allowed.

SHEAR AND TORSION DESIGN

Span 1: From 0.00 ft To 45.00 ft

Loc ft	AbsLoc ft	Pos	Vu kips	Comb	Tu kips-ft	Comb	phi*Vc kips	T-lim kips-ft	Avs/s in^2/ft	2Ats/s in^2/ft	Av/s in^2/ft	Aprv/s in^2/ft	Alt in^2
0.25	0.25	L	0.4	1264	0.0	0	139.3	52.3	0.00	0.00	0.00	0.00	0.00

Span 2: From 45.00 ft To 90.00 ft

Loc ft	AbsLoc ft	Pos	Vu kips	Comb	Tu kips-ft	Comb	phi*Vc kips	T-lim kips-ft	Avs/s in^2/ft	2Ats/s in^2/ft	Av/s in^2/ft	Aprv/s in^2/ft	Alt in^2
44.75	89.75	R	0.4	1264	0.0	0	139.3	52.3	0.00	0.00	0.00	0.00	0.00

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.
- 2Ats/s is the required area of steel per unit length for two legs 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 steel per unit length due to shear (stirrups).
- Alt is the total longitudinal steel required due to torsion in addition to the REQUIRED flexural steel.

CRACKING/FATIGUE CHECK

Span 1: From 0.00 ft To 45.00 ft

Loc ft	AbsLoc ft	H in	Cracking fs-t fs-b ksi	Cracking ratio fs-t ratio fs-b	Cracking Comb	Fatigue fs-t fs-b ksi	Fatigue ratio fs-t ratio fs-b
0.25	0.3	36.0	0.0	-0.00	0	0.0	0.00
			0.0	-0.00	0	0.0	0.00

Span 2: From 45.00 ft To 90.00 ft

Loc ft	AbsLoc ft	H in	Cracking fs-t fs-b ksi	Cracking ratio fs-t ratio fs-b	Cracking Comb	Fatigue fs-t fs-b ksi	Fatigue ratio fs-t ratio fs-b
44.75	89.8	36.0	0.0	-0.00	0	0.0	0.00
			0.0	-0.00	0	0.0	0.00

* Cracking / fatigue checking failed.

COLUMN DESIGN

COLUMN DESIGN - Column: 1

Code: AASHTO STANDARD (17th Edition 2002) - Factored Load Design
Units: US
Pier View: Upstation.
Design/Analysis Method: No Slenderness Considered.

Column Type: Rectangular 1074.00 x 36.00 in

DESIGN PARAMETERS

f'c = 4000.0 psi	fy = 60000.0 psi
phi flex = 0.90	phi axial = 0.70
Ec = 3834.3 ksi	Es = 29000 ksi
Concrete Type : Normal Weight.	

Reinforcement

Rebar Pattern: Rectangular
Rebar Orientation: Face Parallel

Reinforcement Schedule

Layer	Dir	Size	No. bars	Bar Dist. in
1	X	5	45	4.00
1	Z	5	3	4.00

Reinforcement summary

Main bars summary:
92 # 5 bars
Total number of bars in the column: 92
Ties size: # 3

Design values used - (e-min effect included).

Loc ft	Comb	Fx kips	Fy kips	Fz kips	Mx kips-ft	My kips-ft	Mz kips-ft
0.00	1C	0.0	3310.9	0.0	-463.5	0.0	-9055.3
2.13	1C	-0.0	3273.0	0.0	-458.2	-0.0	-8951.7

COLUMN DESIGN

Loc ft	Comb	Pu kips	Mux kips-ft	Muz kips-ft	pMn kips-ft	Incl deg	pPn/Pu	pMn/Mu
0.00	1C	3310.9	463.5	9055.3	43655.2	87.07	1.00	4.81**
2.13	1C	3273.0	458.2	8951.7	43735.6	87.07	1.00	4.88**

Column Design : Notes

** Minimum/Maximum requirement for reinforcement ratio or bar spacing violated.

ISOLATED FOOTING DESIGN

ISOLATED FOOTING DESIGN

Code: AASHTO STANDARD (17th Edition 2002) - Ultimate Strength Design
Units: US
Pier View: Upstation.

GEOMETRY

Name : FTG-1
Shape : Rectangular, Type : Spread
Bf(X) = 90.00 ft, Hf(Z) = 10.50 ft, Thickness(Y) = 36.00 in
Footing concentric.
Columns located on the footing:
Column No. 1 at x = 0.00 ft, Rectangular 1074.00 in x 36.00 in
Ag = 945.00 ft^2, Ix = 8682.19 ft^4, Iz = 637875.11 ft^4
Surcharge = 0.00 ksf

DESIGN PARAMETERS

f'c = 4000.00 psi	fy = 60000.00 psi
phi flex = 0.90	phi shear = 0.85
Ec = 3834.3 ksi	Es = 29000.0 ksi

Crack control factor z = 130.00 kips/in
Concrete Type : Normal Weight.

Max Soil Pressures, Service (Without the reduction of overstress allowance)

Corner	X ft	Z ft	comb	Ovs	P kips	Mxx kft	Mzz kft	Soil press. ksf
1	45.00	-5.25	851	1.400	-1932.02	-337.40	-493.90	2.73
			398	1.250	-1404.24	172.38	2497.67	1.66
2	-45.00	-5.25	747	1.400	-1910.60	-358.11	120.88	2.70
			572	1.250	-1425.66	193.08	-2155.80	1.69
3	-45.00	5.25	348	1.250	-1910.60	193.08	120.88	2.60
			815	1.400	-1425.66	-358.11	-2155.80	1.59
4	45.00	5.25	530	1.250	-1932.02	172.38	-493.90	2.63
			719	1.400	-1404.24	-337.40	2497.67	1.56

Max Soil Pressures, Factored

Corner	X ft	Z ft	comb	Ovs	P kips	Mxx kft	Mzz kft	Soil press. ksf
1	45.000	-5.250	1264	---	-2978.51	0.00	0.00	3.74

Corner	X ft	Z ft	comb	Ovs	P kips	Mxx kft	Mzz kft	Soil press. ksf
			2497	—	-1525.45	0.00	1583.36	2.04
2	-45.000	-5.250	1264	—	-2978.51	0.00	0.00	3.74
			2513	—	-1611.12	82.99	-77.09	2.19
3	-45.000	5.250	1264	—	-2978.51	0.00	0.00	3.74
			2078	—	-1782.07	-447.63	-2694.75	1.99
4	45.000	5.250	1264	—	-2978.51	0.00	0.00	3.74
			1982	—	-1755.30	-421.75	3122.09	1.94

Max Soil Pressures, Service (After the reduction of overstress allowance)

Corner	X ft	Z ft	comb	Ovs	P kips	Mxx kft	Mzz kft	Soil press. ksf
1	45.00	-5.25	1	1.000	-1910.60	0.00	0.00	2.47
			1234	1.500	-1271.21	0.00	1319.47	1.13
2	-45.00	-5.25	1	1.000	-1910.60	0.00	0.00	2.47
			1250	1.500	-1342.60	69.15	-64.24	1.22
3	-45.00	5.25	1	1.000	-1910.60	0.00	0.00	2.47
			815	1.400	-1425.66	-358.11	-2155.80	1.14
4	45.00	5.25	1	1.000	-1910.60	0.00	0.00	2.47
			719	1.400	-1404.24	-337.40	2497.67	1.11

Footing Design : Notes

Only max. positive pressure is considered for design.

Max. Soil Pressure Used in Design: (without selfweight and surcharge)

Factored soil pressure	3.20 ksf
Service soil pressure	2.43 ksf
Fatigue soil pressure	0.60 ksf

Reinforcement Schedule

Dir	Quantity	Size	Bar dist. in	As total in^2	Spacing in	Hook
X	10	# 8	4.50	7.90	13.22	Both
X	10	# 8	31.50	7.90	13.22	Both
Z	90	# 6	3.50	39.60	12.06	Both
Z	90	# 6	32.50	39.60	12.06	Both

Flexure

Dir	Loc ft	d in	Mmax kft	Comb	Asb_req in^2	Asb_prv in^2	Asb_eff in^2	Ast_req in^2	Ast_prv in^2	Ast_eff in^2
X	-44.75	31.50	1.0	1264	0.01	7.90	1.25	0.00	7.90	1.25
X	44.75	31.50	1.0	1264	0.01	7.90	1.25	0.00	7.90	1.25
Z	-1.50	32.50	2023.0	1264	18.48	39.60	39.60	0.00	39.60	39.60
Z	1.50	32.50	2023.0	1264	18.48	39.60	39.60	0.00	39.60	39.60

Cracking/Fatigue

Dir	Loc ft	d in	Cracking Mmax kft	Cracking Comb	Cracking fs ksi	Cracking ratio fs	Fatigue Mmax kft	Fatigue Comb	Fatigue fs ksi	Fatigue ratio fs
X	-44.75	31.50	0.8	1229	0.25	0.01	0.2	1	0.06	0.00
X	44.75	31.50	0.8	1229	0.25	0.01	0.2	1	0.06	0.00
Z	-1.50	32.50	1539.8	1229	14.99	0.59	380.4	1	3.70	0.18
Z	1.50	32.50	1539.8	1229	14.99	0.59	380.4	1	3.70	0.18

One Way Shear

Col	Dir	Dist ft	Comb	d in	Vu kips	phi*Vc kips
1	X	-47.38	Outside of Footing	---	---	---
	X	47.38	Outside of Footing	---	---	---
	Z	-4.21	1264	32.50	299.7	3773.9
	Z	4.21	1264	32.50	299.7	3773.9

Two Way Shear

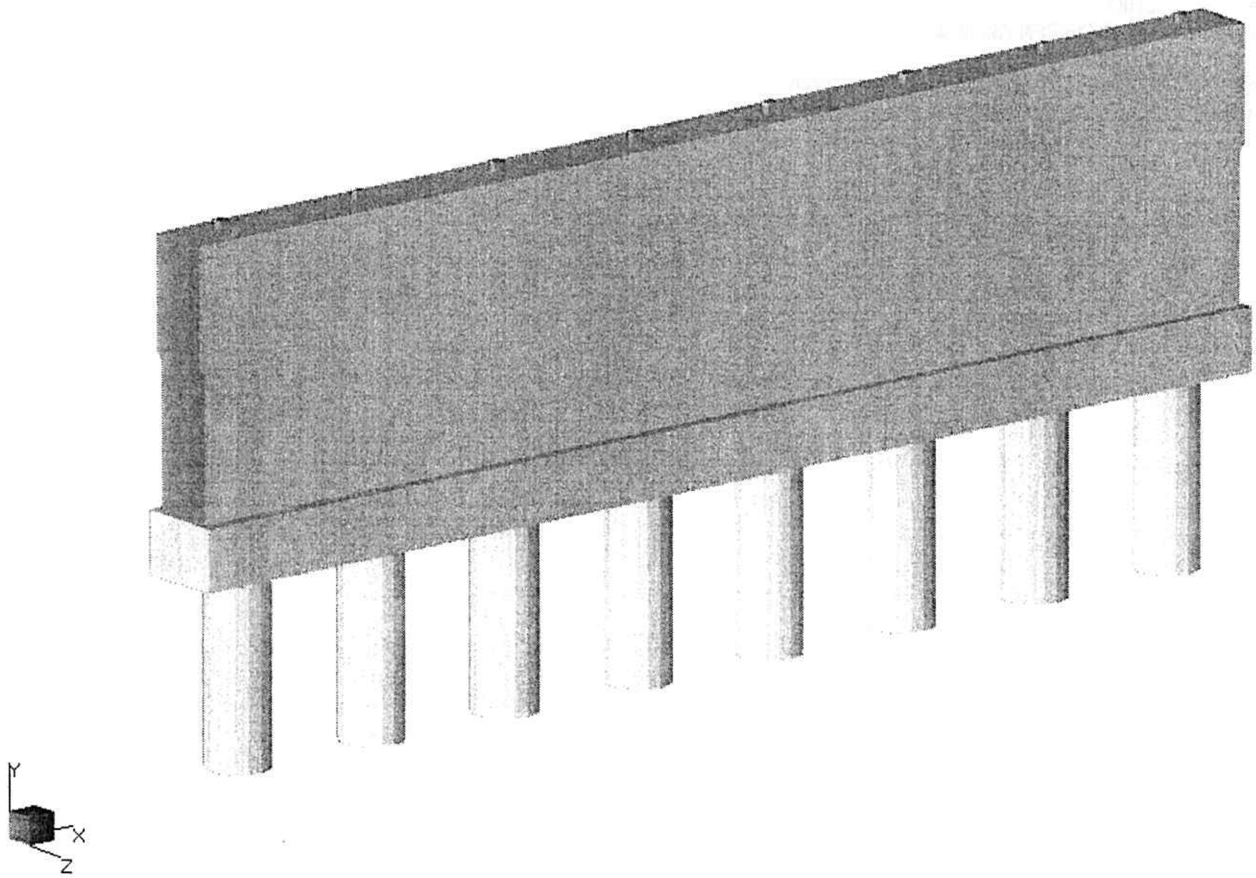
#	Bo ft	Ao ft^2	Comb	Avg. d in	Vu kips	phi*Vc kips
Columns						
1	191.33	510.00	1264	32.00	1390.6	8429.1

Two Way Shear Note

TWO WAY SHEAR IN FOOTING IS NOT DESIGNED AND STIRRUPS ARE NOT CONSIDERED.

PROJECT: SCI-823-TR234 at CSXT RR

FULL IMAGE:



PROJECT DATA

PROJECT DATA

Project:	SCI-823-TR234 at CSXT RR
User Job No.:	5355.02
State:	OHIO
State Job No.:	PID 19415/SFN 7336934
Pier View:	Upstation.
Code:	AASHTO STANDARD (17th Edition 2002)
Comments:	Forward Abutment Pier on Drilled Shafts

PIER GEOMETRY

Pier Info:

Pier View: Upstation.
Pier Type: Hammer Head

Column Shape: Rectangular Non Tapered

Length(X) = 59.00 ft Height max(Y) = 6.00 ft Height min(Y) = 5.50 ft
Bottom length(X) = 58.50 ft Depth(Z) = 3.00 ft Skew angle = 0.00 Reduction of I = 1.000

SUPERSTRUCTURE INFO

Superstructure info:

Total number of spans:	1
Span number rear to current pier:	1
Number of traffic lanes:	4
Barrier height :	42.00 in
Depth of slab :	8.50 in
Curb to curb distance: 0.000	59.000 ft

Beam info:

Height in	Section area in^2	Inertia (Ixx) in^4	Inertia (Iyy) in^4	Beam CG in
72.00	956.00	616018.00	50000.00	34.43

Span #	Span length ft	Bridge Width ft
1	121.000	59.000
		59.000

BEARING POINTS

Number of bearing lines: 1

First bearing line **Eccentricity = 0.00 ft**

Point	Distance ft
1	2.38
2	10.13
3	17.88
4	25.63
5	33.38
6	41.13
7	48.88
8	56.63

MATERIAL PROPERTIES

MATERIAL PROPERTIES

	Cap	Column	Footing
Concrete Type	normal	normal	normal
Concrete Strength (psi)	4000.00	4000.00	4000.00
Concrete Density (lb/ft3)	150.00	150.00	150.00
Concrete Modulus Ec (ksi)	3834.30	3834.30	3834.30
Steel Strength Fy (ksi)	60.00	60.00	60.00

DESIGN PARAMETERS

DESIGN PARAMETERS

AASHTO STANDARD Code

Strength Reduction factors for reinf. concrete

Flexure and tension	0.90
Shear and torsion (normal)	0.85
Shear and torsion (lightweight)	0.85
Axial compression (ties)	0.70
Axial compression (spiral)	0.75

Multi presence factors for live load

1 Lane	1.00
2 Lanes	1.00
3 Lanes	0.90
more than 3 Lanes	0.75

	Crack control factor kip/ft	Clear cover in	Clear side cover in	Impact factors (auto calculation)
Cap	170.00	2.00	3.00	1.27
Column	170.00	2.00		1.27
Footing	130.00	3.00	3.00	1.00

Degree of fixity in foundations for Moment Magnify Method: Ga = 5.00

SEISMIC DESIGN PARAMETERS

Strength Reduction factors for reinf. Concrete Seismic Design

Flexure and tension	0.90
Shear and torsion (normal)	0.85
Shear and torsion (lightweight)	0.85
Axial compression (ties)	0.70
Axial compression (spiral)	0.75

Seismic Overstrength

Flexure and tension	1.30
Axial compression (ties)	1.30
Axial compression (spiral)	1.30

Response Modification Factor 3.00

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: Upstation.

Load Cases: 41

Loadcase ID: D1 Name:

Multiplier = 1.000

Column loads

Col #	Type	Dir	Mag1	y1/L	Mag2	y2/L
1	UDL	X	-3.150 klf	0.00		1.00

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	-149.00
1	2	Y	-149.00
1	3	Y	-149.00
1	4	Y	-149.00
1	5	Y	-149.00
1	6	Y	-149.00
1	7	Y	-149.00
1	8	Y	-149.00

Auto generation details:

Generated Dead Load

Slab weight = 150.00 pcf
Girder weight = 150.00 pcf
Wearing weight = 3360.00 plf
Barrier load = 1060.00 plf

Loadcase ID: W1 Name: Angle: 75

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-3.59 kips	0.50		
UDL	Z		0.23 klf	0.00		1.00

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.85
1	1	Y	-0.64
1	1	Z	1.70
1	2	X	-0.85
1	2	Y	-0.00
1	2	Z	1.70
1	3	X	-0.85
1	3	Y	-0.00
1	3	Z	1.70
1	4	X	-0.85
1	4	Y	-0.00
1	4	Z	1.70
1	5	X	-0.85
1	5	Y	-0.00
1	5	Z	1.70
1	6	X	-0.85
1	6	Y	-0.00
1	6	Z	1.70
1	7	X	-0.85
1	7	Y	-0.00
1	7	Z	1.70
1	8	X	-0.85
1	8	Y	0.64
1	8	Z	1.70

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 75.00 deg Elevation above which wind load acts = 27.00 ft

Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 11.000 psf Cap 40.000 psf
Longitudinal 22.000 psf Column 40.000 psf
Overturning not considered

Loadcase ID: W2 Name: Angle: 60

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-6.31 kips	0.50		
UDL	Z		0.19 klf	0.00		1.00

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-1.31
1	1	Y	-0.99
1	1	Z	1.47
1	2	X	-1.31
1	2	Y	-0.00
1	2	Z	1.47
1	3	X	-1.31
1	3	Y	-0.00
1	3	Z	1.47
1	4	X	-1.31
1	4	Y	-0.00
1	4	Z	1.47
1	5	X	-1.31
1	5	Y	-0.00
1	5	Z	1.47
1	6	X	-1.31
1	6	Y	-0.00
1	6	Z	1.47
1	7	X	-1.31
1	7	Y	-0.00
1	7	Z	1.47
1	8	X	-1.31
1	8	Y	0.99
1	8	Z	1.47

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 60.00 deg Elevation above which wind load acts = 27.00 ft
Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 17.000 psf Cap 40.000 psf
Longitudinal 19.000 psf Column 40.000 psf
Overturning not considered

Loadcase ID: W3 Name: Angle: 45

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-7.44 kips	0.50		
UDL	Z		0.13 klf	0.00		1.00

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-2.55
1	1	Y	-1.92
1	1	Z	1.24
1	2	X	-2.55
1	2	Y	-0.00
1	2	Z	1.24
1	3	X	-2.55
1	3	Y	-0.00
1	3	Z	1.24
1	4	X	-2.55
1	4	Y	-0.00
1	4	Z	1.24
1	5	X	-2.55
1	5	Y	-0.00
1	5	Z	1.24
1	6	X	-2.55
1	6	Y	-0.00
1	6	Z	1.24
1	7	X	-2.55

Line #	Bearing #	Dir	Load kips
1	7	Y	-0.00
1	7	Z	1.24
1	8	X	-2.55
1	8	Y	1.92
1	8	Z	1.24

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 45.00 deg Elevation above which wind load acts = 27.00 ft
Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 33.000 psf Cap 40.000 psf
Longitudinal 16.000 psf Column 40.000 psf
Overturning not considered

Loadcase ID: W4 Name: Angle: 30

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-6.67 kips	0.50		
UDL	Z		0.07 klf	0.00		1.00

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-3.17
1	1	Y	-2.38
1	1	Z	0.93
1	2	X	-3.17
1	2	Y	-0.00
1	2	Z	0.93
1	3	X	-3.17
1	3	Y	-0.00
1	3	Z	0.93

Line #	Bearing #	Dir	Load kips
1	4	X	-3.17
1	4	Y	-0.00
1	4	Z	0.93
1	5	X	-3.17
1	5	Y	-0.00
1	5	Z	0.93
1	6	X	-3.17
1	6	Y	-0.00
1	6	Z	0.93
1	7	X	-3.17
1	7	Y	-0.00
1	7	Z	0.93
1	8	X	-3.17
1	8	Y	2.38
1	8	Z	0.93

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 30.00 deg Elevation above which wind load acts = 27.00 ft
Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 41.000 psf Cap 40.000 psf
Longitudinal 12.000 psf Column 40.000 psf
Overturning not considered

Loadcase ID: W5 Name: Angle: 15

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-4.21 kips	0.50		
UDL	Z		0.02 klf	0.00		1.00

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-3.40
1	1	Y	-2.56
1	1	Z	0.46
1	2	X	-3.40
1	2	Y	-0.00
1	2	Z	0.46
1	3	X	-3.40
1	3	Y	-0.00
1	3	Z	0.46
1	4	X	-3.40
1	4	Y	-0.00
1	4	Z	0.46
1	5	X	-3.40
1	5	Y	-0.00
1	5	Z	0.46
1	6	X	-3.40
1	6	Y	-0.00
1	6	Z	0.46
1	7	X	-3.40
1	7	Y	-0.00
1	7	Z	0.46
1	8	X	-3.40
1	8	Y	2.56
1	8	Z	0.46

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 15.00 deg Elevation above which wind load acts = 27.00 ft
Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 44.000 psf Cap 40.000 psf
Longitudinal 6.000 psf Column 40.000 psf
Overturning not considered

Loadcase ID: W6 Name: Angle: 0

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-0.72 kips	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-3.86
1	1	Y	-13.39
1	1	Z	0.00
1	2	X	-3.86
1	2	Y	8.92
1	2	Z	0.00
1	3	X	-3.86
1	3	Y	8.92
1	3	Z	0.00
1	4	X	-3.86
1	4	Y	8.92
1	4	Z	0.00
1	5	X	-3.86
1	5	Y	8.92
1	5	Z	0.00
1	6	X	-3.86
1	6	Y	8.92
1	6	Z	0.00
1	7	X	-3.86
1	7	Y	8.92
1	7	Z	0.00
1	8	X	-3.86
1	8	Y	31.24
1	8	Z	0.00

Auto generation details:

Generated Wind Load on Structure

Angle of wind = 0.00 deg Elevation above which wind load acts = 27.00 ft
Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 50.000 psf Cap 40.000 psf
Longitudinal 0.000 psf Column 40.000 psf
Overturning 20.000 psf

Loadcase ID: W7 Name: Angle: -15

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-4.21 kips	0.50		
UDL	Z		-0.02 klf	0.00		1.00

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-3.40
1	1	Y	-2.56
1	1	Z	-0.46
1	2	X	-3.40
1	2	Y	-0.00
1	2	Z	-0.46
1	3	X	-3.40
1	3	Y	-0.00
1	3	Z	-0.46
1	4	X	-3.40
1	4	Y	-0.00
1	4	Z	-0.46
1	5	X	-3.40
1	5	Y	-0.00
1	5	Z	-0.46
1	6	X	-3.40
1	6	Y	-0.00
1	6	Z	-0.46
1	7	X	-3.40
1	7	Y	-0.00
1	7	Z	-0.46
1	8	X	-3.40
1	8	Y	2.56
1	8	Z	-0.46

Auto generation details:

Generated Wind Load on Structure

Angle of wind = -15.00 deg Elevation above which wind load acts = 27.00 ft
Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 44.000 psf Cap 40.000 psf
Longitudinal 6.000 psf Column 40.000 psf
Overturning not considered

Loadcase ID: W8 Name: Angle: -30

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-6.67 kips	0.50		
UDL	Z		-0.07 klf	0.00		1.00

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-3.17
1	1	Y	-2.38
1	1	Z	-0.93
1	2	X	-3.17
1	2	Y	-0.00
1	2	Z	-0.93
1	3	X	-3.17
1	3	Y	-0.00
1	3	Z	-0.93
1	4	X	-3.17
1	4	Y	-0.00
1	4	Z	-0.93
1	5	X	-3.17
1	5	Y	-0.00
1	5	Z	-0.93
1	6	X	-3.17
1	6	Y	-0.00
1	6	Z	-0.93
1	7	X	-3.17
1	7	Y	-0.00
1	7	Z	-0.93
1	8	X	-3.17
1	8	Y	2.38
1	8	Z	-0.93

Auto generation details:

Generated Wind Load on Structure

Angle of wind = -30.00 deg Elevation above which wind load acts = 27.00 ft

Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 41.000 psf Cap 40.000 psf
Longitudinal 12.000 psf Column 40.000 psf
Overturning not considered

Loadcase ID: W9 Name: Angle: -45

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-7.44 kips	0.50		
UDL	Z		-0.13 klf	0.00		1.00

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-2.55
1	1	Y	-1.92
1	1	Z	-1.24
1	2	X	-2.55
1	2	Y	-0.00
1	2	Z	-1.24
1	3	X	-2.55
1	3	Y	-0.00
1	3	Z	-1.24
1	4	X	-2.55
1	4	Y	-0.00
1	4	Z	-1.24
1	5	X	-2.55
1	5	Y	-0.00
1	5	Z	-1.24
1	6	X	-2.55
1	6	Y	-0.00
1	6	Z	-1.24
1	7	X	-2.55
1	7	Y	-0.00
1	7	Z	-1.24
1	8	X	-2.55
1	8	Y	1.92
1	8	Z	-1.24

Auto generation details:

Generated Wind Load on Structure

Angle of wind = -45.00 deg Elevation above which wind load acts = 27.00 ft
Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 33.000 psf Cap 40.000 psf
Longitudinal 16.000 psf Column 40.000 psf
Overturning not considered

Loadcase ID: W10 Name: Angle: -60

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-6.31 kips	0.50		
UDL	Z		-0.19 klf	0.00		1.00

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-1.31
1	1	Y	-0.99
1	1	Z	-1.47
1	2	X	-1.31
1	2	Y	-0.00
1	2	Z	-1.47
1	3	X	-1.31
1	3	Y	-0.00
1	3	Z	-1.47
1	4	X	-1.31
1	4	Y	-0.00
1	4	Z	-1.47
1	5	X	-1.31
1	5	Y	-0.00
1	5	Z	-1.47
1	6	X	-1.31
1	6	Y	-0.00
1	6	Z	-1.47
1	7	X	-1.31

Line #	Bearing #	Dir	Load kips
1	7	Y	-0.00
1	7	Z	-1.47
1	8	X	-1.31
1	8	Y	0.99
1	8	Z	-1.47

Auto generation details:

Generated Wind Load on Structure

Angle of wind = -60.00 deg Elevation above which wind load acts = 27.00 ft
Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 17.000 psf Cap 40.000 psf
Longitudinal 19.000 psf Column 40.000 psf
Overturning not considered

Loadcase ID: W11 Name: Angle: -75

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Force	X	0.00	-3.59 kips	0.50		
UDL	Z		-0.23 klf	0.00		1.00

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.85
1	1	Y	-0.64
1	1	Z	-1.70
1	2	X	-0.85
1	2	Y	-0.00
1	2	Z	-1.70
1	3	X	-0.85
1	3	Y	-0.00
1	3	Z	-1.70

Line #	Bearing #	Dir	Load kips
1	4	X	-0.85
1	4	Y	-0.00
1	4	Z	-1.70
1	5	X	-0.85
1	5	Y	-0.00
1	5	Z	-1.70
1	6	X	-0.85
1	6	Y	-0.00
1	6	Z	-1.70
1	7	X	-0.85
1	7	Y	-0.00
1	7	Z	-1.70
1	8	X	-0.85
1	8	Y	0.64
1	8	Z	-1.70

Auto generation details:

Generated Wind Load on Structure

Angle of wind = -75.00 deg Elevation above which wind load acts = 27.00 ft
Default wind pressure:

Wind pressure for superstructure: Wind pressure for substructure:
Transverse 11.000 psf Cap 40.000 psf
Longitudinal 22.000 psf Column 40.000 psf
Overturning not considered

Loadcase ID: WL1 Name: Angle: 75

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.11
1	1	Y	-0.20
1	1	Z	0.32
1	2	X	-0.11
1	2	Y	-0.00
1	2	Z	0.32

Line #	Bearing #	Dir	Load kips
1	3	X	-0.11
1	3	Y	-0.00
1	3	Z	0.32
1	4	X	-0.11
1	4	Y	-0.00
1	4	Z	0.32
1	5	X	-0.11
1	5	Y	-0.00
1	5	Z	0.32
1	6	X	-0.11
1	6	Y	-0.00
1	6	Z	0.32
1	7	X	-0.11
1	7	Y	-0.00
1	7	Z	0.32
1	8	X	-0.11
1	8	Y	0.20
1	8	Z	0.32

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = 75.00 deg Live load length = 60.50 ft

Loadcase ID: WL2 Name: Angle: 60

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.26
1	1	Y	-0.48
1	1	Z	0.29
1	2	X	-0.26
1	2	Y	-0.00
1	2	Z	0.29
1	3	X	-0.26
1	3	Y	-0.00
1	3	Z	0.29

Line #	Bearing #	Dir	Load kips
1	4	X	-0.26
1	4	Y	-0.00
1	4	Z	0.29
1	5	X	-0.26
1	5	Y	-0.00
1	5	Z	0.29
1	6	X	-0.26
1	6	Y	-0.00
1	6	Z	0.29
1	7	X	-0.26
1	7	Y	-0.00
1	7	Z	0.29
1	8	X	-0.26
1	8	Y	0.48
1	8	Z	0.29

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = 60.00 deg Live load length = 60.50 ft

Loadcase ID: WL3 Name: Angle: 45

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.50
1	1	Y	-0.94
1	1	Z	0.24
1	2	X	-0.50
1	2	Y	-0.00
1	2	Z	0.24
1	3	X	-0.50
1	3	Y	-0.00
1	3	Z	0.24
1	4	X	-0.50
1	4	Y	-0.00
1	4	Z	0.24

Line #	Bearing #	Dir	Load kips
1	5	X	-0.50
1	5	Y	-0.00
1	5	Z	0.24
1	6	X	-0.50
1	6	Y	-0.00
1	6	Z	0.24
1	7	X	-0.50
1	7	Y	-0.00
1	7	Z	0.24
1	8	X	-0.50
1	8	Y	0.94
1	8	Z	0.24

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = 45.00 deg Live load length = 60.50 ft

Loadcase ID: WL4   Name: Angle: 30

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.62
1	1	Y	-1.16
1	1	Z	0.18
1	2	X	-0.62
1	2	Y	-0.00
1	2	Z	0.18
1	3	X	-0.62
1	3	Y	-0.00
1	3	Z	0.18
1	4	X	-0.62
1	4	Y	-0.00
1	4	Z	0.18
1	5	X	-0.62
1	5	Y	-0.00
1	5	Z	0.18

Line #	Bearing #	Dir	Load kips
1	6	X	-0.62
1	6	Y	-0.00
1	6	Z	0.18
1	7	X	-0.62
1	7	Y	-0.00
1	7	Z	0.18
1	8	X	-0.62
1	8	Y	1.16
1	8	Z	0.18

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = 30.00 deg Live load length = 60.50 ft

Loadcase ID: WL5 Name: Angle: 15

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.67
1	1	Y	-1.25
1	1	Z	0.09
1	2	X	-0.67
1	2	Y	-0.00
1	2	Z	0.09
1	3	X	-0.67
1	3	Y	-0.00
1	3	Z	0.09
1	4	X	-0.67
1	4	Y	-0.00
1	4	Z	0.09
1	5	X	-0.67
1	5	Y	-0.00
1	5	Z	0.09
1	6	X	-0.67
1	6	Y	-0.00
1	6	Z	0.09

Line #	Bearing #	Dir	Load kips
1	7	X	-0.67
1	7	Y	-0.00
1	7	Z	0.09
1	8	X	-0.67
1	8	Y	1.25
1	8	Z	0.09

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = 15.00 deg Live load length = 60.50 ft

Loadcase ID: WL6 Name: Angle: 0

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.76
1	1	Y	-1.42
1	1	Z	0.00
1	2	X	-0.76
1	2	Y	-0.00
1	2	Z	0.00
1	3	X	-0.76
1	3	Y	-0.00
1	3	Z	0.00
1	4	X	-0.76
1	4	Y	-0.00
1	4	Z	0.00
1	5	X	-0.76
1	5	Y	-0.00
1	5	Z	0.00
1	6	X	-0.76
1	6	Y	-0.00
1	6	Z	0.00
1	7	X	-0.76
1	7	Y	-0.00
1	7	Z	0.00

Line #	Bearing #	Dir	Load kips
1	8	X	-0.76
1	8	Y	1.42
1	8	Z	0.00

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = 0.00 deg Live load length = 60.50 ft

Loadcase ID: WL7 Name: Angle: -15

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.67
1	1	Y	-1.25
1	1	Z	-0.09
1	2	X	-0.67
1	2	Y	-0.00
1	2	Z	-0.09
1	3	X	-0.67
1	3	Y	-0.00
1	3	Z	-0.09
1	4	X	-0.67
1	4	Y	-0.00
1	4	Z	-0.09
1	5	X	-0.67
1	5	Y	-0.00
1	5	Z	-0.09
1	6	X	-0.67
1	6	Y	-0.00
1	6	Z	-0.09
1	7	X	-0.67
1	7	Y	-0.00
1	7	Z	-0.09
1	8	X	-0.67
1	8	Y	1.25
1	8	Z	-0.09

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = -15.00 deg Live load length = 60.50 ft

Loadcase ID: WL8 Name: Angle: -30

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.62
1	1	Y	-1.16
1	1	Z	-0.18
1	2	X	-0.62
1	2	Y	-0.00
1	2	Z	-0.18
1	3	X	-0.62
1	3	Y	-0.00
1	3	Z	-0.18
1	4	X	-0.62
1	4	Y	-0.00
1	4	Z	-0.18
1	5	X	-0.62
1	5	Y	-0.00
1	5	Z	-0.18
1	6	X	-0.62
1	6	Y	-0.00
1	6	Z	-0.18
1	7	X	-0.62
1	7	Y	-0.00
1	7	Z	-0.18
1	8	X	-0.62
1	8	Y	1.16
1	8	Z	-0.18

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = -30.00 deg Live load length = 60.50 ft

Loadcase ID: WL9 Name: Angle: -45

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.50
1	1	Y	-0.94
1	1	Z	-0.24
1	2	X	-0.50
1	2	Y	-0.00
1	2	Z	-0.24
1	3	X	-0.50
1	3	Y	-0.00
1	3	Z	-0.24
1	4	X	-0.50
1	4	Y	-0.00
1	4	Z	-0.24
1	5	X	-0.50
1	5	Y	-0.00
1	5	Z	-0.24
1	6	X	-0.50
1	6	Y	-0.00
1	6	Z	-0.24
1	7	X	-0.50
1	7	Y	-0.00
1	7	Z	-0.24
1	8	X	-0.50
1	8	Y	0.94
1	8	Z	-0.24

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = -45.00 deg Live load length = 60.50 ft

Loadcase ID: WL10 Name: Angle: -60

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.26
1	1	Y	-0.48
1	1	Z	-0.29
1	2	X	-0.26
1	2	Y	-0.00
1	2	Z	-0.29
1	3	X	-0.26
1	3	Y	-0.00
1	3	Z	-0.29
1	4	X	-0.26
1	4	Y	-0.00
1	4	Z	-0.29
1	5	X	-0.26
1	5	Y	-0.00
1	5	Z	-0.29
1	6	X	-0.26
1	6	Y	-0.00
1	6	Z	-0.29
1	7	X	-0.26
1	7	Y	-0.00
1	7	Z	-0.29
1	8	X	-0.26
1	8	Y	0.48
1	8	Z	-0.29

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = -60.00 deg Live load length = 60.50 ft

Loadcase ID: WL11 Name: Angle: -75

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	-0.11
1	1	Y	-0.20
1	1	Z	-0.32
1	2	X	-0.11
1	2	Y	-0.00
1	2	Z	-0.32
1	3	X	-0.11
1	3	Y	-0.00
1	3	Z	-0.32
1	4	X	-0.11
1	4	Y	-0.00
1	4	Z	-0.32
1	5	X	-0.11
1	5	Y	-0.00
1	5	Z	-0.32
1	6	X	-0.11
1	6	Y	-0.00
1	6	Z	-0.32
1	7	X	-0.11
1	7	Y	-0.00
1	7	Z	-0.32
1	8	X	-0.11
1	8	Y	0.20
1	8	Z	-0.32

Auto generation details:

Generated Wind Load on Live Load

Angle of wind = -75.00 deg Live load length = 60.50 ft

Loadcase ID: T1 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	X	0.00
1	1	Z	-5.82
1	2	X	0.00
1	2	Z	-5.82
1	3	X	0.00
1	3	Z	-5.82
1	4	X	0.00
1	4	Z	-5.82
1	5	X	0.00
1	5	Z	-5.82
1	6	X	0.00
1	6	Z	-5.82
1	7	X	0.00
1	7	Z	-5.82
1	8	X	0.00
1	8	Z	-5.82

Auto generation details:

Bearing type

Elastomeric Bearings
Direction of thermal force: +(Z)
Length of Superstructure Contributing, L: 82.000 ft
Change in temperature: 80.000 °F
Coefficient of thermal expansion: 6.0e-006 ft/°F
Area of bearing: 384.00 in^2
Shear modulus of Elastomer: 0.13 kips
Total Elastomer Thickness: 4.00 in

Loadcase ID: E1 Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
UDL	X		3.54 klf	0.00		1.00

Column loads

Col #	Type	Dir	Mag1	y1/L	Mag2	y2/L

Col #	Type	Dir	Mag1	y1/L	Mag2	y2/L
1	Pres.	X	0.060 ksf	0.00		1.00

Loadcase ID: (L+In)1 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	-71.00
1	2	Y	-71.00
1	3	Y	-71.00
1	4	Y	-71.00
1	5	Y	-71.00
1	6	Y	-71.00
1	7	Y	-71.00
1	8	Y	-71.00

Auto generation details:

Generated Live Load

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:
HS25 truck
H25/HS25 Lane Load
Military

Transverse Positioning

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Not Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 42
Total number of Possible Combination = 6263

Loadcase ID: (L+In)2 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	-50.91
1	2	Y	-58.94
1	3	Y	-50.91
1	4	Y	-5.36
1	5	Y	0.00
1	6	Y	0.00
1	7	Y	0.00
1	8	Y	0.00

Auto generation details:

Generated Live Load

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:
HS25 truck
H25/HS25 Lane Load
Military

Transverse Positioning

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Not Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 42
Total number of Possible Combination = 6263

Loadcase ID: (L+In)3 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	0.00
1	2	Y	0.00
1	3	Y	0.00
1	4	Y	0.00
1	5	Y	-5.36
1	6	Y	-50.91
1	7	Y	-54.93
1	8	Y	-54.93

Auto generation details:

Generated Live Load

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:
HS25 truck
H25/HS25 Lane Load
Military

Transverse Positioning

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Not Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 42
Total number of Possible Combination = 6263

Loadcase ID: (L+In)4 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	0.00
1	2	Y	0.00
1	3	Y	0.00
1	4	Y	0.00
1	5	Y	0.00
1	6	Y	0.00
1	7	Y	-28.13
1	8	Y	-54.93

Auto generation details:

Generated Live Load

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:
HS25 truck
H25/HS25 Lane Load
Military

Transverse Positioning

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Not Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 42
Total number of Possible Combination = 6263

Loadcase ID: (L+In)5 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	0.00
1	2	Y	0.00
1	3	Y	0.00
1	4	Y	-18.09
1	5	Y	-50.64
1	6	Y	-56.67
1	7	Y	-49.43
1	8	Y	-49.43

Auto generation details:

Generated Live Load

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:
HS25 truck
H25/HS25 Lane Load
Military

Transverse Positioning

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Not Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 42
Total number of Possible Combination = 6263

Loadcase ID: (L+In)6 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	-45.82
1	2	Y	-53.05
1	3	Y	-56.67
1	4	Y	-50.64
1	5	Y	-18.09
1	6	Y	0.00
1	7	Y	0.00
1	8	Y	0.00

Auto generation details:

Generated Live Load

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:
HS25 truck
H25/HS25 Lane Load
Military

Transverse Positioning

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Not Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 42
Total number of Possible Combination = 6263

Loadcase ID: (L+In)7 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir.	Load kips
1	1	Y	-50.91
1	2	Y	-32.15
1	3	Y	0.00
1	4	Y	0.00
1	5	Y	0.00
1	6	Y	0.00
1	7	Y	0.00
1	8	Y	0.00

Auto generation details:

Generated Live Load

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:
HS25 truck
H25/HS25 Lane Load
Military

Transverse Positioning

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Not Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 42
Total number of Possible Combination = 6263

Loadcase ID: (L+In)8 Name:

Multiplier = 1.000

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Y	-28.13
1	2	Y	-50.91
1	3	Y	-4.02
1	4	Y	0.00
1	5	Y	0.00
1	6	Y	0.00
1	7	Y	0.00
1	8	Y	0.00

Auto generation details:

Generated Live Load

Longitudinal Reaction: Simple Span Distribution

Selected Vehicles:
HS25 truck
H25/HS25 Lane Load
Military

Transverse Positioning

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Not Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 42
Total number of Possible Combination = 6263

Loadcase ID: LF1 Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Moment	X		-135.15 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Z	-1.33
1	2	Z	-1.33
1	3	Z	-1.33
1	4	Z	-1.33
1	5	Z	-1.33
1	6	Z	-1.33
1	7	Z	-1.33
1	8	Z	-1.33

Auto generation details:

Selected Live Load for LF generation

Load: H25/HS25 Lane Load
Number of loaded lanes = 4
Contributing longitudinal length = 60.50 ft

Loadcase ID: LF2 Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Moment	X		-135.15 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Z	-1.33
1	2	Z	-1.33
1	3	Z	-1.33
1	4	Z	-1.33
1	5	Z	-1.33
1	6	Z	-1.33
1	7	Z	-1.33
1	8	Z	-1.33

Auto generation details:

Selected Live Load for LF generation

Number of loaded lanes = all combinations
Contributing longitudinal length = 0.00 ft

Loadcase ID: LF3 Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Moment	X		-135.15 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Z	-1.33
1	2	Z	-1.33
1	3	Z	-1.33
1	4	Z	-1.33
1	5	Z	-1.33
1	6	Z	-1.33
1	7	Z	-1.33
1	8	Z	-1.33

Auto generation details:

Selected Live Load for LF generation

Number of loaded lanes = all combinations
Contributing longitudinal length = 0.00 ft

Loadcase ID: LF4 Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Moment	X		-135.15 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Z	-1.33
1	2	Z	-1.33
1	3	Z	-1.33
1	4	Z	-1.33
1	5	Z	-1.33
1	6	Z	-1.33
1	7	Z	-1.33
1	8	Z	-1.33

Auto generation details:

Selected Live Load for LF generation

Number of loaded lanes = all combinations
Contributing longitudinal length = 0.00 ft

Loadcase ID: LF5 Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Moment	X		-135.15 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Z	-1.33
1	2	Z	-1.33
1	3	Z	-1.33
1	4	Z	-1.33
1	5	Z	-1.33
1	6	Z	-1.33
1	7	Z	-1.33
1	8	Z	-1.33

Auto generation details:

Selected Live Load for LF generation

Number of loaded lanes = all combinations
Contributing longitudinal length = 0.00 ft

Loadcase ID: LF6   Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Moment	X		-135.15 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Z	-1.33
1	2	Z	-1.33
1	3	Z	-1.33
1	4	Z	-1.33
1	5	Z	-1.33
1	6	Z	-1.33
1	7	Z	-1.33
1	8	Z	-1.33

Auto generation details:

Selected Live Load for LF generation

Number of loaded lanes = all combinations
Contributing longitudinal length = 0.00 ft

Loadcase ID: LF7   Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Moment	X		-135.15 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Z	-1.33
1	2	Z	-1.33
1	3	Z	-1.33
1	4	Z	-1.33
1	5	Z	-1.33
1	6	Z	-1.33
1	7	Z	-1.33
1	8	Z	-1.33

Auto generation details:

Selected Live Load for LF generation

Number of loaded lanes = all combinations
Contributing longitudinal length = 0.00 ft

Loadcase ID: LF8 Name:

Multiplier = 1.000

Cap loads

Type	Dir	Arm ft	Mag1	x1/L	Mag2	x2/L
Moment	X		-135.15 k-ft	0.50		

Bearing loads

Line #	Bearing #	Dir	Load kips
1	1	Z	-1.33
1	2	Z	-1.33
1	3	Z	-1.33
1	4	Z	-1.33
1	5	Z	-1.33
1	6	Z	-1.33
1	7	Z	-1.33
1	8	Z	-1.33

Auto generation details:

Selected Live Load for LF generation

Number of loaded lanes = all combinations
Contributing longitudinal length = 0.00 ft

Selected load groups

SERVICE GROUP I
SERVICE GROUP II
SERVICE GROUP III
SERVICE GROUP IV
SERVICE GROUP V
SERVICE GROUP VI
SERVICE GROUP VII
SERVICE GROUP VIII
SERVICE GROUP IX
SERVICE GROUP X
LOAD FACTOR GROUP I
LOAD FACTOR GROUP II
LOAD FACTOR GROUP III
LOAD FACTOR GROUP IV
LOAD FACTOR GROUP V
LOAD FACTOR GROUP VI
LOAD FACTOR GROUP VII
LOAD FACTOR GROUP VIII
LOAD FACTOR GROUP IX
LOAD FACTOR GROUP X

CAP DESIGN

CAP DESIGN

Code: AASHTO STANDARD (17th Edition 2002) - Ultimate Strength Design
Units: US
Pier View: Upstation.

DESIGN PARAMETERS

f'c = 4000.0 psi
Fy flex = 60000.0 psi — Fy shear = 60000.0 psi
phi flex = 0.90 — phi shear = 0.85
Ec = 3834.3 ksi — Es = 29000.0 ksi
crack control factor z = 170.00 kips / in
Concrete Type : Normal Weight.
Design of cap at face of column.

CAP GEOMETRY

Hammer Head Cap : Length(X) = 59.00 ft Depth(Z) = 36.00 in

MAIN REINFORCEMENT

	Bar size	Quantity	Bar dist. in	As total in^2	From ft	To ft	Hook
TOP							
	# 6	2	4.00	0.880	0.50	59.00	None
	# 6	2	16.00	0.880	0.50	59.00	None
	# 6	2	28.00	0.880	0.50	59.00	None
BOTTOM							
	# 6	2	4.00	0.880	0.50	59.00	None
	# 6	2	16.00	0.880	0.50	59.00	None
	# 6	2	28.00	0.880	0.50	59.00	None

STIRRUPS

From ft	To ft	Stirrup Size	n legs	Spacing in	Aprv/s in^2 / ft
0.50	78.50	# 8	2	12.00	1.58

Clear Cover on Sides = 2.00 in

FLEXURE DESIGN

Span 1: From 0.00 ft To 29.50 ft

Loc ft	AbsLoc ft	H in	Mmax Mmin kips-ft	pMn kips-ft	Comb	Asb-req in^2	Asb-prv in^2	Asb-eff in^2	Ast-req in^2	Ast-prv in^2	Ast-eff in^2
0.3	0.3	72	0.0	0.0	0	0.00	0.00	0.00	0.00	0.00	0.00
			-0.1	-0.1	809	0.00	0.00	0.00	0.00	0.00*	0.00*

Span 2: From 29.50 ft To 59.00 ft

Loc ft	AbsLoc ft	H in	Mmax Mmin kips-ft	pMn kips-ft	Comb	Asb-req in^2	Asb-prv in^2	Asb-eff in^2	Ast-req in^2	Ast-prv in^2	Ast-eff in^2
29.3	58.8	72	0.0	385.0	0	0.00	2.64	0.44	0.00	2.64	0.56
			-0.1	-358.7	804	0.00	2.64	0.56	0.00	2.64	0.31

Flexure Design : Notes

* The provided reinforcement is not adequate, either less than required or larger than maximum allowed.

SHEAR AND TORSION DESIGN

Span 1: From 0.00 ft To 29.50 ft

Loc ft	AbsLoc ft	Pos	Vu kips	Comb	Tu kips-ft	Comb	phi*Vc kips	T-lim kips-ft	Avs/s in^2/ft	2Ats/s in^2/ft	Av/s in^2/ft	Aprv/s in^2/ft	Alt in^2
0.25	0.25	L	0.8	809	0.0	0	278.7	139.3	0.00	0.00	0.00	0.00	0.00

Span 2: From 29.50 ft To 59.00 ft

Loc ft	AbsLoc ft	Pos	Vu kips	Comb	Tu kips-ft	Comb	phi*Vc kips	T-lim kips-ft	Avs/s in^2/ft	2Ats/s in^2/ft	Av/s in^2/ft	Aprv/s in^2/ft	Alt in^2
29.25	58.75	R	0.8	806	0.0	0	216.8	139.3	0.00	0.00	0.00	1.58	0.00

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.
- 2Ats/s is the required area of steel per unit length for two legs 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 steel per unit length due to shear (stirrups).
- Alt is the total longitudinal steel required due to torsion in addition to the REQUIRED flexural steel.

CRACKING/FATIGUE CHECK

Span 1: From 0.00 ft To 29.50 ft

Loc ft	AbsLoc ft	H in	Cracking fs-t fs-b ksi	Cracking ratio fs-t ratio fs-b	Cracking Comb	Fatigue fs-t fs-b ksi	Fatigue ratio fs-t ratio fs-b
0.25	0.3	72.0	0.0	-0.00	0	0.0	0.00
			0.0	-0.00	0	0.0	0.00

Span 2: From 29.50 ft To 59.00 ft

Loc ft	AbsLoc ft	H in	Cracking fs-t fs-b ksi	Cracking ratio fs-t ratio fs-b	Cracking Comb	Fatigue fs-t fs-b ksi	Fatigue ratio fs-t ratio fs-b
29.25	58.8	72.0	0.0	0.00	159	0.0	0.00
			0.0	0.00	0	0.0	0.00

* Cracking / fatigue checking failed.

COLUMN DESIGN

COLUMN DESIGN - Column: 1

Code: AASHTO STANDARD (17th Edition 2002) - Factored Load Design
Units: US
Pier View: Upstation.
Design/Analysis Method: No Slenderness Considered.

Column Type: Rectangular 702.00 x 36.00 in

DESIGN PARAMETERS

f'c = 4000.0 psi	fy = 60000.0 psi
phi flex = 0.90	phi axial = 0.70
Ec = 3834.3 ksi	Es = 29000 ksi
Concrete Type : Normal Weight.	

Reinforcement

Rebar Pattern: Rectangular
Rebar Orientation: Face Parallel

Reinforcement Schedule

Layer	Dir	Size	No. bars	Bar Dist. in
1	X	8	58	4.00
1	Z	8	4	4.00

Reinforcement summary

Main bars summary:
120 # 8 bars
Total number of bars in the column: 120
Ties size: # 3

Design values used - (e-min effect included).

Loc ft	Comb	Fx kips	Fy kips	Fz kips	Mx kips-ft	My kips-ft	Mz kips-ft
0.00	9C	-314.1	2614.3	0.0	-366.0	0.0	11917.6
10.25	9C	353.0	2374.7	0.0	-332.5	-0.0	-8498.7
0.00	1006C	-317.1	1703.2	84.8	1257.9	0.0	8451.3
10.25	1006C	344.3	1530.4	-84.8	-388.4	-0.0	-5061.5

COLUMN DESIGN

Loc ft	Comb	Pu kips	Mux kips-ft	Muz kips-ft	pMn kips-ft	Incl deg	pPn/Pu	pMn/Mu
0.00	1006C	1703.2	1257.9	8451.3	41253.6	81.53	1.00	4.83**
10.25	9C	2374.7	332.5	8498.7	100128.9	87.76	1.00	11.77**

Column Design : Notes

** Minimum/Maximum requirement for reinforcement ratio or bar spacing violated.

ISOLATED FOOTING DESIGN

ISOLATED FOOTING DESIGN

Code: AASHTO STANDARD (17th Edition 2002) - Ultimate Strength Design
Units: US
Pier View: Upstation.

GEOMETRY

Name : Pier-Col
Shape : Rectangular, Type : Pile/Shaft Cap
Bf(X) = 59.00 ft, Hf(Z) = 4.00 ft, Thickness(Y) = 36.00 in
Footing concentric.
Columns located on the footing:
Column No. 1 at x = 0.00 ft, Rectangular 702.00 in x 36.00 in
Ag = 236.00 ft^2, Ix = 475.41 ft^2, Iz = 2362.50 ft^2
Surcharge = 0.00 ksf
Piles: Circular Size: 36.00 in Capacity: 600.00 kips

DESIGN PARAMETERS

f'c = 4000.00 psi	fy = 60000.00 psi
phi flex = 0.90	phi shear = 0.85
Ec = 3834.3 ksi	Es = 29000.0 ksi

Crack control factor z = 130.00 kips/in
Concrete Type : Normal Weight.

Pile Reactions, Service (Without the reduction of overstress allowance)

Pile	Loc(X) ft	X In	Z in	comb	Ovs	P kips	Mxx kft	Mzz kft	Pile Reac. kips
1	-26.25	39.00	0.0	71	1.250	-2082.10	-273.41	-1343.43	258.61 #
				161	1.250	-1723.05	-273.41	-5792.75	164.29 #
2	-18.75	129.00	0.0	71	1.250	-2082.10	-273.41	-1343.43	262.88 #
				162	1.250	-1639.99	-273.41	-4661.08	181.28 #
3	-11.25	219.00	0.0	63	1.250	-2103.52	-246.05	-1687.39	268.18 #
				24	1.250	-1535.52	-59.74	-2519.93	193.22 #
4	-3.75	309.00	0.0	159	1.250	-2124.94	-273.41	-2626.17	274.72 #
				14	1.250	-1464.13	0.00	-365.35	195.71
5	3.75	399.00	0.0	159	1.250	-2124.94	-273.41	-2626.17	283.06 #
				14	1.250	-1464.13	0.00	-365.35	196.87

Pile	Loc(X) ft	X in	Z in	comb	Ovs	P kips	Mxx kft	Mzz kft	Pile Reac. kips
6	11.25	489.00	0.0	159	1.250	-2124.94	-273.41	-2626.17	291.40 #
				14	1.250	-1464.13	0.00	-365.35	198.03
7	18.75	579.00	0.0	159	1.250	-2124.94	-273.41	-2626.17	299.73 #
				14	1.250	-1464.13	0.00	-365.35	199.19
8	26.25	669.00	0.0	159	1.250	-2124.94	-273.41	-2626.17	308.07 #
				14	1.250	-1464.13	0.00	-365.35	200.35

Pile Reactions, Factored

Pile	Loc(X) ft	X in	Z in	comb	Ovs	P kips	Mxx kft	Mzz kft	Pile Reac. kips
1	-26.25	39.0	0.0	804	—	-3229.30	0.00	-3418.84	382.93
				1346	—	-2153.81	-1098.91	-8047.29	196.41 #
2	-18.75	129.0	0.0	804	—	-3229.30	0.00	-3418.84	393.79
				1592	—	-1842.62	-71.68	-3798.01	216.11 #
3	-11.25	219.0	0.0	804	—	-3229.30	0.00	-3418.84	404.64
				1592	—	-1842.62	-71.68	-3798.01	228.17 #
4	-3.75	309.0	0.0	804	—	-3229.30	0.00	-3418.84	415.49
				1582	—	-1756.95	0.00	-1212.52	233.62
5	3.75	399.0	0.0	804	—	-3229.30	0.00	-3418.84	426.35
				1582	—	-1756.95	0.00	-1212.52	237.47
6	11.25	489.0	0.0	804	—	-3229.30	0.00	-3418.84	437.20
				1582	—	-1756.95	0.00	-1212.52	241.32
7	18.75	579.0	0.0	804	—	-3229.30	0.00	-3418.84	448.05
				1582	—	-1756.95	0.00	-1212.52	245.17
8	26.25	669.0	0.0	804	—	-3229.30	0.00	-3418.84	458.91
				1582	—	-1756.95	0.00	-1212.52	249.02

$P_u = 459^K$

Hand Calcs $= 451^K$ ✓

OK

Pile Reactions, Service (After the reduction of overstress allowance)

Pile	Loc(X) ft	X in	Z in	comb	Ovs	P kips	Mxx kft	Mzz kft	Pile Reac. kips
1	-26.25	39.00	0.0	1	1.000	-2103.52	0.00	-1984.80	254.16
				790	1.500	-1606.91	0.00	-3604.24	116.06
2	-18.75	129.00	0.0	1	1.000	-2103.52	0.00	-1984.80	260.46
				789	1.500	-1535.52	-59.74	-2519.93	123.48 #
3	-11.25	219.00	0.0	1	1.000	-2103.52	0.00	-1984.80	266.76
				789	1.500	-1535.52	-59.74	-2519.93	128.81 #
4	-3.75	309.00	0.0	1	1.000	-2103.52	0.00	-1984.80	273.06
				779	1.500	-1464.13	0.00	-365.35	130.47
5	3.75	399.00	0.0	1	1.000	-2103.52	0.00	-1984.80	279.37
				779	1.500	-1464.13	0.00	-365.35	131.25
6	11.25	489.00	0.0	1	1.000	-2103.52	0.00	-1984.80	285.67
				779	1.500	-1464.13	0.00	-365.35	132.02
7	18.75	579.00	0.0	1	1.000	-2103.52	0.00	-1984.80	291.97
				779	1.500	-1464.13	0.00	-365.35	132.79
8	26.25	669.00	0.0	1	1.000	-2103.52	0.00	-1984.80	298.27
				779	1.500	-1464.13	0.00	-365.35	133.57

Footing Design : Notes

Piles are collinear and there exists an unbalanced moment in the direction perpendicular to the line of piles.
Only max. force in piles is considered for design.
Pile coordinates X and Z are from the most left edge of the footing.

Max. Pile Reaction Used in Design: (without selfweight and surcharge)

Factored pile reaction	441.65 kips
Service pile reaction	294.80 kips
Fatigue pile reaction	71.00 kips

Reinforcement Schedule

Dir	Quantity	Size	Bar dist. in	As total in^2	Spacing in	Hook
X	5	# 8	4.00	3.95	10.25	None
X	5	# 8	32.00	3.95	10.25	None
Z	58	# 6	3.00	25.52	12.30	None
Z	58	# 6	34.50	25.52	12.30	None

Flexure

Dir ft	Loc	d in	Mmax kft	Comb	Asb_req in^2	Asb_prv in^2	Asb_eff in^2	Ast_req in^2	Ast_prv in^2	Ast_eff in^2
X	-29.25	32.00	0.0	804	0.00	3.95	0.40	0.00	3.95	0.62
X	29.25	32.00	0.0	804	0.00	3.95	0.40	0.00	3.95	0.62
Z	-1.50	33.00	0.0	804	0.00	25.52	8.51	0.00	25.52	10.76
Z	1.50	33.00	0.0	804	0.00	25.52	8.51	0.00	25.52	10.76

Cracking/Fatigue

Dir	Loc ft	d in	Cracking Mmax kft	Cracking Comb	Cracking fs ksi	Cracking ratio fs	Fatigue Mmax kft	Fatigue Comb	Fatigue fs ksi	Fatigue ratio fs
X	-29.25	32.00	0.0	1	0.00	0.00	0.0	1	0.00	0.00
X	29.25	32.00	0.0	1	0.00	0.00	0.0	1	0.00	0.00
Z	-1.50	33.00	0.0	1	0.00	0.00	0.0	1	0.00	0.00
Z	1.50	33.00	0.0	1	0.00	0.00	0.0	1	0.00	0.00

One Way Shear

Col	Dir	Dist ft	Comb	d in	Vu kips	phi*Vc kips
1	X	-31.92	Outside of Footing	—	—	—
	X	31.92	Outside of Footing	—	—	—
	Z	-4.25	Outside of Footing	—	—	—
	Z	4.25	Outside of Footing	—	—	—

Two Way Shear

#	Bo ft	Ao ft^2	Comb	Avg. d in	Vu kips	phi*Vc kips
Columns						
1	No Two Way Shear					
Piles - max						
0	0.00	0.00	804	32.50	0.0	0.0
Piles - min						
0	0.00	0.00	804	32.50	0.0	0.0

Two Way Shear Note

TWO WAY SHEAR IN FOOTING IS NOT DESIGNED AND STIRRUPS ARE NOT CONSIDERED.

E. SUBSURFACE EXPLORATION

(PREPARED BY DLZ, INC. OCTOBER, 2007)

Final Report of:

Subsurface Exploration
Bridge and Retaining Walls
Relocated Shumway Hollow Road Over CSXT Railroad
Project SCI-823-6.81 Portsmouth Bypass (PID 19415)
Scioto County, Ohio

Prepared for:

TranSystems Corporation
5747 Perimeter Drive, Suite 240
Dublin, Ohio 43017

Ohio Department of Transportation
District 9

DLZ Ohio, Inc.
6121 Huntley Road
Columbus, OH 43229
Phone: (614) 888-0040
Fax: (614) 436-0161

DLZ Job No. 0121-3070.03
October 1, 2007

Prepared by:

DLZ

REPORT

OF

SUBSURFACE EXPLORATION

FOR

BRIDGE AND RETAINING WALLS

RELOCATED SHUMWAY HOLLOW ROAD OVER CSXT RAILROAD

PROJECT SCI-823-6.81 PORTSMOUTH BYPASS (PID 19415)

SCIOTO COUNTY, OHIO

For:

TranSystems Corporation
5747 Perimeter Drive, Suite 240
Dublin, Ohio 43017

By:

DLZ

DLZ OHIO, INC.
6121 Huntley Road
Columbus, Ohio 43229

DLZ Job. No. 0121-3070.03

October 1, 2007

TABLE OF CONTENTS

REPORT
OF
SUBSURFACE EXPLORATION
FOR
BRIDGE AND RETAINING WALLS
RELOCATED SHUMWAY HOLLOW ROAD OVER C
PROJECT SCI-823-6.81 PORTSMOUTH BYPASS
SCIOTO COUNTY, OHIO

Apr. 19, 2007 report?

1.0 INTRODUCTION

This report includes the findings of evaluations for foundations and retaining walls for the structure at the relocated Shumway Hollow Road over the CSXT Railroad. The retaining walls evaluated include mechanically stabilized earth (MSE) and drilled shaft retaining walls. The findings included in this report pertain to the structure at relocated Shumway Hollow Road over the CSXT railroad only and supercede recommendations presented in previous reports dated November 20, 2006 and April 19, 2007. The findings of other structure evaluations will be submitted in separate documents. This document presents updated recommendations for foundations and retaining walls at the forward abutment location.

The purpose of this exploration was to 1) determine the subsurface conditions to the depths of the borings, 2) evaluate the engineering characteristics of the subsurface materials, and 3) provide information to assist in the design of the structure foundations, MSE walls, and the approach embankments. The exploration presented in this report was performed essentially in accordance with DLZ Ohio, Inc.'s (DLZ) proposal for the project.

The geotechnical engineer has planned and supervised the performance of the geotechnical engineering services, considered the findings, and prepared this report in accordance with generally accepted geotechnical engineering practices. No other warranties, either expressed or implied, are made as to the professional advice included in this report.

2.0 GENERAL PROJECT INFORMATION

The currently proposed structure is shown on the provided plan and profile drawing in Appendix I. This portion of the project consists of constructing a single-span bridge on relocated Shumway Hollow Road over the CSXT Railroad. It is anticipated that the proposed rear abutment will be founded on a fill section, contained using an MSE wall. It is also anticipated that the forward abutment will be founded on the slope of a soil/rock cut near station 37+90 (Shumway Hollow stationing) using a drilled shaft foundation / retaining wall system.

Based upon the structure plan and profile drawing, it is assumed that the maximum height of the fill at station 36+70 (rear abutment) will be approximately 43 feet. It is understood that the forward abutment will be founded on the slope of a soil/rock cut near station 37+90. The proposed roadway grade at the structure varies from approximate elevation 660 to 662.

The analyses and recommendations presented in this report have been made on the basis of the foregoing information. If the proposed locations or structural concepts are changed or differ from that assumed, DLZ should be informed of the changes so that recommendations and conclusions presented in this report may be revised as necessary.

3.0 FIELD EXPLORATION

The field exploration consisted of drilling a total of six structural borings. Borings B-24 through B-27 were drilled for the currently proposed bridge plan, essentially consisting of proposed Shumway Hollow Road over CSXT Railroad, as shown on the structural site plan in Appendix I. Structure borings, B-24 through B-27 were drilled between January 17 and 30, 2007. Structure borings TR-27 and TR-28 were drilled on August 25, 2004 and February 2, 2005, respectively for a previous design configuration. The boring logs are presented in Appendix II. Information concerning the drilling procedures is also presented in Appendix II.

The boring locations were planned and staked in the field by representatives of DLZ. The surveyed locations and ground surface elevations of the borings were determined by representatives from Lockwood, Lanier, Mathias & Noland, Inc. (2LMN).

4.0 FINDINGS

4.1 Geology of the Site

The area of this structure is characterized by gently sloping to steeply sloping topography. The project area is located in the Shawnee-Mississippian Plateau of the unglaciated portion of the Appalachian Plateau Physiographic Region. The Shawnee-Mississippian Plateau is characterized by Devonian aged to Pennsylvanian aged rocks and contains residual, colluvial, glacial, alluvial, and lacustrine soils.

The genesis of the soils varies across the site. Residual and colluvial soils are found on the ridge tops and the hillsides near the site. These soils are generally thin to moderately deep, covering moderate to steep slopes. Lacustrine soils in this area are commonly known as "Minford Silts" or the Minford Complex. These deposits were formed during the early to middle Pleistocene age when the northward flowing Teays River system was blocked by the southward advance of the Kansan aged ice sheets. As the glaciers advanced, the course of the Teays River was blocked south of Chillicothe and a large lake was formed from the impoundment of the waterways. As a result of the impoundment, vast quantities of sediments were deposited ranging from 10 to 80 feet in thickness, thinning towards the margins. The area of soils of the Minford Complex generally overlie a layer of sand and gravel which is directly above bedrock. In this area, the Minford Complex is characterized by clays of high plasticity and high moisture content. Although borings drilled for this structure did not encounter soils of the Minford Complex, several other borings drilled for the Shumway Hollow / SR 823 interchange did encounter these soils.

Bedrock within the structure area is primarily sandstone of the Logan Formation of Mississippian Age. Bedrock of the Pennsylvanian Breathitt Formation can be found at the top of the slopes to the west of the structure, roughly above elevation 860.

4.2 Field Reconnaissance

The proposed structure location lies in a shallow railroad cut located immediately west of SR 335. A visual inspection of the cut slope near the forward abutment was performed on September 15, 2006. A log of the exposed rock was created and is included in Appendix II. The cut consists of moderately steep to steep slopes of soil and rock, which are approximately 30 feet high. Elevations cited in the field reconnaissance should be considered approximate due to the accuracy of elevations reported by the field equipment.

At the eastern slope, the soil was relatively thin, and consisted primarily of residual and colluvial soils. Under the soil, exposed sandstone was evident, beginning approximately at elevation 645. The exposed rock was highly weathered and highly fractured. Bands of interbedded shale or siltstone were present in the sandstone south of the proposed structure, below approximate elevation 638. Areas of isolated seepage were evident in this layer south of the proposed structure. Additionally, several high angle fractures were noted in the rock face, however, no appreciable lateral movement of the rock mass was apparent.

Reconnaissance of the site at the bottom of the cut confirmed the presence of a very soft and wet environment at the proposed rear abutment location. Drainage channels have been established along the railroad cut, which currently run near the rear abutment location. These drainage paths have deposited approximately 3 to 5 feel of soil, as confirmed by the borings drilled for the rear abutment.

4.3 Subsurface Conditions

The following sections present the generalized subsurface conditions encountered by the borings. For more detailed information, refer to the boring logs presented in Appendix II. Laboratory test results are presented on the boring logs and also in Appendix III.

4.3.1 Soil Conditions

Borings B-24 and B-25 were drilled for the rear abutment of the currently proposed structure. Similarly, borings B-26 and B-27 were drilled for the forward abutment of the currently proposed structure. Boring TR-27 was also considered in the evaluation of the rear abutment location. Similarly, boring TR-28 was considered in the evaluation of the forward abutment location.

Borings TR-27 and B-24 through B-27 encountered surficial material consisting of 5 to 8 inches of topsoil while boring TR-28 encountered 8 inches of asphalt concrete pavement. All borings encountered native cohesive and granular soil deposits below the surficial material. The cohesive deposits consisted mainly of medium stiff to very stiff silt and clay (A-6a), medium stiff clay (A-6b), and medium stiff to hard sandy silt (A-4a), while the granular soil deposits consisted mainly of loose to dense coarse and fine sand (A-3a) and medium dense sand (A-3). Boring B-26 encountered a relatively thin soft silt and clay (A-6a) layer (approximately 2-foot thick) above the sandstone. The native soil deposits were 3.0 feet thick at the rear abutment and between 16.5 and 17.5 feet thick at the forward abutment. It should be noted that the presence of organic material was noted in boring B-24, drilled at the rear abutment location.

4.3.2 Bedrock Conditions

In the area of the proposed structure, bedrock was encountered in all the borings and was confirmed by coring between 10 and 20 feet of rock in each boring. The bedrock consisted of medium hard to hard, slightly to highly weathered, slightly fractured sandstone. A layer of severely weathered rock, ranging in thickness between 1.5 to 3 feet was encountered above the more competent cored bedrock in borings B-24, B-25, and TR-28. The amount of rock recovered in each core run varied between 50 and 100 percent. The rock quality designation (RQD) of the bedrock ranged between 12 and 100 percent with an average of 80 percent indicating "good" rock.

Unconfined compressive strength of tested cores ranged between 9,952 psi and 13,148 psi. The tested cores correspond to samples at depths between 10.0 feet and 32.5 feet below the ground surface. A summary of the unconfined compressive strength of the tested cores is shown in Table 1. Also, the elastic modulus of selected cores was also measured. The results of these tests are presented in Appendix III.

Table 1 - Rock Core Test Results

Boring	Depth (ft)	Unit Weight (pcf)	Unconfined Compressive Strength (psi)
B-24	10.0-10.5	157.7	9,952
B-25	10.5-11.0	155.8	10,295
B-26	22.5-23.0	140.5	10,454
	32.0-32.5	146.9	12,453
B-27	21.5-22.0	136.6	13,148
	32.0-32.5	143.5	12,949

4.3.3 Groundwater Conditions

Minor seepage was encountered at depths between 8.5 and 18.5 feet below the ground surface in borings B-26, B-27, and TR-28 only. Measurable water levels prior to rock coring were encountered at depths between 14.3 and 36.1 in borings B-26 and B-27 only. Water was used during rock coring and masked any seepage zones that might exist in the rock. Measurable water levels were present in all borings except TR-27 upon the completion of coring between approximate depths of 5.5 and 35.8 feet. Boring TR-27 collapsed at a depth of 6.0 feet and did not have a measurable water level. It should be noted that the final water levels include drilling water and consequently may not be representative of the actual groundwater conditions.

It should also be noted that groundwater levels may fluctuate with seasonal variations and following periods of heavy or prolonged precipitation, and therefore, the readings indicated on the boring logs may not be representative of the long-term groundwater level. Long-term monitoring would be needed to obtain a more accurate estimate of the groundwater table elevation.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 General Information

Based upon the amount of embankment fill and the approximate depth to bedrock, spread footings, drilled shafts, or CIP piles socketed into bedrock are considered suitable to support the rear abutment. Additionally, it is understood that the forward abutment will be located on a rock/soil slope, with the centerline of bearing at approximate station 37+90. Given the highly weathered nature of the bedrock near the face of the slope, and the abutment location with respect to the slope, drilled shafts socketed into bedrock are considered best suited to support the forward abutment.

It is understood that maintenance of traffic issues at the intersection of SR 335 and relocated Shumway Hollow Road have limited the type of the foundation and retaining wall systems that may be considered at the forward abutment location. It has been expressed that at least one lane of SR 335 must remain open during the entire construction process. In accordance with this, several retaining wall options have been eliminated due to the impact of excavations or required construction limits. If this requirement is modified, alternative foundation and retaining wall system recommendations can be provided upon request.

The current configuration includes the retention of approximately 20 feet of fill at the forward abutment. Consequently, the currently proposed drilled shaft foundations will have to be designed to resist the lateral loading of the fill material. Recommendations for this drilled shaft foundation / retaining wall system at the forward abutment location are included in the following sections. The following sections also contain additional recommendations and information for the design of the proposed structural foundations and MSE wall. Table 2 summarizes the site conditions and foundation recommendations. Calculations are presented in Appendix IV.

Table 2 - Summary of Foundation Recommendation

Structural Element	Structure / Boring	Existing Ground Elevation (Feet)	Foundation Type	Approximate Bearing Elevation (Feet)	Allowable Bearing Capacity
Rear Abutment	Left / B-24	625.9	CIP Piles	615.0 *	Maximum Allowable Capacity
			Drilled Shafts	615.0 *	80 ksf++
			Spread Footings	MSE Fill	4 ksf
	Right / B-25	625.0	CIP Piles	615.0 *	Maximum Allowable Capacity
			Drilled Shafts	615.0 *	80 ksf++
			Spread Footings	MSE Fill	4 ksf
Forward Abutment	Left / B-26	660.2	Drilled Shafts	615.0+	80 ksf++
	Right TR-28 / B-27	659.7 656.8	Drilled Shafts	615.0+	80 ksf++

* Includes 5-foot socket into competent rock.
\+ Minimum tip elevation 615 for drilled shafts subject to lateral loading (forward abutment wall).
++ End bearing capacity only.

5.2 Bridge Foundation Recommendations

5.2.1 Rear Abutment (Sta. 36+70)

Spread footings bearing in the MSE wall fill could be considered to support the rear abutment. As per the Bridge Design Manual 204.6.2.1, an allowable bearing capacity of 4 ksf may be used to design the footings. The MSE wall, as proposed, will be founded on bedrock. As such, the anticipated settlements of spread footings bearing on the fill are anticipated to be negligible.

However, it should be noted that the proposed rear abutment lies in close proximity to a drainage ditch, which runs essentially parallel to the railroad tracks. The area where the MSE wall is currently proposed is prone to frequent flooding. Consequently, over time there is a risk of migration of the select granular fill from the reinforced zone.

As an alternative to spread footings, CIP piles could be used to support the rear abutment. The CIP piles would be placed in prebored holes 12 inches larger than the diameter of the pile and 5 feet deep into bedrock. After installing the steel CIP pile in the prebored hole, grout or cement should be placed in the void area around the pile in the prebored hole prior to constructing the embankment granular fill. Pile sleeves should also be used to mitigate down drag effects from compaction and to protect the pile during the embankment and MSE wall construction. The maximum allowable pile capacity, as per ODOT BDM 202.2.3.2.b, may be utilized in this configuration.

At this time, excessive lateral loading and uplift is not anticipated to be a concern at the rear abutment. However, if these forces are significant, longer socket lengths may be required. Due to the relatively small rigidity of the steel CIP piles compared to drilled shafts, the steel CIP piles are anticipated to provide low resistance to lateral earth pressures that can be induced in high embankment fills. Therefore, the prebored and socketed steel pipe pile foundation system may not be suitable if significant lateral loads are present.

As mentioned above, drilled shafts may also be considered for the support of the rear abutment. Drilled shafts, socketed a minimum of 5 feet into competent rock are recommended to support the proposed rear abutment. This corresponds to an approximate bearing elevation of 615 at the rear abutment. The drilled shafts should be straight (not belled) and may be designed based on an allowable bearing pressure of 80 ksf (40 tsf).

If adequate capacity cannot be developed with reasonable shaft diameter, consideration should be given to the use of deeper rock sockets. Neglecting the upper two feet of the socket, allowable sidewall shear stress/adhesion of 7,500 pounds per square foot may be used. If deeper sockets are used, the shafts should be designed such that design loads are carried entirely by the socket resistance, ignoring any end bearing capacity.

5.2.2 Forward Abutment (Sta. 37+90)

The forward abutment of the proposed structure lies on the eastern slope of a railroad cut, which is approximately 33 feet deep. Based upon provided drawings and the available subsurface information, it is anticipated that approximately 20 feet of fill will be retained at the forward abutment location. Based upon these conditions, it is recommended that a drilled shaft foundation / retaining wall system be used to support the structure and retain the fill at the forward abutment location. The drilled shafts will need to be designed to resist the lateral loading from the fill material as well as any loads from the proposed structure.

From the borings, it is anticipated that competent bedrock will be encountered within 3 to 5 feet below the soil-rock interface at the proposed centerline of bearing for the forward abutment. This corresponds to an approximate elevation of 637. However, based upon field reconnaissance, it is anticipated that the degree of weathering and fractures present in the rock located at the centerline of bearing for the forward abutment will be more severe than that encountered in the borings drilled for the proposed structure, at the crest of the slope.

It is believed that significant fracturing of the rock is present at the location of the proposed abutment wall. Because of this fracturing, the resistance to lateral loading, provided by the upper bedrock could be decreased. Analyses determined that the weight of the rock wedge providing passive resistance to lateral earth pressures above elevation 630 is inadequate. Small lateral movements of the upper bedrock (passive side) could occur due to the laminated nature of the bedrock positioned on the steep rock slope. If such movements are realized, the lateral resistance will need to be provided by deeper drilled shaft sockets. Below elevation 630, although the rock is laminated, it is assumed that it will generally behave as a confined unit. This is due to the elevation relative to the bottom of the adjacent rock cut. Consequently, it is recommended that the lateral capacity provided by the drilled shaft rock socket above elevation 630 be ignored. Furthermore, it is recommended that the socket extend a minimum of 15 feet below elevation 630 to elevation 615 to resist the lateral loading. The drilled shafts should be straight (not belled) and may be designed based on an allowable end bearing pressure of 80 ksf (40 tsf). It should be noted that the required socket length cited here is based upon geotechnical considerations only. Additional socket length may be required for structural purposes such as sufficient reinforcement development length.

If sufficient axial capacity cannot be obtained with a reasonable shaft diameter, the shafts could be designed as friction-type shafts. As discussed earlier, at a minimum, the rock socket should extend to elevation 615. If designed as friction-type shafts, the shafts should be designed such that design loads are carried entirely by the socket resistance, ignoring any end bearing capacity. Any resistance provided from the rock socket above elevation 630 should also be ignored. An allowable sidewall shear stress/adhesion of 7,500 pounds per square foot may be used to design the shafts.

It is understood that only very small deflections could be tolerated by the integral-type abutment proposed at the forward abutment location. However, the drilled shaft foundation / retaining wall system proposed for the forward abutment would deflect under the influence of lateral earth pressure from the backfill materials. Based upon discussions with TranSystems, it is understood that deflections of the drilled shaft foundation / retaining wall prior to making the attachment to the superstructure should be limited to 1/8 of an inch or less, which is typically cited as being within acceptable construction tolerances. Considering the top of rock and the proposed grade at the forward abutment location, it is assumed that 14 feet of fill will be placed behind the drilled shaft foundation / retaining wall system prior to making the connection to the superstructure and placing the remaining fill. Based upon this assumption, it is evident that a deflection-based design, to limit the amount of deflection, is appropriate for the proposed drilled shaft foundation / retaining wall system. Considering the amount of allowable deflection and the compaction effort required for the backfill materials, it is recommended that the at-rest condition be assumed for analyses that include lateral earth pressures.

Originally, a tangent drilled shaft foundation / retaining wall system was considered as a possible configuration. However, this type of shaft layout is more difficult to construct due to the strict tolerance required to avoid overlap of the shafts and conflicts between the temporary casing and the adjacent shafts. Consequently, a configuration utilizing a drilled shaft spacing wider than the shaft diameter was assumed for the analyses. To complete the retaining wall, the void space between shafts will have to be filled, such as with bentonite-cement grout or with some form of lagging.

Several LPILE analyses were performed to determine a preliminary configuration (diameter, reinforcement ratio, and spacing) for the proposed drilled shaft foundation / retaining wall. Based upon the prescribed deflection criterion and discussions with TranSystems, the use of 48-inch diameter, reinforced concrete drilled shafts on a 60-inch center–to-center spacing is recommended to support the structure and retain the approach embankment fill. As discussed above, a permanent lagging will be required for this drilled shaft system. It is understood that the design of lagging will be provided by others.

It is understood that the structural design of the drilled shafts will be determined by others. However, an estimate of the longitudinal reinforcing steel was required in order to model the rock-structure interaction while using a non-linear bending stiffness in the LPILE program. If final design uses a reinforcement ratio, diameter, or spacing that differs significantly from that which was assumed, DLZ should be informed so that the model and recommendations may be revised as necessary. A sample LPILE output file is presented in Appendix IV. Additionally, a summary of the unfactored shear forces and bending moments generated in the drilled shafts from the lateral earth pressures for various configurations is also presented in Appendix IV.

5.2.3 Drilled Shaft Foundations: General Recommendations

For end bearing drilled shafts it is recommended that skin friction in the overburden soil/fill and shallow rock socket be neglected. The bearing surface should be clean and free of loose material and water prior to placement of concrete. A qualified representative of the Geotechnical Engineer should field verify that the drilled shafts are founded on competent bearing materials and the installation procedures meet specifications.

Precautions should be taken to permit the shafts to be drilled and the concrete placed under relatively dry conditions. Although the borings did not encounter significant seepage, water could flow into the drilled shafts during installation particularly below the drainage channel level (rear abutment) and within wet zones that may be present in the rock or soil. It should be anticipated that materials across the site could vary considerably and temporary casing will be required during the drilling and concrete placement to seal out water seepage in the overburden and prevent cave-in. During simultaneous concrete placement and casing removal operations, sufficient concrete should be maintained inside the casing to offset the hydrostatic head of any groundwater. Extreme care must be exercised during concrete placement and removal of the casing so that soil intrusion is avoided.

Special considerations need to be given to the use of drilled shaft foundations with MSE walls. If drilled shafts are used at the rear abutment, the drilled shafts should be set back from the MSE wall panel a sufficient distance to allow reinforcing straps to be splayed around the shafts at an angle of 15 degrees or less.

5.3 Mechanically Stabilized Earth (MSE) Retaining Wall Recommendations

It is understood that an MSE wall will be used to construct the embankment and contain the rear abutment at station 36+75. Recommendations for this MSE wall are presented in the following sections. The MSE wall should be constructed according to the recommendations presented in this report and in conformance with the manufacturer's specifications.

5.3.1 MSE Walls: General Information

An MSE retaining wall essentially consists of good quality backfill material with layers of metal or plastic reinforcing that are attached to concrete facing panels. The MSE wall and associated backfill should be constructed in accordance with the specifications of the manufacturer of the MSE wall.

The parameters required to perform the stability analyses are presented in Table 3, below. As outlined in section 5.3.2, it is recommended that the existing soils at the rear abutment location be removed to the top of rock and replaced with compacted granular fill. Consequently, the properties of the compacted granular fill are assumed for the foundation soil used in the stability and bearing capacity calculations of the MSE wall. In accordance with ODOT guidelines, a unit weight of 120 pcf and a friction angle of 34 degrees were selected for the backfill material in the reinforced zone. Additionally, the fill material used to construct the roadway embankments is assumed to have a unit weight of 120 pcf and a friction angle of 30 degrees. If the embankment fill material or backfill material for the reinforcing zone has properties significantly different from these values, DLZ should be informed so that the analyses may be revised as necessary.

Table 3 - Soil Parameters Used in MSE Wall Stability Analyses

Zone	Soil Type	Unit Weight (pcf)	Strength Parameters			
			Undrained		Drained	
			c	ϕ	c'	ϕ'
Reinforced Fill	Compacted Granular Fill	120	0	34	0	34
Retained Soil	Compacted Embankment Fill	120	0	30	0	30
Foundation Soil (Rear Abutment)	Compacted Granular Fill	120	0	34	0	34

5.3.2 MSE Wall Evaluations and Recommendations

The MSE fill at the rear abutment is understood to have a maximum height of approximately 43 feet. Borings drilled at the rear abutment first encountered bedrock at 3.0 to 5.0 feet below the ground surface. Based on the field reconnaissance of the site, very soft and wet soils were observed at the proposed rear abutment location. The conditions at the site may vary greatly depending upon the amount of recent precipitation. Consequently, it is recommended that soils overlying bedrock be removed, and the leveling pad be placed on bedrock. Additionally, provisions for diverting water away from the proposed MSE wall should be made to prevent any scour of the MSE wall materials.

If the MSE wall is founded on bedrock, the bearing capacity, global stability, and settlement of the wall are assumed to be adequate and thus calculations are not necessary. Calculations for stability (sliding and overturning) and bearing capacity are included in Appendix IV. Other internal stability (i.e. strap design) analyses are required for the design of an MSE wall, but are considered outside the scope of this report. For stability, calculations have shown that a minimum reinforcement length of 0.7 times the full wall height, or 30.1 feet, should be used for the proposed MSE wall at this location. This length is a minimum and may be increased if necessary for internal stability.

A summary of soil properties, summary of the results of calculations, and MSE retaining wall parameters are presented in Table 4.

Table 4 - MSE Retaining Wall Parameters and Results of Analyses (Rear Abutment) ***Borings B-24 & B-25***

<u>Retained Soil (New Embankment)</u> Unit Weight = 120 pcf Coefficient of Active Earth Pressure (K_a) = 0.33 (Based on $\Phi = 30°$)
<u>Sliding along base of MSE wall</u> Sliding Coefficient (μ)(0.67) = tan 34°(0.67) = 0.45
<u>Allowable Bearing Capacity – Drained Condition</u> q_{all} = 12,129 psf (Assumes Compacted Granular Fill Foundation on Rock)
<u>Global Stability</u> Factor of Safety – Undrained Condition > 1.5 (Founded on Bedrock) Factor of Safety – Drained Condition > 1.5 (Founded on Bedrock) Factor of Safety – Seismic Condition > 1.3 (Founded on Bedrock)
<u>Estimated Settlement of MSE volume</u> Total settlement = negligible (Assumes Compacted Granular Fill on Rock) Differential settlement < 1/100
Approximate Maximum Height of MSE Wall = 43.0 feet Approximate Embedment Depth = 0.0 feet (Bedrock) Minimum Length of Reinforcement for External Stability = (0.7)(H+D) or 30.1 feet

5.4 General Earthwork Recommendations

Only boring B-24 encountered organic material in the borings drilled for the proposed structure. However, since organic or very soft soils may be encountered at locations other than where the borings were drilled, the contractor should be prepared to perform overexcavation of any poor soils at other locations and replace the overexcavated soil with compacted engineered fill as needed. Additionally, all topsoil, organic soil within 3 feet of subgrade level, and vegetation should also be removed prior to placing fill or pavement materials. For satisfactory performance of the proposed MSE wall, it is recommended that the existing soils be overexcavated to the top of rock and replaced with compacted granular fill. The area of overexcavation should extend beyond the edge of the MSE wall/select granular footprint by a distance equal to the depth of the aggregate base or 3 feet, whichever is greater.

Durable sandstone is evident at the rear abutment location in the rock cut. Significant rock excavation to accommodate the reinforcing straps of the MSE retaining wall is not anticipated at this time. However, if necessary, the contractor should be prepared to excavate hard, durable sandstone by blasting or other appropriate means. In places where fill is to be placed on bedrock, a level bench should be cut into the bedrock prior to the placement of fill for stability purposes.

5.5 Groundwater Considerations

Minor seepage was encountered in borings TR-28, B-26, and B-27. In these borings, seepage was first encountered at depths ranging from 8.5 to 14.0 feet below the ground surface. Groundwater was noted prior to adding drilling water in boring B-27 at a depth of 14.4 feet. Representative, final water levels could not be obtained due to the use of water during rock coring operations. The use of drilling water in rock coring operations also masked any seepage zones that may be present in the rock. Excavations for shafts extending below the soil-rock interface may encounter significant seepage through fractured zones in the rock.

Based on the field reconnaissance of the site, very soft and wet soils were observed at the proposed rear abutment location. Consequently, the contractor should anticipate significant seepage in the excavations for the proposed MSE retaining wall. The contractor should also be prepared to deal with any unexpected seepage, precipitation, or water flow that may enter any excavations.

5.6 Scour Analysis

Particle-size analyses were performed on samples collected in the area of the rear abutment for possible scour analysis. The flow line elevation in the existing drainage channel is reported to be approximate elevation 623.1. Table 5 presents the sample locations and the D_{50} and D_{85} sizes from the particle size analyses. The samples tested are considered representative of the alluvial material, which has been deposited in the area of the proposed rear abutment.

Table 5 – Particle Size Data

Boring Number	Ground Surface Elevation ft. (at boring)	Sample Depth (below ground surface)	ODOT Classification	D_{85} (mm)	D_{50} (mm)
B-24	625.9	1.0-2.5	A-6a	0.0337	0.0100
B-25	625.0	1.0-2.5	A-6a	0.0255	0.0061

6.0 CLOSING REMARKS

We appreciate having the opportunity to be of service to you on this project. Please do not hesitate to call if you have any questions concerning our report.

Respectfully submitted,

DLZ OHIO, INC.

Steven J. Riedy
Geotechnical Engineer

Dorothy A. Adams (SJR)

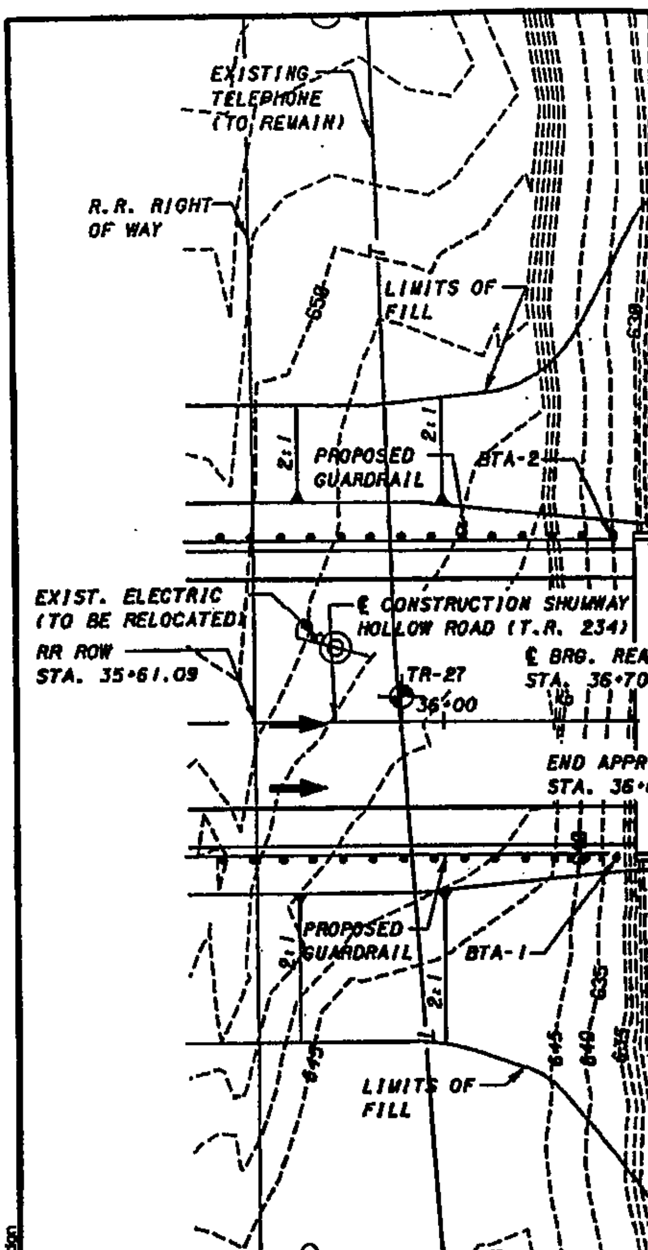
Dorothy A. Adams, P.E.
Senior Geotechnical Engineer

sjr

M:\proj\0121\3070.03\Stability Analyses\Documents\MSE Wall letters\Shumway Hollow Road over CSXT RR\Final\Shumway Hollow Rd over CSXT Report sjr 10-1-2007.doc

APPENDIX I

Structure Plan and Profile Drawing – 11"x17"



GUARDRAIL POST		
LOCATION	STATION	OFFSET
REAR ABUT	36+35.37	38' LEFT
REAR ABUT	36+35.24	28' RIGHT

BENCHMARK 1	BENCHMARK 2
(TO BE PROVIDED LATER)	(TO BE PROVIDED LATER)

TRAFFIC DATA

(SHUMWAY HOLLOW ROAD)
CURRENT YEAR ADT (2010) = 3,800
DESIGN YEAR ADT (2030) = 7,800
CURRENT YEAR ADTT (2010) = 228
DESIGN YEAR ADTT (2030) = 468

TABLE OF VERTICAL CLEARANCES

LOCATION	"A"	"B"	"C"	"D"	"E"*	"F"*
PROPOSED	24.16'	23.49'	24.30'	23.52'	24.29'	23.46'
REQUIRED	23.0'	23.0'	23.0'	23.0'	23.0'	23.0'

* 6'-0" FROM ℄ CSXT TRACK 2

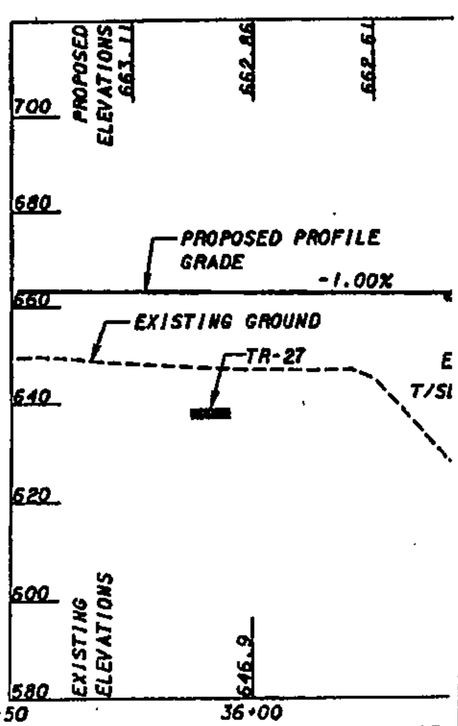
NOTES:

1. EARTHWORK LIMITS SHOWN ARE APPROXIMATE. ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.

FOUNDATION DATA:
REAR ABUTMENT SHALL BE ON SPREAD FOOTING WITH AN ALLOWABLE BEARING CAPACITY OF 4 KSF. FORWARD ABUTMENT SHALL DRILLED SHAFTS WITH AN ALLOWABLE BEARING CAPACITY OF 40 KSF.

LEGEND

BTA-1 = BRIDGE TERMINAL ASSEMBLY TYPE 1
BTA-2 = BRIDGE TERMINAL ASSEMBLY TYPE 2
= BORING LOCATION



PROPOSED STRUCTURE

TYPE: SINGLE SPAN 72" MODIFIED AASHTO TYPE 4 PRESTRESSED CONCRETE I-BEAMS WITH COMPOSITE REINFORCED CONCRETE DECK ON SEMI-INTEGRAL REAR ABUTMENT AND INTEGRAL FORWARD ABUTMENT.

SPANS: 119'-6" C/C BEARINGS

ROADWAY: 66'-0" TOE TO TOE OF BARRIER

LOADING: HS-25 AND ALTERNATE MILITARY LOADING, FWS = 60 PSF

SKEW: NONE

CROWN: VARIES

ALIGNMENT: TANGENT

WEARING SURFACE: MONOLITHIC CONCRETE

APPROACH SLABS: AS-1-81 (30'-0" LONG)

LATITUDE: 38° 50' 30" N

LONGITUDE: 82° 51' 00" W

DESIGNED MSL | CHECKED PJP | DRAWN CAS | REVIEWED MTN | REVISED JRC | DATE 10/12/06 | STRUCTURE FILE NUMBER 7336934

SCIOTO COUNTY
STA. 36+69.50
STA. 37+91.00

SITE PLAN
BRIDGE NO. SCI-TR234-0122
SHUMWAY HOLLOW ROAD OVER CSXT RAILROAD

SCI-823-6.81
PID 19415

1 / 4

568 / 571

3/5/2007

APPENDIX II

General Information – Drilling Procedures and Logs of Borings

Legend – Boring Log Terminology

Boring Logs – Six (6) Borings

Log of Rock Cut – Eastern Slope

GENERAL INFORMATION
DRILLING PROCEDURES AND LOGS OF BORINGS

Drilling and sampling were conducted in accordance with procedures generally recognized and accepted as standardized methods of investigation of subsurface conditions concerning geotechnical engineering considerations. Borings were drilled with either a truck-mounted or ATV-mounted drill rig.

Drive split-barrel sampling was performed in 1.5 foot increments at intervals not exceeding 5 feet. In the event the sampler encountered resistance to penetration of 6 inches or less after 50 blows of the drop hammer, the sampling increment was discontinued. Standard penetration data were recorded and one or more representative samples were preserved from each sampling increment.

In borings where rock was cored, NXM or NQ size diamond coring tools were used.

In the laboratory all samples were visually classified by a geotechnical engineer. Moisture contents of representative fine-grained soil samples were determined. A limited number of samples, considered representative of foundation materials present, were selected for performance of grain-size analyses and plasticity characteristics tests. The results of these tests are shown on the boring logs.

The boring logs included in the Appendix have been prepared on the basis of the field record of drilling and sampling, and the results of the laboratory examination and testing of samples. Stratification lines on the boring logs indicating changes in soil stratigraphy represent depths of changes approximated by the driller, by sampling effort and recovery, and by laboratory test results. Actual depths to changes may differ somewhat from the estimated depths, or transitions may occur gradually and not be sharply defined. The boring logs presented in this report therefore contain both factual and interpretative information and are not an exact copy of the field log.

Although it is considered that the borings have disclosed information generally representative of site conditions, it should be expected that between borings conditions may occur which are not precisely represented by any one of the borings. Soil deposition processes and natural geologic forces are such that soil and rock types and conditions may change in short vertical intervals and horizontal distances.

Soil/rock samples will be stored at our laboratory for a period of six months. After this period of time, they will be discarded, unless notified to the contrary by the client.

LEGEND – BORING LOG TERMINOLOGY

Explanation of each column, progressing from left to right

1. Depth (in feet) – refers to distance below the ground surface.

2. Elevation (in feet) – is referenced to mean sea level, unless otherwise noted.

3. Standard Penetration (N) – the number of blows required to drive a 2-inch O.D., 1-3/8 inch I.D., split-barrel sampler, using a 140-pound hammer with a 30-inch free fall. The blows are recorded in 6-inch drive increments. Standard penetration resistance is determined from the total number of blows required for one foot of penetration by summing the second and third 6-inch increments of an 18-inch drive.

   50/n – indicates number of blows (50) to drive a split-barrel sampler a certain number of inches (n) other than the normal 6-inch increment.

4. The length of the sampler drive is indicated graphically by horizontal lines across the "Standard Penetration" and "Recovery" columns.

5. Sample recovery from each drive is indicated numerically in the column headed "Recovery".

6. The drive sample location is designated by the heavy vertical bar in the "Sample No., Drive" column.

7. The length of hydraulically pressed "Undisturbed" samples is indicated graphically by horizontal lines across the "Press" column.

8. Sample numbers are designated consecutively, increasing in depth.

9. Soil Description

   a. The following terms are used to describe the relative compactness and consistency of soils:

   Granular Soils – Compactness

Term	Blows/Foot Standard Penetration
Very Loose	0 – 4
Loose	4 – 10
Medium Dense	10 – 30
Dense	30 – 50
Very Dense	over 50

   Cohesive Soils – Consistency

Term	Unconfined Compression tons/sq.ft.	Blows/Foot Standard Penetration	Hand Manipulation
Very Soft	less than 0.25	below 2	Easily penetrated by fist
Soft	0.25 – 0.50	2 – 4	Easily penetrated by thumb
Medium Stiff	0.50 – 1.0	4 – 8	Penetrated by thumb with moderate pressure
Stiff	1.0 – 2.0	8 – 15	Readily indented by thumb but not penetrated
Very Stiff	2.0 – 4.0	15 – 30	Readily indented by thumb nail
Hard	over 4.0	over 30	Indented with difficulty by thumb nail

   b. Color – If a soil is a uniform color throughout, the term is single, modified by such adjective as light and dark. If the predominant color is shaded by a secondary color, the secondary color precedes the primary color. If two major and distinct colors are swirled throughout the soil, the colors are modified by the term "mottled".

   c. Texture is based on the Ohio Department of Transportation Classification System. Soil particle size definitions are as follows:

Description		Size	Description		Size
Boulders		Larger than 8"	Sand	– Coarse	2.0 mm to 0.42 mm
Cobbles		8" to 3"		– Fine	0.42 mm to 0.074 mm
Gravel	– Coarse	3" to ¾"	Silt		0.074 mm to 0.005 mm
	– Fine	¾" to 2.0 mm	Clay		smaller than 0.005 mm

d. The main soil component is listed first. The minor components are listed in order of decreasing percentage of particle size.

e. Modifiers to main soil descriptions are indicated as a percentage by weight of particle sizes.

trace	0 to 10%
little	10 to 20%
some	20 to 35%
"and"	35 to 50%

f. Moisture content of **cohesionless soils** (sands and gravels) is described as follows:

Term	Relative Moisture or Appearance
Dry	No moisture present
Damp	Internal moisture, but none to little surface moisture
Moist	Free water on surface
Wet	Voids filled with free water

g. The moisture content of **cohesive soils** (silts and clays) is expressed relative to plastic properties.

Term	Relative Moisture or Appearance
Dry	Powdery
Damp	Moisture content slightly below plastic limit
Moist	Moisture content above plastic limit but below liquid limit
Wet	Moisture content above liquid limit

10. Rock Hardness and Rock Quality Designation

a. The following terms are used to describe the relative hardness of the **bedrock.**

Term	Description
Very Soft	Permits denting by moderate pressure of the fingers. Resembles hard soil but has rock structure. (Crushes under pressure of fingers and/or thumb)
Soft	Resists denting by fingers, but can be abraded and pierced to shallow depth by a pencil point. (Crushes under pressure of pressed hammer)
Medium Hard	Resists pencil point, but can be scratched with a knife blade. (Breaks easily under single hammer blow, but with crumbly edges.)
Hard	Can be deformed or broken by light to moderate hammer blows. (Breaks under one or two strong hammer blow, but with resistant sharp edges.)
Very Hard	Can be broken only by heavy and in some rocks repeated hammer blows.

b. Rock Quality Designation, RQD – This value is expressed in percent and is an indirect measure of rock soundness. It is obtained by summing the total length of all core pieces which are at least four inches long, and then dividing this sum by the total length of the core run.

11. Gradation – when tests are performed, the percentage of each particle size is listed in the appropriate column (defined in Item 9c).

12. When a test is performed to determine the natural moisture content, liquid limit moisture content, or plastic limit moisture content, the moisture content is indicated graphically.

13. The standard penetration (N) value in blows per foot is indicated graphically.

DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-24 | Location: Sta. 36+59.8, 43.4 ft. LT of Rel. Shumway Hollow CL | Date Drilled: 1/17/07

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: None (prior to coring)
6.6' (inside hollowstem augers, includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N)
0 – 0.6	625.9 – 625.3						Topsoil - 7"							
0.6 – 3.0	625.3 – 622.9	WOH 1 1	18	1		1.0	Medium stiff to stiff brown SILT AND CLAY (A-6a), trace fine sand; contains organic material; damp to moist.	0	1	–	2	65	33	
3.0 – 5.0	622.9 – 620.9	9 50/3	5	2			Severely weathered gray SANDSTONE, argillaceous, micaceous.							50+
5.0 – 15.0	620.9 – 610.9	Core 120"	Rec 120"	RQD 90%	R1		Hard gray SANDSTONE; fine grained, slightly weathered, argillaceous, micaceous, thinly laminated, slightly fractured. @ 5.2', 5.4', 6.9', 7.9', 11.3', 12.3', 12.6', 13.8', low angle clay filled fractures. @ 10.0'-10.5', qu = 9,952 psi.							
15.0	610.9						Bottom of Boring - 15.0'							

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ├──┤ LL
Blows per foot - ○
10 20 30 40

DLZ OHIO INC. * 6121 HUNTLEY ROAD, C[illegible]MBUS, OHIO 43229 * (614)888-0040

Client: Tran[illegible]ms, Inc. | Project: SCI-823-0.00 | Job No. 0121[illegible]0.03

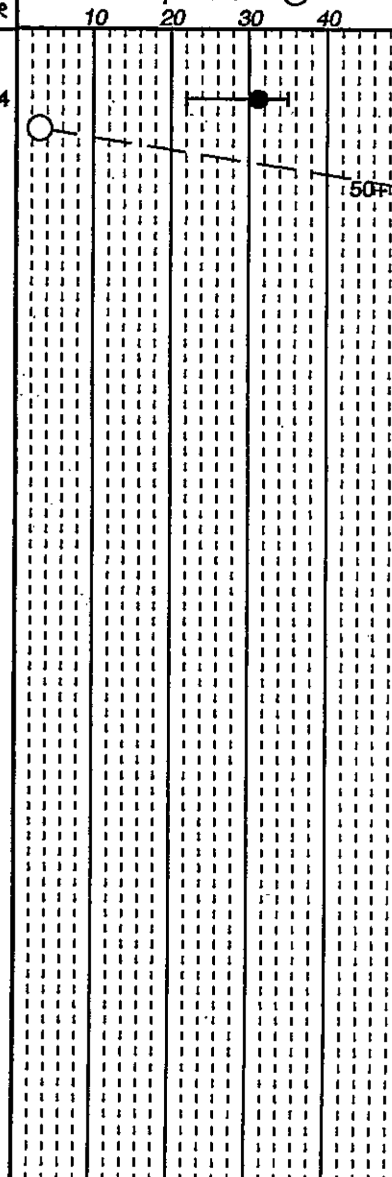
LOG OF: Boring B-25 | Location: Sta. 36+59.3, 49.2 ft. RT of Rel. Shumway Hollow CL | Date Drilled: 1/17/07

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: None (prior to coring)
5.5' (inside hollowstem augers, includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	625.0												
0.5	624.5						Topsoil - 6"						
		1 1 2	18	1		1.25	Stiff brown SILT AND CLAY (A-6a), trace fine sand, trace gravel; damp to moist.	2	0	–	2	53	44
3.0	622.0	50/2	1	2			Severely weathered gray SANDSTONE, argillaceous, micaceous.						
4.5	620.5	Core 120"	Rec 120"	RQD 85%	R1		Hard gray SANDSTONE interbedded with SILTSTONE; very fine to fine grained, slightly weathered, argillaceous, micaceous, thinly laminated, slightly fractured. @ 4.5'-5.0', 6.2'-7.0', broken zones. @ 10.5'-11.0', qu = 10,295 psi. @ 11.6', 11.7', 13.5', low angle clay filled fractures.						
14.5	610.5						Bottom of Boring - 14.5'						

Depth scale: 0, 5, 10, 15, 20, 25, 30

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ├──┤ LL
Blows per foot - ○



DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

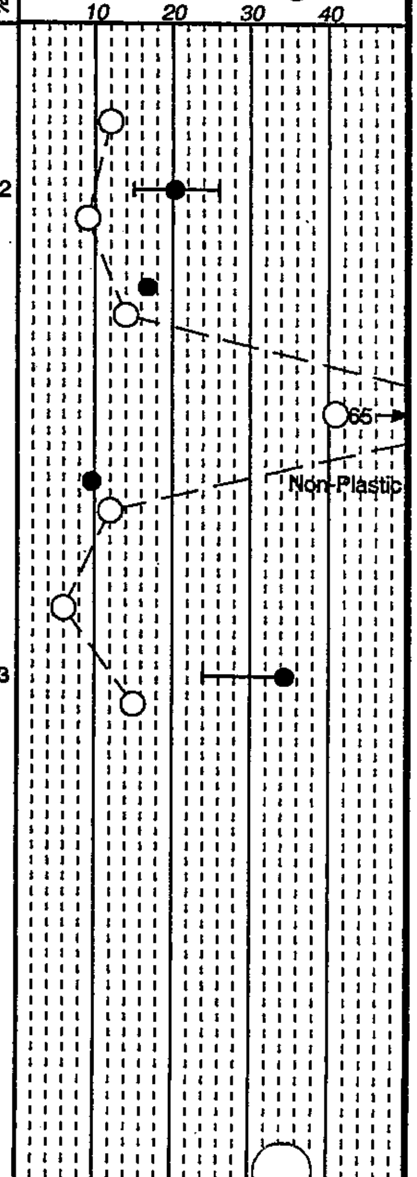
Client: Trans[illegible]ems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-26 | Location: Sta. 38+06.7, 35.3 ft. LT of Rel. Shumway Hollow CL | Date Drilled: 01/30/07

WATER OBSERVATIONS: Water seepage at: 13.5'-15.0'
Water level at completion: 36.1' (prior to coring)
18.3' (inside hollowstem augers, includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	660.2												
0.7	659.5						Topsoil - 8"						
		WOH 5 7	3	1		1.25	Medium stiff to stiff brown SANDY SILT (A-4a), trace to little clay; damp.						
5		1 4 5	18	2		0.75		3	3	–	32	30	32
5.5	654.7	4 7 7	18	3			Medium dense brown FINE SAND (A-3), trace coarse sand, trace silty clay; damp.						
10		45 37 28	18	4			@ 8.5'-10.0', very dense.						
		5 6 6	18	5			@ 11.0', trace clay.	0	7	–	88	6	
15		7 3 3	12	6			@ 13.5', loose, wet.						
15.5	644.7	WOH 6 9	18	7		0.5	Soft to medium stiff brown SILT AND CLAY (A-6a), trace fine sand; moist to wet.	0	0	–	0	56	43
17.5	642.7						Hard gray SANDSTONE; medium grained, unweathered to slightly weathered, micaceous, thinly bedded, slightly fractured.						
20		Core 120"	Rec 120"	RQD 100%	R1		@ 22.5'-23.0', qu = 10,454 psi, Er = 2,484,015 psi.						
25							@ 27.8'-27.9', broken zone.						
30													

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ├──┤ LL
Blows per foot - ○



DLZ OHIO INC. * 6121 HUNTLEY ROAD, C[illegible]MBUS, OHIO 43229 * (614)888-0040

Client: Tran[illegible]ms, Inc. | Project: SCI-823-0.00 | Job No. 0121-[illegible]070.03

LOG OF: Boring B-26 | Location: Sta. 38+06.7, 35.3 ft. LT of Rel. Shumway Hollow CL | Date Drilled: 01/30/07

WATER OBSERVATIONS: Water seepage at: 13.5'-15.0'
Water level at completion: 36.1' (prior to coring)
18.3' (inside hollowstem augers, includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ├─┤ LL Blows per foot - ○ (10 20 30 40)
30	630.2													
		Core 120"	Rec 120"	RQD 95%	R2		Hard gray SANDSTONE; medium grained, unweathered to slightly weathered, micaceous, thinly bedded, slightly fractured. @ 32.0'-32.5', qu = 12,453 psi.							
35														
37.5	622.7						Bottom of Boring - 37.5'							
40														
45														
50														
55														
60														

FILE: 0121-3070-03 [9/13/2007 4:26 PM]

DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

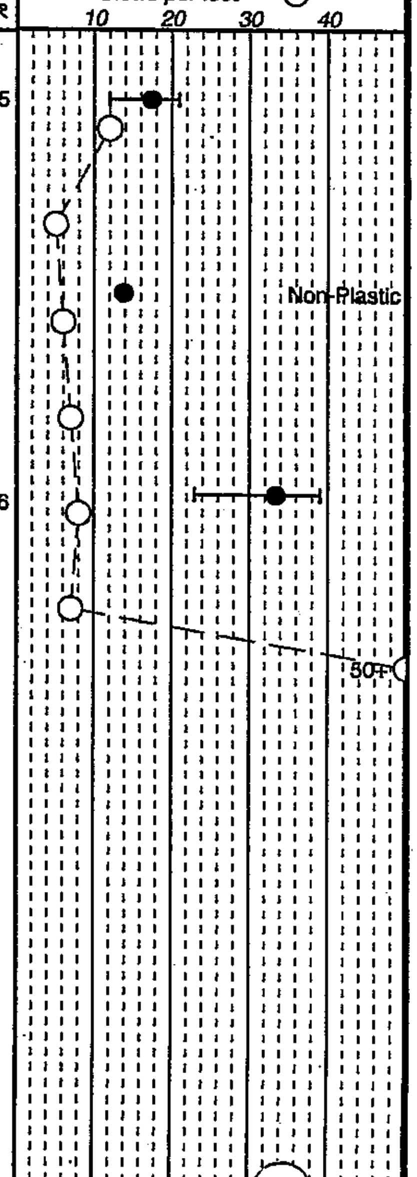
Client: Tran[illegible]ems, Inc. | Project: SCI-823-0.00 | Job No. 012[illegible]070.03

LOG OF: Boring B-27 | Location: Sta. 38+04.1, 39.7 ft. RT of Rel. Shumway Hollow CL | Date Drilled: 01/29/07 to 01/30/07

WATER OBSERVATIONS: Water seepage at: 8.5'-11.6', 13.0'-15.5'
Water level at completion: 14.4' (prior to coring)
35.8' (inside hollowstem augers, includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	GRADATION % Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	656.8												
0.4	656.4						Topsoil - 5"						
		1 5 7	18	1		1.75	Stiff brown SANDY SILT (A-4a), some clay, trace gravel; moist.	1	4	–	33	36	25
3.0	653.8	4 2 3	18	2			Loose brown COARSE AND FINE SAND (A-3a), trace to little clay, trace silt; damp.						
5													
		5 3 3	18	3				0	7	–	80	13	
		3 3 4	18	4			@ 8.5'-10.0', wet.						
10													
11.6	645.2	3 5 3	18	5A 5B		0.75	Medium stiff brown SILTY CLAY (A-6b), trace fine to coarse sand; moist.	0	1	–	1	52	46
13.0	643.8	5 4 3	18	6			Loose brown COARSE AND FINE SAND (A-3a), trace clay; wet.						
15													
16.5	640.3	50/3	3	7			@ 16.0', contains rock fragments.						
		Core 120"	Rec 119"	RQD 80%	R1		Hard gray SANDSTONE; medium grained, unweathered to slightly weathered, micaceous, thinly bedded, slightly fractured. @ 16.5'-18.5', broken zone. @ 17.1'-17.5', lost recovery. @ 21.5'-22.0', qu = 13,148 psi, Er = 2,674,792 psi.						
20													
25													
30													

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ├──┤ LL
Blows per foot - ○



FILE: 0121-3070-03 [9/13/2007 4:26 PM]

DLZ OHIO INC. * 6121 HUNTLEY ROAD, C UMBUS, OHIO 43229 * (614)888-0040

Client: Tran ems, Inc. | Project: SCI-823-0.00 | Job No. 0121-070.03

LOG OF: Boring B-27 | Location: Sta. 38+04.1, 39.7 ft. RT of Rel. Shumway Hollow CL | Date Drilled: 01/29/07 to 01/30/07

WATER OBSERVATIONS: Water seepage at: 8.5'-11.6', 13.0'-15.5'
Water level at completion: 14.4' (prior to coring)
35.8' (inside hollowstem augers, includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ├──┤ LL Blows per foot - ○ 10 20 30 40
30	626.8	Core 120"	Rec 120"	RQD 100%	R2		Hard gray SANDSTONE; medium grained, unweathered to slightly weathered, micaceous, thinly bedded, slightly fractured. @ 32.0'-32.5', qu = 12,949 psi. @ 33.7', calcareous.							
35														
36.5	620.3						Bottom of Boring - 36.5'							
40														
45														
50														
55														
60														

DLZ OHIO INC. * 6121 HUNTLEY ROAD, C[illegible]UMBUS, OHIO 43229 * (614)888-0040

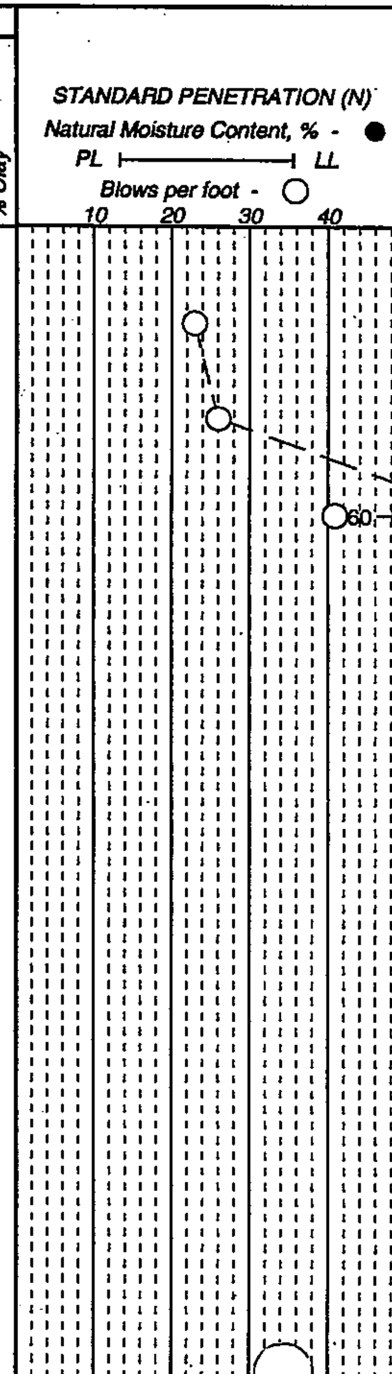
Client: TranS[illegible]ems, Inc. | Project: SCI-823-0.00 | Job No. 0121-[illegible]070.03

LOG OF: Boring TR-27 | Location: Sta. 35+91.3, 5.9 ft. LT of Rel. Shumway Hollow CL | Date Drilled: 8/25/04

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: None (boring collapsed @ 6.0')

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetro-meter (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	646.3												
0.4	645.9						Topsoil - 5"						
		7 10 13	18	1		4.5+	Hard brown SANDY SILT (A-4a), trace clay, trace to little gravel; damp.						
		8 13 13	18	2		4.5+							
		4 10 50	16	3		4.5+	@ 6.0'-7.5', contains sandstone fragments.						
7.5	638.8	Core 120"	Rec 120"	RQD 98%	R-1		Medium hard to hard brown and gray SANDSTONE; very fine to fine grained, slightly to highly weathered, argillaceous, micaceous, massive, slightly fractured. @ 7.5'-10.0', rust stained. @ 7.8',8.9', 15.6', low angle fractures. @ 14.9'-15.2', high angle fracture.						
17.5	628.8						Bottom of Boring - 17.5'						

Depth scale: 0, 5, 7.5, 10, 15, 17.5, 20, 25, 30



FILE: 0121-3070-03 [9/13/2007 4:26 PM]

DLZ OHIO INC. * 6121 HUNTLEY ROAD, C[illegible]MBUS, OHIO 43229 * (614)888-0040

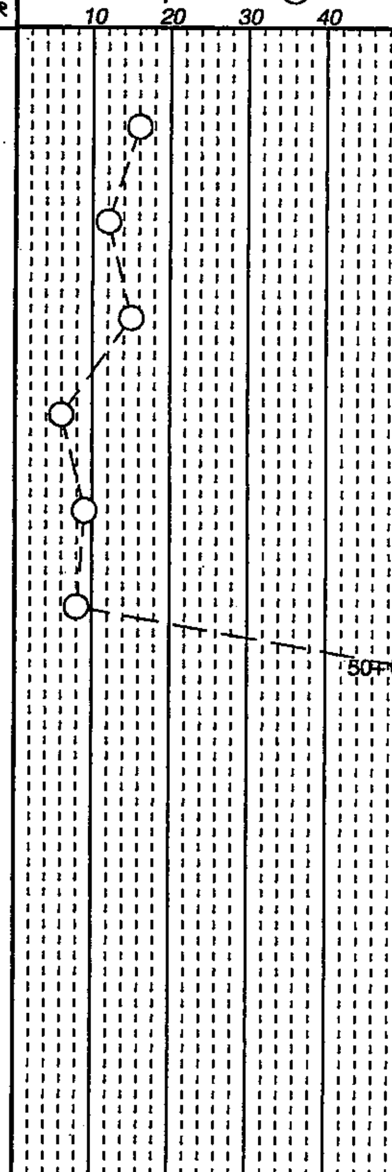
Client: Tran[illegible]ms, Inc. | Project: SCI-823-0.00[illegible] | Job No. 0121[illegible]70.03

LOG OF: Boring TR-28 | Location: Sta. 38+20.7, 17.8 ft. RT of Rel. Shumway Hollow CL | Date Drilled: 02/02/05

WATER OBSERVATIONS: Water seepage at: 14.0', 18.5'
Water level at completion: 10.0' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	659.7												
0.7	659.0						Asphalt Concrete Pavement - 8"						
		4 8 8	16	1		4.0	Very stiff to hard brown SILT AND CLAY (A-6a), trace fine to coarse sand; damp.						
3.0	656.7	5 5 7	15	2			Medium dense reddish brown COARSE AND FINE SAND (A-3a); moist. (residual soil)						
		8 8 7	18	3									
		6 4 2	18	4									
		3 5 4	14	5									
		1 4 4	13	6									
15.5	644.2	50/2	2	7			Severely weathered gray SANDSTONE argillaceous.						
18.5	641.2	Core 60"	Rec 30"	RQD 12%	R-1		Medium hard to hard gray SANDSTONE; very fine to fine grained, moderately to highly weathered, argillaceous, massively bedded, slightly fractured. @ 18.5'-24.0', broken.						
		Core 84"	Rec 84"	RQD 100%	R-2								
30													

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ├──┤ LL
Blows per foot - ○



DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

Client: Tran[illegible]tems, Inc.

Project: SCI-823-0.0[illegible]

Job No. 012[illegible]070.03

LOG OF: Boring TR-28

Location: Sta. 38+20.7, 17.8 ft. RT of Rel. Shumway Hollow CL   Date Drilled: 02/02/05

WATER OBSERVATIONS: Water seepage at: 14.0', 18.5'
Water level at completion: 10.0' (includes drilling water)

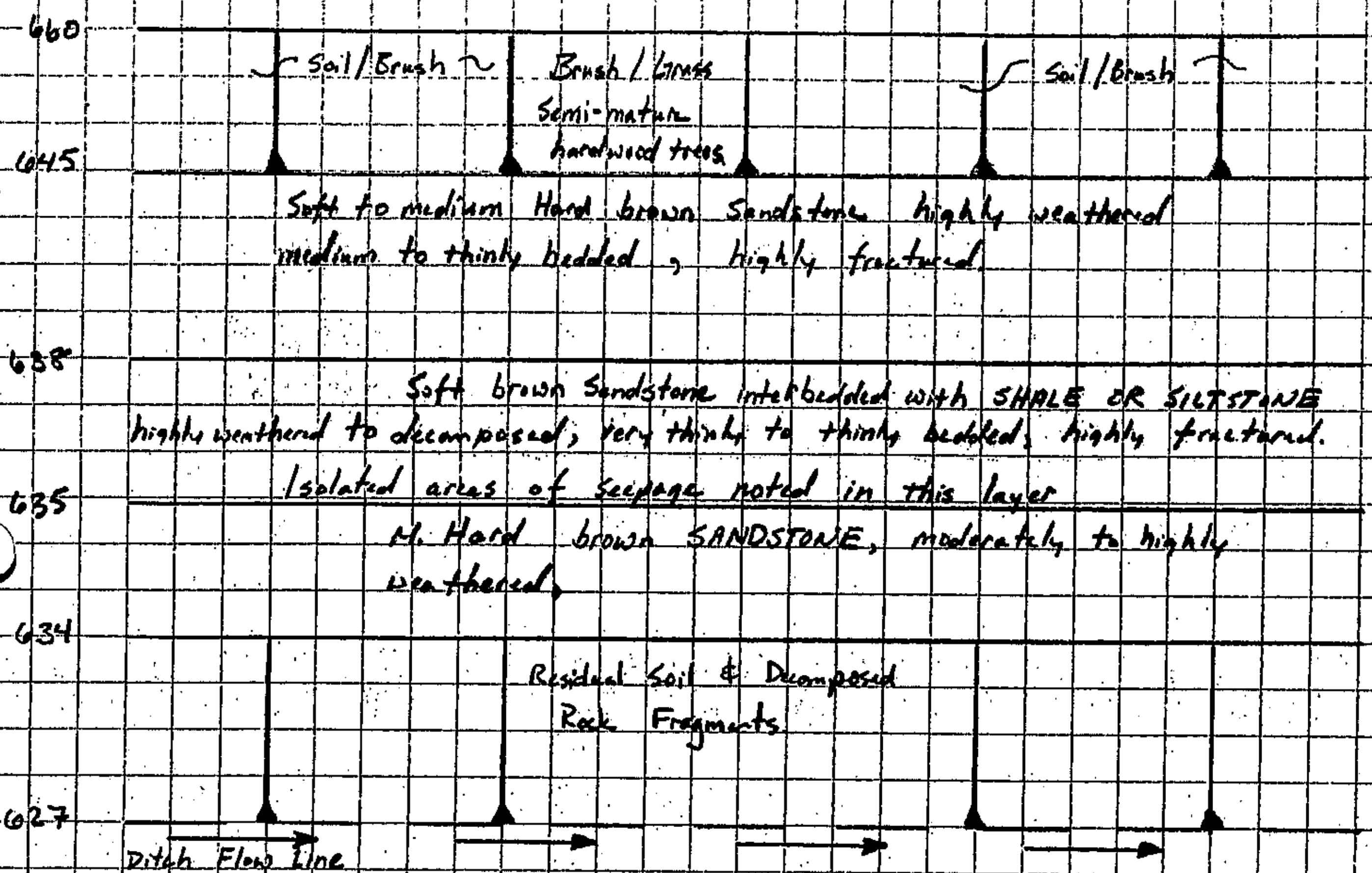
Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	GRADATION % Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ├─┤ LL Blows per foot - ○ 10 20 30 40
30	629.7													
30.5	629.2						Bottom of Boring - 30.5'							
35														
40														
45														
50														
55														
60														

DLZ ENGINEERS • ARCHITECTS • SCIENTISTS PLANNERS • SURVEYORS

CLIENT: Transystems Corp / ODOT D-9
PROJECT: SCI-823 Portsmouth Bypass
SUBJECT: Shumway Hollow Rd over CSX RR
Log of Railroad Rock Cut

PROJECT NO. 0121-3070.03
SHEET NO. 1 OF 1
COMP. BY SJK DATE 11-14-06
CHECKED BY ______ DATE ______

Log of Railroad Rock Cut: East Cut - Forward Abutment Location
At structure location rock obscured by soil and rock fragments. Slope located approximately 50' South of proposed structure centerline was logged.



Vertical fractures were noted in the exposed rock. No significant lateral movement is evident from visual inspection of these fractures.

Isolated areas of seepage are evident in rock layers between approximately elevations 635 and 638.

* Elevation view of Eastern slope. View facing East.

DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

Client: TranS[illegible]ms, Inc. | **Project:** SCI-823-0.00 | **Job No.** 0121[illegible]0.03

LOG OF: Boring RR Cut | **Location:** Shumway over CSXT RR-Forward Abutment Location **Date Drilled:** 9/15/06

WATER OBSERVATIONS: Water seepage at:
Water level at completion:

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N)
0	660.0						Soil / Brush; contains semi-mature hardwood trees.							
15.0	645.0						Soft to medium hard brown SANDSTONE; highly weathered medium to thinly bedded, highly fractured. Vertical fractures were noted in the exposed rock. No significant lateral movement is evident from visual inspection of these fractures.							
22.0	638.0						Soft brown SANDSTONE; interbedded with shale or siltstone, highly weathered to decomposed, very thin to thinly bedded, highly fractured Isolated Seepage from elevation 635 to 638							
25.0	635.0						Medium hard brown SANDSTONE; moderately to highly weathered.							
26.0	634.0						Residual soil and highly weathered rock fragments.							
30														

STANDARD PENETRATION (N): Natural Moisture Content, % - ●; PL ├──┤ LL; Blows per foot - ○; scale 10, 20, 30, 40

Depth scale (ft): 0, 5, 10, 15.0, 20, 22.0, 25.0, 26.0, 30

DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

Client: TranS[illegible]ns, Inc. | Project: SCI-823-0.00 | Job No. 0121[illegible].03

LOG OF: Boring RR Cut | Location: Shumway over CSXT RR-Forward Abutment Location | Date Drilled: 9/15/06

WATER OBSERVATIONS: Water seepage at:
Water level at completion:

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ⊢—⊣ LL Blows per foot - ○ 10 20 30 40
30	630.0													
33.0	627.0						@ 33.0' (elev 627) Ditch Flow Line Bottom of Boring - 33.0'							
35														
40														
45														
50														
55														
60														

APPENDIX III

Laboratory Test Results

Unconfined Compression of Rock Core Specimens

(ASTM D-2938)

DLZ Project No.: 0121-3070.03 **Client: TranSystems**

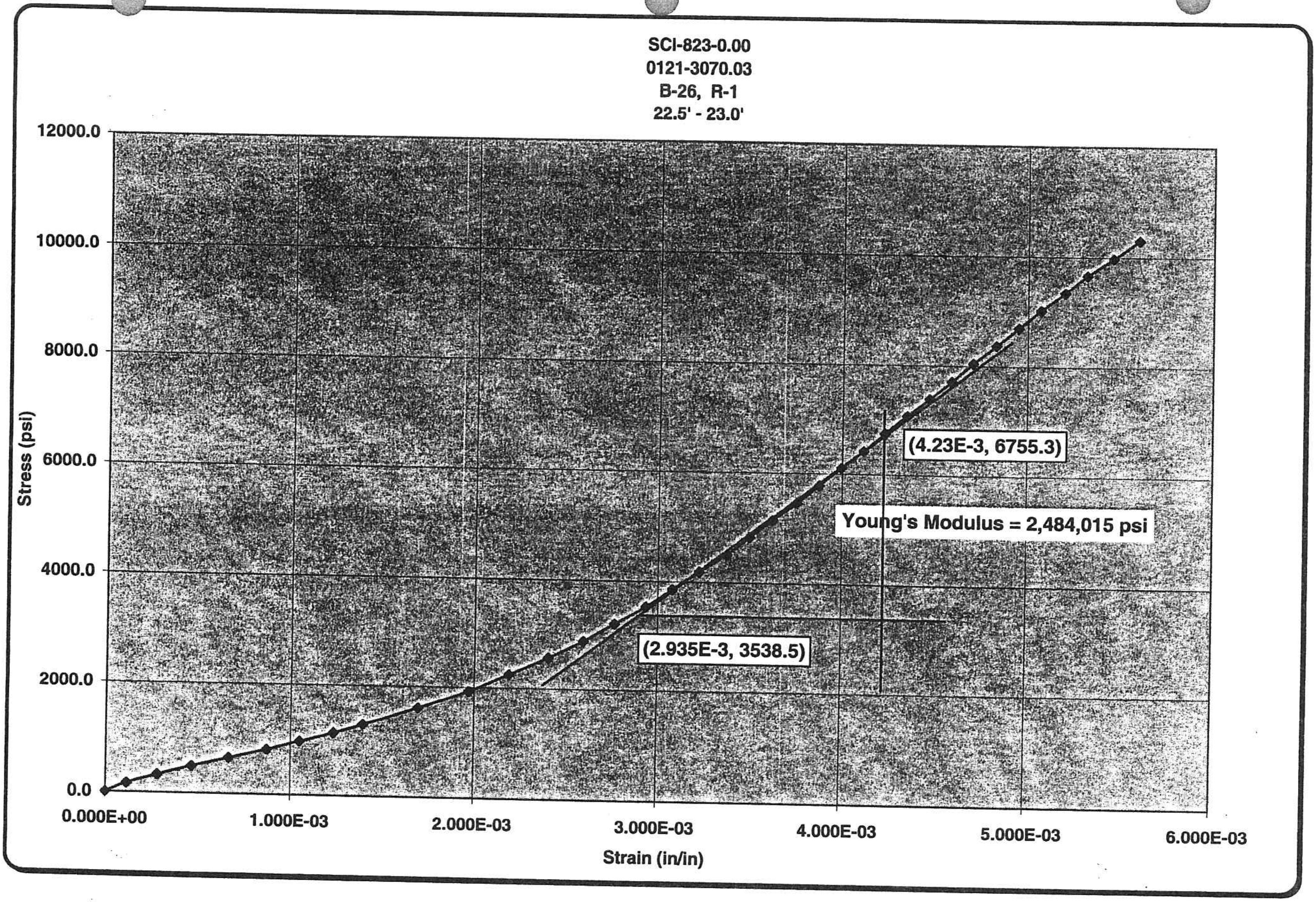
Project Name: SCI-823-0.00 **Date: 2/2/07**

Boring	Run	Depth (ft.)	D_1	D_2	D_3	$D_{(ave)}$	L_1	L_2	L_3	$L_{(ave)}$	L/D	Volume (ft^3)	Mass (Gram)	Unit Wt.(pcf)	Load (lbs)	Strength (psi)
B-24	1	10.0'-10.5'	1.985	1.986	1.988	1.987	4.522	4.522	4.521	4.522	2.276	0.0081103	580.30	157.7	30,860	9,952
			1.987	1.989	1.987											
B-25	1	10.5'-11.0'	1.981	1.980	1.981	1.980	4.469	4.471	4.472	4.471	2.258	0.0079624	562.65	155.8	31,700	10,295
			1.981	1.980	1.977											

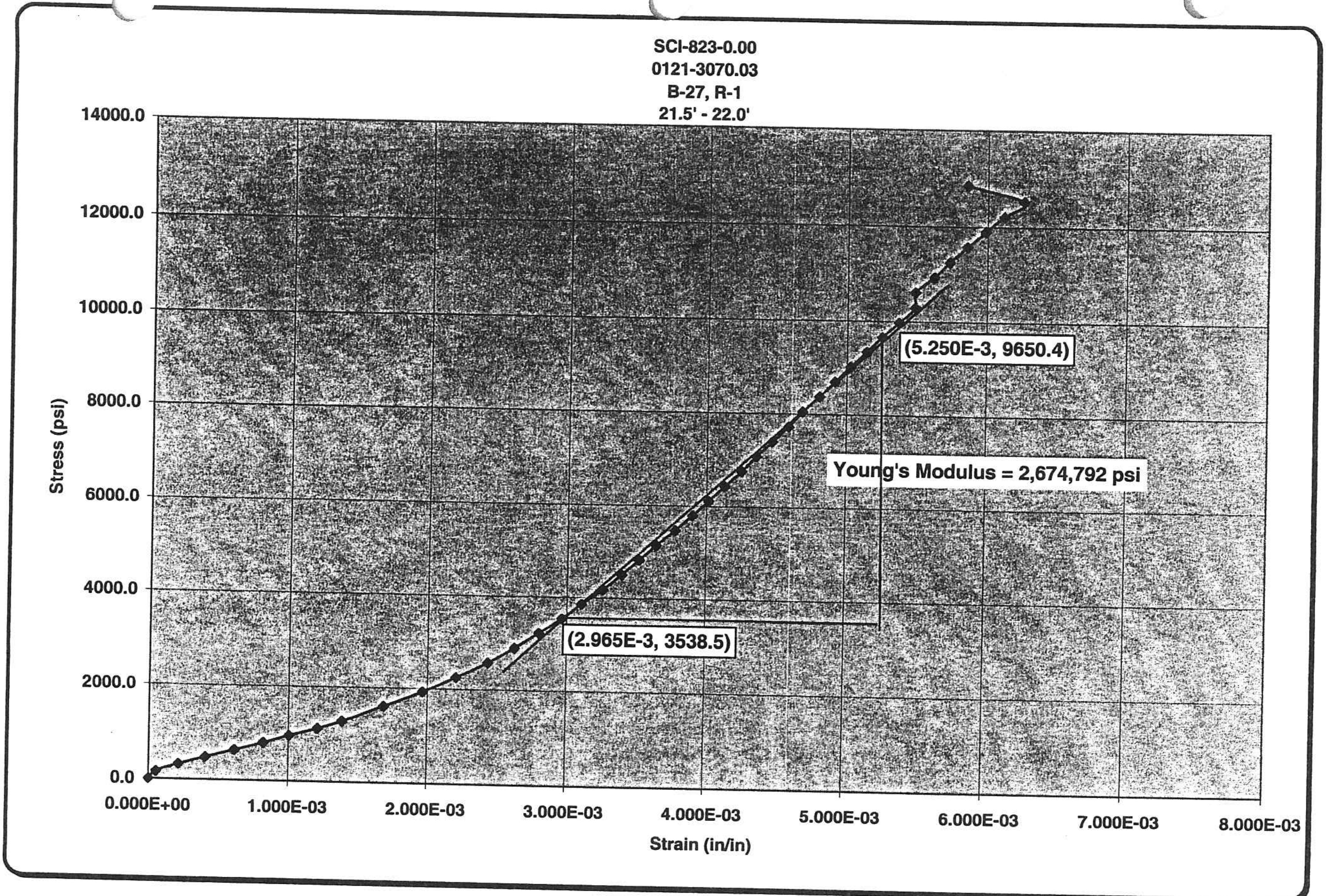
DLZ

Engineers * Architects * Scientists

6121 Huntley Road * Columbus, Ohio * 43229-1003 * Phone: (614) 888-0576 * Fax (614) 888-6415

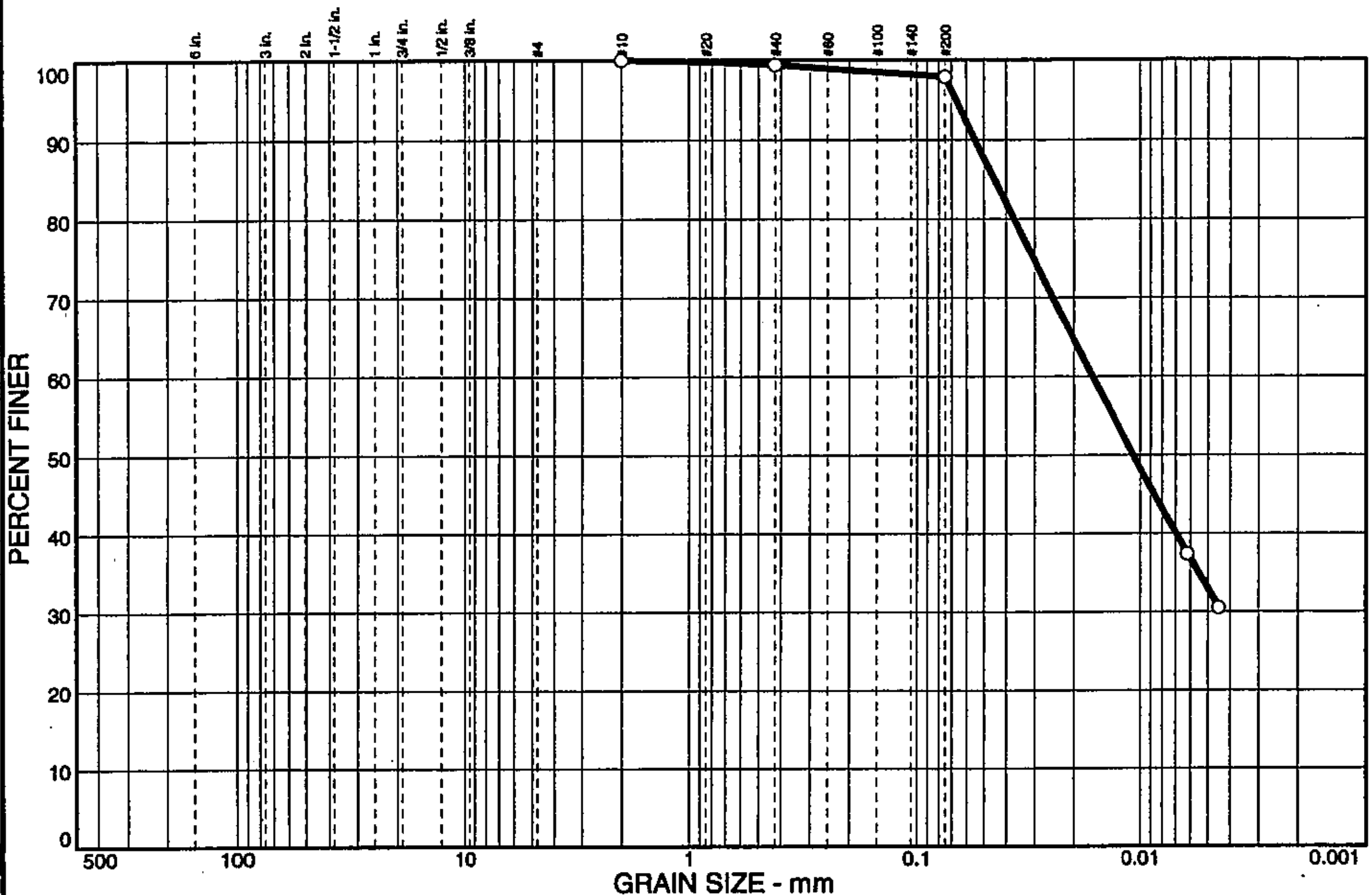


01-307003 B-26 R1 Chart 1



01-307003 B27 R-1 Chart 1

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.0	0.0	0.6	1.5	65.1	32.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#40	99.4		
#200	97.9		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 21 LL= 32 PI= 11

Coefficients

D_{85}= 0.0337 D_{60}= 0.0140 D_{50}= 0.0100
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(11)

Remarks

Moisture Content= 30.6%

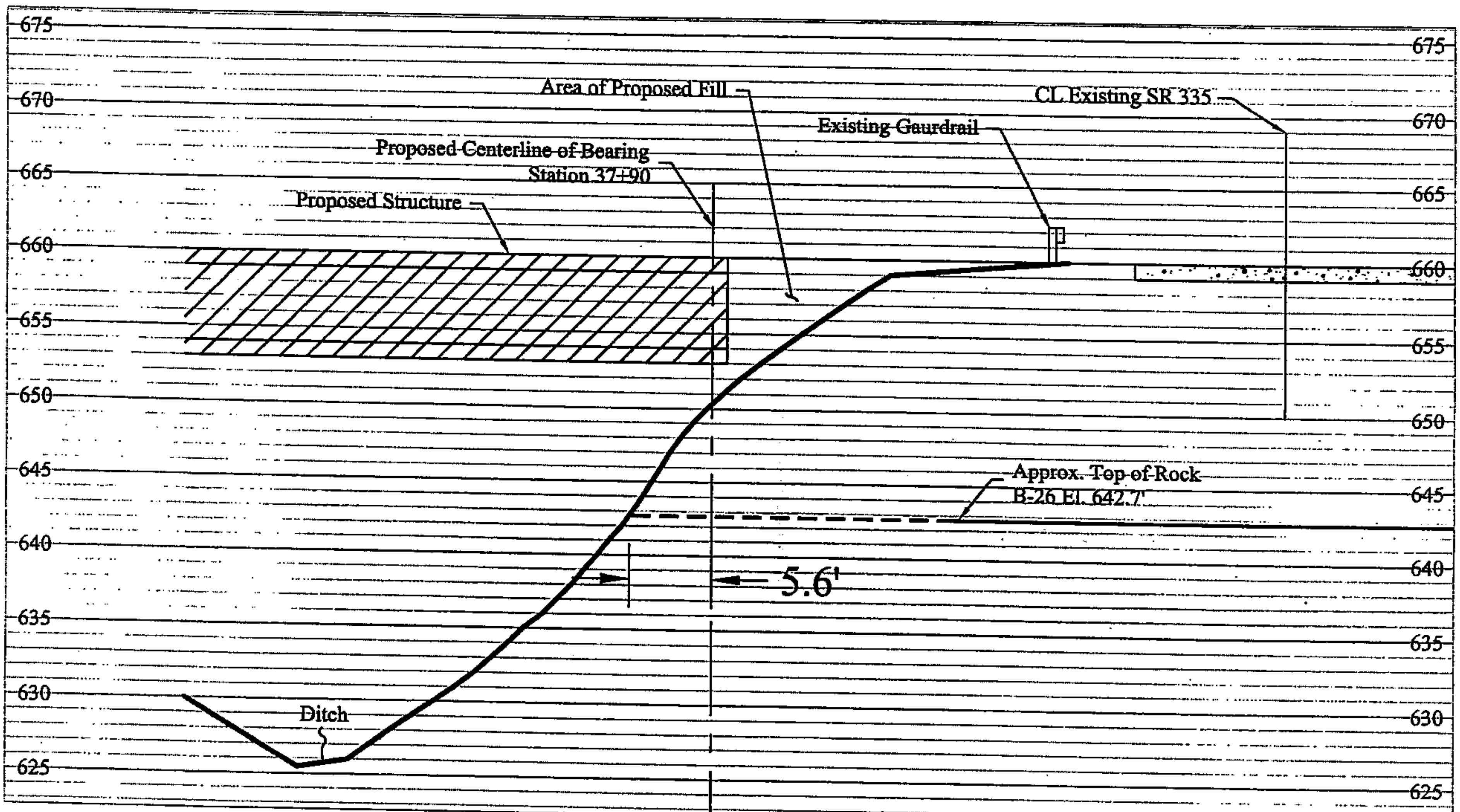
Sample No.: 1 **Source of Sample:** B-24 **Date:** 2/12/07
Location: **Elev./Depth:** 1.0

DLZ

Client: TranSystems, Inc.
Project: SCI-823-0.00
Project No: 0121-3070.03 **Figure**

APPENDIX IV
Forward Abutment Profile Drawings
MSE Wall Calculations
Forward Abutment Calculations
Sample LPILE Output File

PROFILE (VIEW LOOKING NORTH)
RELOCATED SHUMWAY HOLLOW ROAD
25' LEFT OFFSET



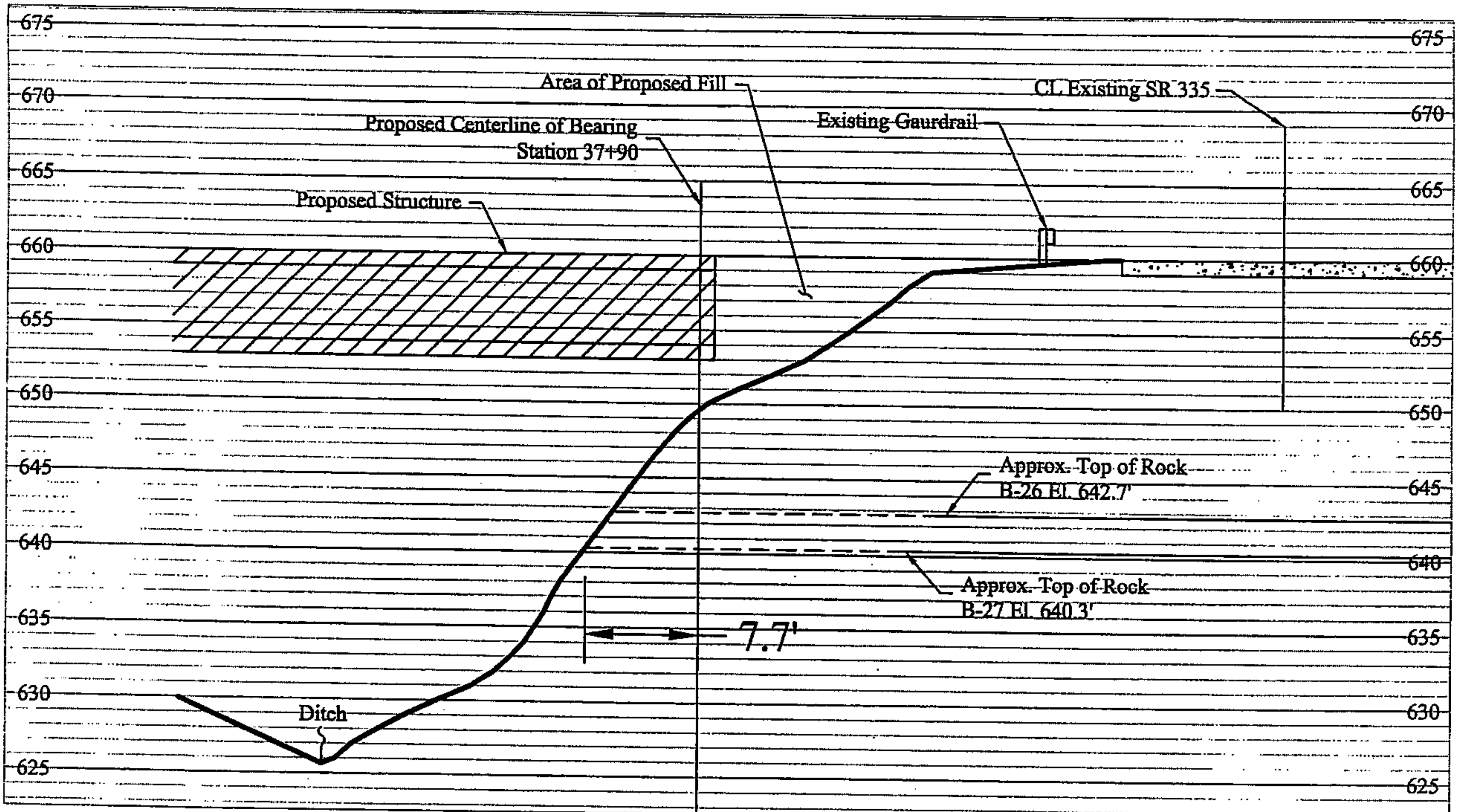
Sheet 1 of 11

RELOCATED SHUMWAY HOLLOW ROAD OVER CSX RR FORWARD ABUTMENT LOCATION 25' LEFT OFFSET	
PROFILE (VIEW LOOKING NORTH) SCI-823-0.00 PORTSMOUTH BYPASS	
PROJECT NO. 0121-3070.03	CALC: SJR · D. 04/04/07

SCALE: 1"=10'

proj\0121\3070.03\Stability An... SE Walls\06 Shumway Hollow over CSX RR\Interim\SCI-823 335 Retaining Wall Topo.dwg, 5/10/2007 10:57:07 AM

PROFILE (VIEW L KING NORTH)
RELOCATED SHUMWA HOLLOW ROAD
ON BASELINE

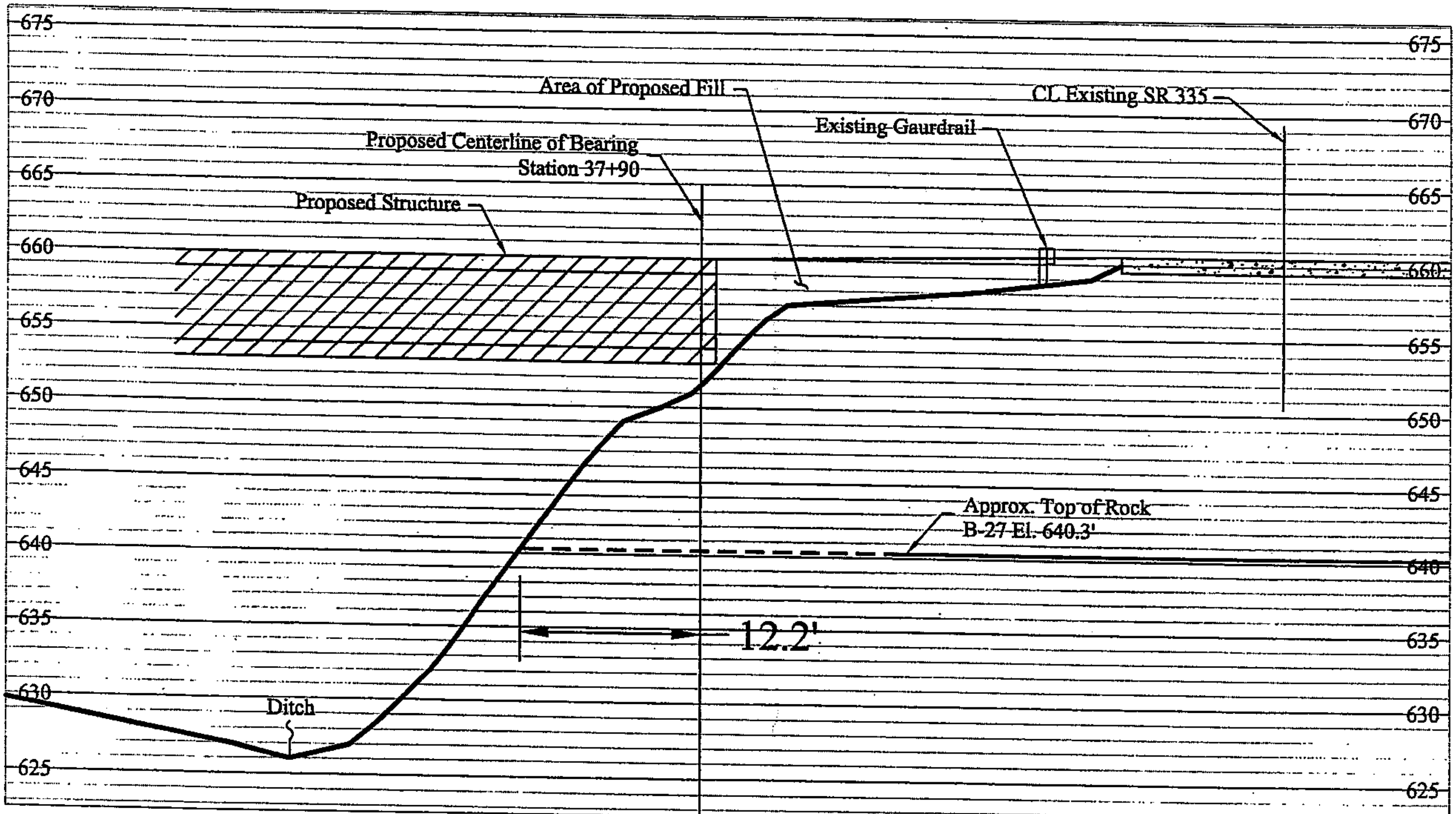


Sheet 2 of 11

RELOCATED SHUMWAY HOLLOW ROAD OVER CSX RR FORWARD ABUTMENT LOCATION ON BASELINE		
PROFILE (VIEW LOOKING NORTH) SCI-823-0.00 PORTSMOUTH BYPASS		
PROJECT NO. 0121-3070.03	CALC: SJR	DATE 04/04/07

I:\proj\0121\3070.03\Stability Analyses\MSE Walls\06 Shumway Hollow over CSX RR\Interim\SCI-823 335 Retaining Wall Topo.dwg, 5/10/2007 10:57:21 AM, \\Dbtree\Q_geotechhp\5100m SCALE: 1"=10'

PROFILE (VIEW LOOKING NORTH)
RELOCATED SHUMWAY HOLLOW ROAD
25' RIGHT OFFSET



RELOCATED SHUMWAY HOLLOW ROAD OVER CSX RR FORWARD ABUTMENT LOCATION 25' RIGHT OFFSET	
PROFILE (VIEW LOOKING NORTH) SCI-823-0.00 PORTSMOUTH BYP	
PROJECT NO. 0121-3070.03	CALC: SJR DATE 04/04/07

SCALE: 1"=10'

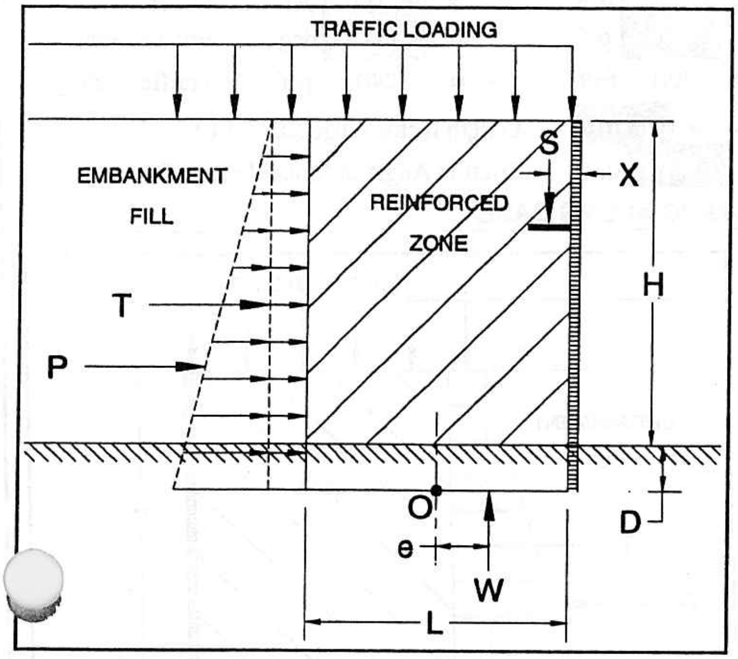
5/ 3 of 11

:\proj\0121\3070.03\Stability ... SE Walls\06 Shumway Hollow over CSX RR\interim\SCI-823 335 Retaining Wall Topo.dwg, 5/10/2007 10:57:33 AM

Assumes 9' Wide Footing at qa=4 ksf

BEARING CAPACITY OF A MSE WALL *(Using Spread footings to Support Abutments)*

Ref: {AASHTO; STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 17th Edition, 2002}



Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	0	psf	Cohesion	Foundation soil
ϕ	=	34	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	34	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
L=B	=	30.1	ft	Length of MSE reinforcement
L factor	=	0.7		Length factor-range (0.7 - 1.0)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	43	ft	
H	=	40	ft	Height of wall
Ka	=	0.33		

Force Moment Arms

Γ Wt	=	21.5	ft	Γ Pa	=	14.3	ft
				Γ S	=	7.6	ft

B'	=	21.34	ft	
γ'	=	57.6	pcf	
W_t		7,224	lb/ft of wall	Weight from traffic
W_{mse}	=	155,316	lb/ft of wall	Weight from MSE wall
S	=	36,000	lb/ft of wall	Force from structure
X	=	7.5	ft	Distance from wall face

3' Setback from MSE wall, 9' wide footing
X = 3' + (9'/2) = 7.5'

Effective Bearing Pressure

$$\sigma_v = \frac{W_t + W_{MSE}}{L - 2e}$$

σ_v = 9,304 psf

Ultimate undrained bearing capacity, q_{ult}

$$q_{ULT} = cN_c + \sigma'_D N_q + \frac{1}{2}\gamma' B N_\gamma$$

q_{ULT} = 30,322 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 12,129 psf

Factor of Safety = **3.26** **OK**

Ultimate drained bearing capacity, q_{ult}

$$q_{ULT} = c'N_c + \sigma'_D N_q + \frac{1}{2}\gamma' B N_\gamma$$

q_{ULT} = 30,322 psf

$$= \frac{q_{ULT}}{FS}$$

q_{ALL} = 12,129 psf

Factor of Safety = **3.26** **OK**

Bearing Capacity Factors for Equations

Undrained		Drained	
N_c	42.16	N_c	42.16
N_q	29.44	N_q	29.44
N_γ	41.06	N_γ	41.06

Eccentricity of Resultant Force

e = 4.38 ft

Kern

e < L/6 = 5.02 ft

DLZ

SUBJECT			
Client	TranSystems Corp	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	5 OF 11
Item	MSE Wall Stability	COMP. BY	SJR DATE 5-10-07
Rear Abutment, Based upon boring B-33		CHECKED BY	DAA DATE 9-12-07
Based on Compacted Granular Fill foundation			

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=40'
2 Sssume bridge is supported on spread footings
3 Ground water; Dw=0.0'
4 Traffic loading is neglected in resisting forces
5 Neglect spread footing load (acts as resisting force)

Wall Properties

$H+D$ = 43 feet
γ_{mse} = 120 pcf
L = 30.1 feet
L factor = 0.70 Length factor-range (0.7 - 1.0)
ϕ = 30 deg Friction Angle of Embankment Fill

Foundational Soil Properties

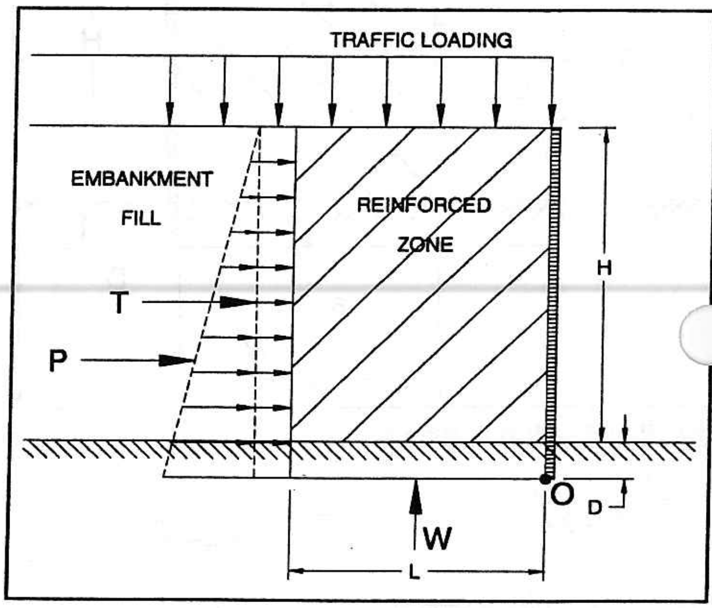
c = 0 psf Cohesion
ϕ' = 34 deg Friction angle
ω_T = 240 psf Traffic loading

RESISTANCE AGAINST SLIDING ALONG BASE

Thrust: $$P_a = K_a\left[\frac{1}{2}\gamma H^2 + \omega_T H\right]$$

where; $K_a = \tan^2(45 - \frac{\phi}{2})$ K_a = 0.33

P_a = 40,016 lbs per foot of wall



Resistance: $P_r = W(\mu)$ (Drained)

where; $\mu = \left(\frac{2}{3}\right)\tan(\phi)$ μ = 0.45

P_r = 69,892 lbs per foot of wall

USE THIS VALUE

$$P_r = L(c)$$ (Undrained)

P_r = 0 lbs per foot of wall

Use Drained Value

$$FS = \frac{P_r}{P_a}$$

	Calculated	Required	
	FS = **1.75**	FS = 1.50	Resistance Against Sliding is **OK**

RESISTANCE AGAINST OVERTURNING

* Summation of Moments about point "O" (base of wall).
* Traffic loading is neglected in resisting forces

$\Sigma M_{resisting}$ = 2,337,506 lb-ft

$$\Sigma M_{resisting} = \gamma HL\left(\frac{L}{2}\right)$$

$\Sigma M_{overturning}$ = 597,967 lb-ft

$$\Sigma M_{overturning} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$$

$$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturning}}$$

	Calculated	Required	
	FS = **3.91**	FS = 2.00	Resistance Against Overturning is **OK**

* From Tested Rock Core Samples $q_u = 9{,}952$ psi (lower bound) $= 68.6$ MPa

1) End Bearing For RQD 70-100% $q_u > 5.2$ tsf (0.5 MPa)

$q_{max} = 4.83\,[q_u\,(MPa)]^{0.51}$ { FHWA-IF-99-025 Eq. 11.6
Drilled Shaft Const. Procedures and Design Methods.

$q_{max} = 4.83\,[68.6\,MPa]^{0.51} = 41.7$ MPa $= 6054$ psi $= 871$ ksf

$q_a = \frac{q_{max}}{F.S.} = \frac{871\ ksf}{3.0} = 290$ ksf

* Use $q_a = 80$ ksf for this type of rock at both rear and forward abutments.

2) Side Friction Assumes Smooth Rock Socket

$f_{max} = 0.65\,p_a\left[\frac{q_u}{p_a}\right]^{0.5} \leq 0.65\,p_a\left[\frac{f'_c}{p_a}\right]^{0.5}$

$f_{max} = 0.65\,(14.7\,psi)\left[\frac{9952}{14.7}\right]^{0.5} \leq 0.65\,(14.7\,psi)\left[\frac{4500}{14.7}\right]^{0.5}$

$f_{max} = 249$ psi $\nleq 167$ psi

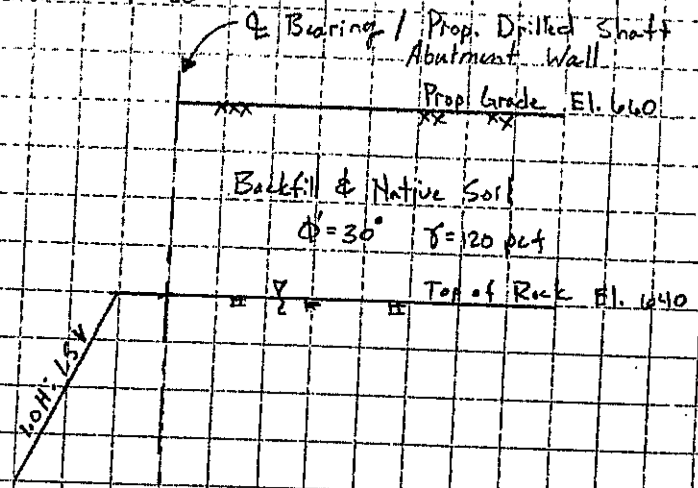
$f_{allow} = \frac{167\ psi}{3.0} = 55.7$ psi $= 8016$ psf

* Use $f_{allow} = 7{,}500$ psf.

DLZ ENGINEERS • ARCHITECTS • SCIENTISTS • PLANNERS • SURVEYORS

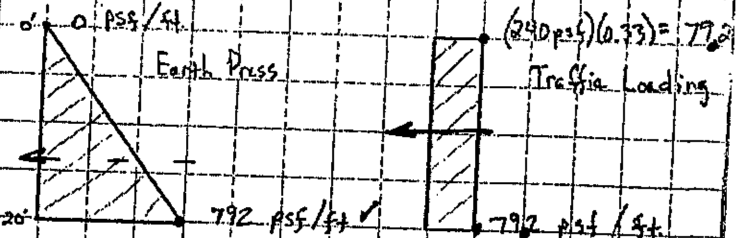
CLIENT: Transystems Corp / ODOT D-9 — PROJECT NO. 0121-3070.03
PROJECT: SCI-823 Portsmouth Bypass — SHEET NO. 7 OF 1_
SUBJECT: Laterally Loaded Drilled Shafts — COMP. BY SJK — DATE _
*Shumway over CSX, Forward Abutment — CHECKED BY TAA — DATE 20-Aug-_

Scale 1" = 20'



Assuming K_a

Lateral Earth Pressure



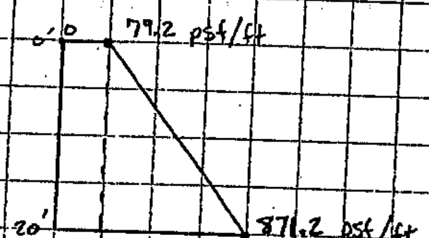
$P_{20} = \frac{1}{2} \cdot K_a \cdot \gamma \cdot H^2$ Where $K_a = \tan^2(45 - \phi/2)$

$K_a = \tan^2(45 - 30/2) = 0.33$

$P_{20} = \frac{1}{2}(0.33)(120\ \text{pcf})(20')^2 + 0.33(240\ \text{psf})(20') = 9{,}504\ \text{lb/ft}$

Lateral Loading from Earth Pressure and Traffic

Combined



* Neglect passive resistance from soil.

* Assume groundwater table at the top of ro[ck]

* Assume 3' Diameter drilled shafts on 2.5 D or 7.5'.

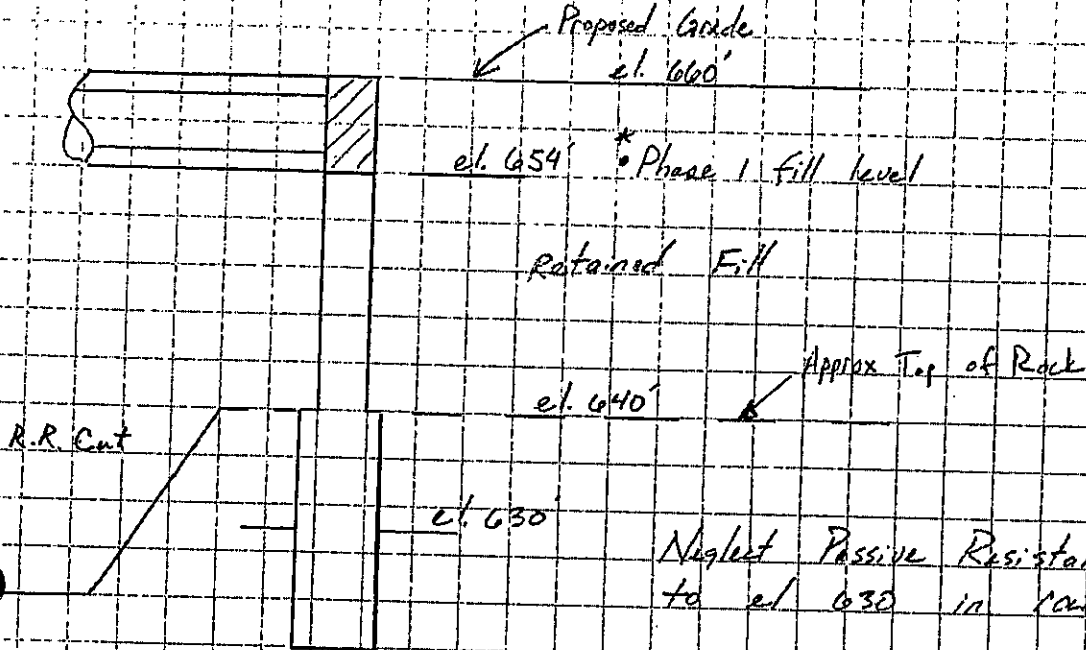
Total Lateral Force (7.5' Spacing) = 71,280 lb.

Assumptions for LPile analysis:

1) Proposed Grade = El 660
2) Approx. top of rock = El. 640 (Boring B-27)
3) Assume Passive wedge cannot resist lateral pressures from El. 640 to 630
4) Assume rock socket development from El. 630 to 615.

DLZ ENGINEERS • ARCHITECTS • SCIENTISTS • PLANNERS • SURVEYORS

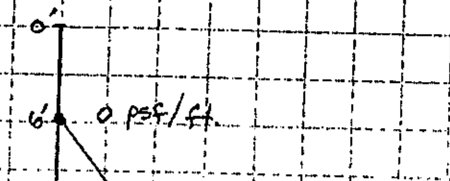
CLIENT Transystem Corp / ODOT D-9	PROJECT NO. 0121-3070.03
PROJECT SCI-823 Portsmouth Bypass	SHEET NO. 8 OF 11
SUBJECT Laterally Loaded Drilled Shafts	COMP. BY SJR DATE 8-14-07
* Shumway over CSX, Forward Abutment	CHECKED BY TAA DATE 20-Aug-200[illegible]



Neglect Passive Resistance from el 640 to el 630 in rock socket.

* Phase 1 fill level. Assume fill to elevation 654' prior to constructing superstructure.

Check deflection filled to el. 654'. Assuming K_a.
• Neglect traffic loading for this case.



$P = \frac{1}{2} K_a \cdot \gamma \cdot H^2$ where $K_a = \tan^2(45 - \frac{\phi}{2})$

$K_a = \tan^2(45 - \frac{30}{2}) = 0.33$

$P = \frac{1}{2}(0.33)(120\ pcf)(20-6)^2 = 3881\ lb.$

$P_{20} = H \cdot \gamma \cdot K_a = (20' - 6)(120\ pcf)(0.33) = 554\ psf/ft.$

DLZ ENGINEERS • ARCHITECTS • SCIENTISTS PLANNERS • SURVEYORS

CLIENT: Tran Systems Corp / ODOT D-9 PROJECT NO. 0121-3070.03
PROJECT: SCI-823 Portsmouth Bypass SHEET NO. 9 OF 11
SUBJECT: Laterally Loaded Drilled Shafts COMP. BY SAK DATE
* Shumway over CSX, Forward Abutment CHECKED BY TAA DATE 20-Aug-20

* Assume At-Rest Earth Pressures

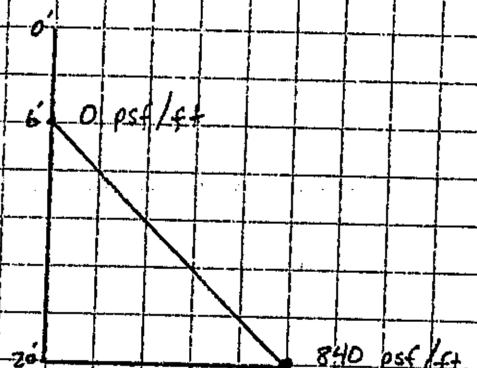
$K_o = (1 - \sin\phi)(1 + \sin\beta)$

1) Assume cohesionless backfill
2) Assume $\phi = \phi' = 30°$
3) $\beta = 0$, horizontal backfill

$K_o = (1 - \sin(30°))(1 + \sin 0) = 0.50$

* Phase 1 H = 14'

Lateral Pressure Distribution.



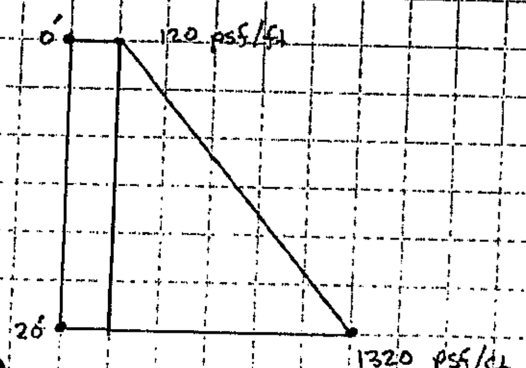
Neglect traffic loading for this case

$p = H \cdot \gamma \cdot K_o$

$p_1 = 14'(120 \text{ pcf})(0.50) = 840 \text{ psf/ft.}$

* Phase 2 H = 20'

Lateral Pressure Distribution



* Include traffic loading, $\omega_T = 240$ psf

$p = H \cdot \gamma \cdot K_o + \omega_T \cdot K_o$

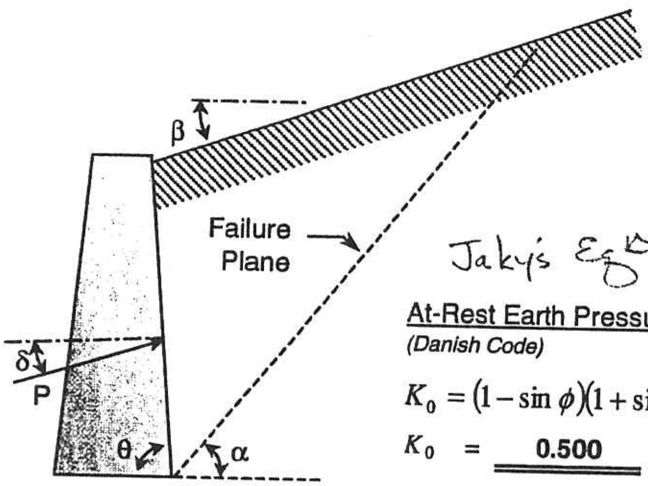
$p_1 = 0(120)(0.50) + 240(0.50) = 120 \text{ psf/ft.}$

$p_2 = 20'(120 \text{ pcf})(0.50) + 240(0.50) = 1320 \text{ psf/ft.}$

EARTH PRESSURE COEFFICIENTS

Ref: EM 1110-2-2502 (1989) Retaining and Floodwalls

with corrections based on Bowles, J.E. (1988) Foundation Analysis and Design, 4th ed.



Parameters

ϕ	=	30	deg.	internal friction angle of soil
δ	=	0	deg.	angle of wall friction
θ	=	90	deg.	angle of wall face from horizontal
β	=	0	deg.	angle of backfill slope from horizontal

Jaky's Eqn

At-Rest Earth Pressure
(Danish Code)

$$K_0 = (1-\sin\phi)(1+\sin\beta)$$

K_0 = **0.500**

Passive Earth Pressure
(Coulomb's Theory, wall friction must be less than ϕ/3)

$$K_p = \frac{\sin^2(\theta-\phi)}{\sin^2\theta\cdot\sin(\theta+\delta)\left[1-\sqrt{\frac{\sin(\phi+\delta)\sin(\phi+\beta)}{\sin(\theta+\delta)\sin(\theta+\beta)}}\right]^2}$$

K_p = **3.000**

Active Earth Pressure
(Coulomb's Theory)

$$K_a = \frac{\sin^2(\theta+\phi)}{\sin^2\theta\cdot\sin(\theta-\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\theta-\delta)\sin(\theta+\beta)}}\right]^2}$$

K_a = **0.333**

Angle between active failure plane and horizontal, α

$$\tan\alpha = \tan\phi + \sqrt{1+\tan^2\phi - \frac{\tan\beta}{\sin\phi\cos\phi}}$$

$\tan\alpha$ = 1.7321

α = **60.0** º

Recommended values for angle of wall friction, δ

- from U.S. Army Corps of Engineers, EM 1110-2-2502 (1989), page 3-37
  Active side, $\delta \le \phi/2$ Resisting side, $\delta = 0$ to $\phi/3$
- from NAVFAC 7.2 (1986) Foundations & Earth Structures, page 7.2-63

Mass concrete on the following foundation materials:	
Clean sound rock	35
Clean gravel, gravel-sand mixtures, coarse sand	29 - 31
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24 - 29
Clean fine sand, silty or clayey fine to medium sand	19 - 24
Fine sandy silt, nonplastic silt	17 - 19
Very stiff and hard residual or preconsolidated clay	22 - 26
Medium stiff and stiff clay and silty clay	17 - 19
(Masonry on foundation materials has same friction factors)	
Steel sheet piles against the following soils:	
Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	22
Clean sand, silty sand-gravel mixture, single size hard rock fill	17
Silty sand, gravel or sand mixed with silt or clay	14
Fine sandy silt, nonplastic silt	11
Formed concrete or concrete sheet piling against the following soils:	
Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	22 - 26
Clean sand, silty sand-gravel mixture, single size hard rock fill	17 - 22
Silty sand, gravel or sand mixed with silt or clay	17
Fine sandy silt, nonplastic silt	14

SCI-823 Porsmouth Bypass
Relocated Shumway Hollow Road over CSXT Railroad Bridge Structure

Forward Abutment Location
Results from LPile analyses | Using non-linear EI | Type III analysis

Assuming Active Condition, K_a=0.33

Diameter of Drilled Shafts (in)	Spacing c-c (in)	Retained Fill (ft)	Reinforcemetn Ratio, ρ (%)	*M_{max} (k-ft)	*V_{max} (k)	Deflection at pile head, δ (in)
36	36	14	5	178	60	0.149
36	36	20	5	502	187	0.809
48	48	14	5	237	63	0.065
48	48	20	5	669	177	0.221
48	72	14	5	355	94	0.098
48	72	20	5	1003	269	0.452
48	96	14	5	473	125	0.131
48	96	20	5	1338	377	0.663

Assuming At-Rest Condition, K_0=0.50

Diameter of Drilled Shafts (in)	Spacing c-c (in)	Retained Fill (ft)	Reinforcemetn Ratio, ρ (%)	*M_{max} (k-ft)	*V_{max} (k)	Deflection at pile head, δ (in)
36	36	14	5	269	91	0.312
36	36	20	5	760	305	1.277
48	48	14	5	359	95	0.099
48	48	20	5	1013	272	0.465
48	60	14	5	449	119	0.124
48	60	20	5	1267	357	0.617
48	72	14	5	538	142	0.149
48	72	20	5	1520	429	0.766
48	96	14	5	718	190	0.230
48	96	20	5	2027	584	1.050

*Maximum moment and shear are unfactored values taken directly from the results of LPile analyses

```
===============================================================================

                 LPILE Plus for Windows, Version 5.0 (5.0.5)

               Analysis of Individual Piles and Drilled Shafts
             Subjected to Lateral Loading Using the p-y Method

                   (c) Copyright ENSOFT, Inc., 1985-2004
                           All Rights Reserved

===============================================================================

   This program is licensed to:

   S Riedy
   DLZ, Ohio Inc.

   Path to file locations:     M:\proj\0121\3070.03\Stability Analyses\MSE Walls\06
   Shumway Hollow over CSX RR\Final\LPile Preliminary\Report\
   Name of input data file:     48 in shaft on 60 in centers 20 ft stage.lpd
   Name of output file:         48 in shaft on 60 in centers 20 ft stage.lpo
   Name of plot output file:    48 in shaft on 60 in centers 20 ft stage.lpp
   Name of runtime file:        48 in shaft on 60 in centers 20 ft stage.lpr

-------------------------------------------------------------------------------
                          Time and Date of Analysis
-------------------------------------------------------------------------------

              Date:  September  5, 2007     Time:  16:29: 8

-------------------------------------------------------------------------------
                                Problem Title
-------------------------------------------------------------------------------

   New LPILE Plus 5.0 Data File

-------------------------------------------------------------------------------
                               Program Options
-------------------------------------------------------------------------------

   Units Used in Computations - US Customary Units, inches, pounds

   Basic Program Options:

   Analysis Type 3:
   - Computation of Nonlinear Bending Stiffness and Ultimate Bending Moment
     Capacity with Pile Response Computed Using Nonlinear EI

   Computation Options:
   - Only internally-generated p-y curves used in analysis
   - Analysis does not use p-y multipliers (individual pile or shaft action only)
   - Analysis assumes no shear resistance at pile tip
   - Analysis includes automatic computation of pile-top deflection vs.
     pile embedment length
   - No computation of foundation stiffness matrix elements
   - Output pile response for full length of pile
   - Analysis assumes no soil movements acting on pile
   - Additional p-y curves computed at specified depths
```

Solution Control Parameters:
- Number of pile increments = 100
- Maximum number of iterations allowed = 200
- Deflection tolerance for convergence = 1.0002E-04 in
- Maximum allowable deflection = 1.0000E+02 in

Printing Options:
- Values of pile-head deflection, bending moment, shear force, and soil reaction are printed for full length of pile.
- Printing Increment (spacing of output points) = 1

Pile Structural Properties and Geometry

Pile Length = 1000.00 in
Depth of ground surface below top of pile = 360.00 in
Slope angle of ground surface = .00 deg.

Structural properties of pile defined using 2 points

Point	Depth X in	Pile Diameter in	Moment of Inertia in**4	Pile Area Sq.in	Modulus of Elasticity lbs/Sq.in
1	0.0000	48.00000000	260576.0000	1810.0000	5000000.
2	1000.0000	48.00000000	260576.0000	1810.0000	5000000.

Please note that because this analysis makes computations of ultimate moment capacity and pile response using nonlinear bending stiffness that the above values of moment of inertia and modulus of are not used for any computations other than total stress due to combined axial loading and bending.

Soil and Rock Layering Information

The soil profile is modelled using 1 layers

Layer 1 is strong rock (vuggy limestone)
Distance from top of pile to top of layer = 360.000 in
Distance from top of pile to bottom of layer = 1000.000 in

(Depth of lowest layer extends .00 in below pile tip)

Effective Unit Weight of Soil vs. Depth

Distribution of effective unit weight of soil with depth is defined using 2 points

Point No.	Depth X in	Eff. Unit Weight lbs/in**3
1	360.00	.08100
2	1000.00	.08100

Shear Strength of Soils

Distribution of shear strength parameters with depth
defined using 2 points

Point No.	Depth X in	Cohesion c lbs/in**2	Angle of Friction Deg.	E50 or k_rm	RQD %
1	360.000	10000.00000	.00	------	------
2	1000.000	10000.00000	.00	------	------

Notes:

(1) Cohesion = uniaxial compressive strength for rock materials.
(2) Values of E50 are reported for clay strata.
(3) Default values will be generated for E50 when input values are 0.
(4) RQD and k_rm are reported only for weak rock strata.

Loading Type

Static loading criteria was used for computation of p-y curves

---

Distributed Lateral Loading

Distributed lateral load intensity defined using 2 points

Point No.	Depth X in	Dist. Load lbs/in
1	.000	50.00000
2	240.000	550.00000

Pile-head Loading and Pile-head Fixity Conditions

Number of loads specified = 1

Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Shear force at pile head = .000 lbs
Bending moment at pile head = .000 in-lbs
Axial load at pile head = .000 lbs

(Zero moment at pile head for this load indicates a free-head condition)

Output of p-y Curves at Specified Depths

---

p-y curves are generated and printed for verification at 2 depths.

Depth No.	Depth Below Pile Head in	Depth Below Ground Surface in
1	380.000	20.000
2	420.000	60.000

Depth of ground surface below top of pile = 360.00 in

Computations of Ultimate Moment Capacity and Nonlinear Bending Stiffness

---

Number of pile sections = 1

Pile Section No. 1

The sectional shape is a circular drilled shaft (bored pile).

Outside Diameter = 48.0000 In

Material Properties:

Compressive Strength of Concrete	=	4.500 Kip/In**2
Yield Stress of Reinforcement	=	60. Kip/In**2
Modulus of Elasticity of Reinforcement	=	29000. Kip/In**2
Number of Reinforcing Bars	=	22
Area of Single Bar	=	4.00000 In**2
Number of Rows of Reinforcing Bars	=	11
Cover Thickness (edge to bar center)	=	2.500 In

Unfactored Axial Squash Load Capacity = 11864.96 Kip

Distribution and Area of Steel Reinforcement

Row Number	Area of Reinforcement In**2	Distance to Centroidal Axis In
1	8.000000	21.2812
2	8.000000	19.5571
3	8.000000	16.2486
4	8.000000	11.6238
5	8.000000	6.0572
6	8.000000	.0000
7	8.000000	-6.0572
8	8.000000	-11.6238
9	8.000000	-16.2486
10	8.000000	-19.5571
11	8.000000	-21.2812

48 in shaft on 60 in centers 20 ft stage.lpo

Axial Thrust Force = .00 lbs

Bending Moment in-lbs	Bending Stiffness lb-in2	Bending Curvature rad/in	Maximum Strain in/in	Neutral Axis Position inches
1593392.	1.593392E+12	.00000100	.00002406	24.06134033
7884053.	1.576811E+12	.00000500	.00012030	24.06060791
7884053.	8.760058E+11	.00000900	.00015868	17.63104248
11249830.	8.653715E+11	.00001300	.00022967	17.66693115
14673859.	8.631682E+11	.00001700	.00030095	17.70318604
18079816.	8.609436E+11	.00002100	.00037254	17.74017334
21467147.	8.586859E+11	.00002500	.00044444	17.77752686
24835428.	8.563941E+11	.00002900	.00051664	17.81524658
28184871.	8.540870E+11	.00003300	.00058918	17.85406494
31514135.	8.517334E+11	.00003700	.00066204	17.89288330
34823813.	8.493613E+11	.00004100	.00073524	17.93280029
38112746.	8.469499E+11	.00004500	.00080879	17.97308350
41380855.	8.445073E+11	.00004900	.00088269	18.01409912
44627675.	8.420316E+11	.00005300	.00095696	18.05584717
47852710.	8.395212E+11	.00005700	.00103160	18.09832764
51055436.	8.369744E+11	.00006100	.00110663	18.14154053
54235301.	8.343892E+11	.00006500	.00118206	18.18548584
57391719.	8.317641E+11	.00006900	.00125788	18.23016357
60524629.	8.291045E+11	.00007300	.00133414	18.27593994
63588474.	8.258243E+11	.00007700	.00141041	18.31695557
66007072.	8.149021E+11	.00008100	.00148130	18.28765869
67892865.	7.987396E+11	.00008500	.00154732	18.20379639
69626546.	7.823207E+11	.00008900	.00161225	18.11517334
71346862.	7.671706E+11	.00009300	.00167742	18.03680420
72509588.	7.475215E+11	.00009700	.00173646	17.90167236
73643705.	7.291456E+11	.00010100	.00179546	17.77679443
80056894.	6.111213E+11	.00013100	.00221564	16.91326904
83902810.	5.211355E+11	.00016100	.00261756	16.25811768
85502495.	4.476570E+11	.00019100	.00302486	15.83697510
86590492.	3.918122E+11	.00022100	.00345708	15.64288330
87450049.	3.484066E+11	.00025100	.00388711	15.48651123

Unfactored (Nominal) Moment Capacity at Concrete Strain of 0.003 = 85404.84888
In-Kip

**** WARNING ****

An unreasonable input value for uniaxial compressive strength has been specified for a soil defined using the vuggy limestone criteria. The input value is greater than 2000 psi. You should check your input data for correctness.

p-y Curve for Vuggy Limestone (Strong Rock) Criteria

Soil Layer Number	=	1
Depth below pile head	=	380.000 in
Depth below Ground Surface	=	20.000 in
Shaft Diameter	=	48.000 in
Uniaxial Compressive Strength	=	10000.000 lbs/in**2
-multiplier	=	1.00000
multiplier	=	1.00000

y, in	p, lbs/in
.00000	.00
.01920	192000.00
.02560	195200.00
.03200	198400.00
.03840	201600.00
.04480	204800.00
.05120	208000.00
.05760	211200.00
.06400	214400.00
.07040	217600.00
.07680	220800.00
.08320	224000.00
.08960	227200.00
.09600	230400.00
.10240	233600.00
.10880	236800.00
.11520	240000.00

p-y Curve for Vuggy Limestone (Strong Rock) Criteria

Soil Layer Number = 1
Depth below pile head = 420.000 in
Depth below Ground Surface = 60.000 in
Shaft Diameter = 48.000 in
Uniaxial Compressive Strength = 10000.000 lbs/in**2
p-multiplier = 1.00000
y-multiplier = 1.00000

y, in	p, lbs/in
.00000	.00
.01920	192000.00
.02560	195200.00
.03200	198400.00
.03840	201600.00
.04480	204800.00
.05120	208000.00
.05760	211200.00
.06400	214400.00
.07040	217600.00
.07680	220800.00
.08320	224000.00
.08960	227200.00
.09600	230400.00
.10240	233600.00
.10880	236800.00
.11520	240000.00

Computed Values of Load Distribution and Deflection
for Lateral Loading for Load Case Number 1

Pile-head boundary conditions are Shear and Moment (BC Type 1)
Specified shear force at pile head = .000 lbs
Specified moment at pile head = .000 in-lbs

48 in shaft on 60 in centers 20 ft stage.lpo

Specified axial load at pile head = .000 lbs

(Zero moment for this load indicates free-head conditions)

Depth x in	Deflect. y in	Moment M lbs-in	Shear V lbs	Slope S Rad.	Total Stress lbs/in**2	Flx. Rig. EI lbs-in**2	Soil Res p lbs/in
0.000	.617530	-7.08E-06	-8.85E-08	-.002011	6.52E-10	1.59E+12	0.000
10.000	.597424	2500.000	604.167	-.002011	.230259	1.59E+12	0.000
20.000	.577319	12083.	1416.667	-.002010	1.113	1.59E+12	0.000
30.000	.557214	30833.	2437.500	-.002010	2.840	1.59E+12	0.000
40.000	.537112	60833.	3666.667	-.002010	5.603	1.59E+12	0.000
50.000	.517013	1.04E+05	5104.167	-.002010	9.594	1.59E+12	0.000
60.000	.496921	1.63E+05	6750.000	-.002009	15.005	1.59E+12	0.000
70.000	.476839	2.39E+05	8604.167	-.002007	22.028	1.59E+12	0.000
80.000	.456772	3.35E+05	10667.	-.002006	30.855	1.59E+12	0.000
90.000	.436726	4.53E+05	12938.	-.002003	41.677	1.59E+12	0.000
100.000	.416709	5.94E+05	15417.	-.002000	54.687	1.59E+12	0.000
110.000	.396728	7.61E+05	18104.	-.001996	70.076	1.59E+12	0.000
120.000	.376796	9.56E+05	21000.	-.001990	88.036	1.59E+12	0.000
130.000	.356923	1.18E+06	24104.	-.001984	108.759	1.59E+12	0.000
140.000	.337125	1.44E+06	27417.	-.001975	132.437	1.59E+12	0.000
150.000	.317417	1.73E+06	30938.	-.001965	159.263	1.59E+12	0.000
160.000	.297817	2.06E+06	34667.	-.001953	189.427	1.59E+12	0.000
170.000	.278347	2.42E+06	38604.	-.001939	223.121	1.59E+12	0.000
180.000	.259030	2.83E+06	42750.	-.001923	260.538	1.58E+12	0.000
190.000	.239891	3.28E+06	47104.	-.001904	301.870	1.58E+12	0.000
200.000	.220959	3.77E+06	51667.	-.001881	347.308	1.58E+12	0.000
210.000	.202266	4.31E+06	56437.	-.001856	397.043	1.58E+12	0.000
220.000	.183845	4.90E+06	61417.	-.001827	451.269	1.58E+12	0.000
230.000	.165735	5.54E+06	66604.	-.001793	510.177	1.58E+12	0.000
240.000	.147976	6.23E+06	72000.	-.001756	573.959	1.58E+12	0.000
250.000	.130611	6.98E+06	74750.	-.001714	642.807	1.58E+12	0.000
260.000	.113689	7.73E+06	74750.	-.001668	711.654	1.58E+12	0.000
270.000	.097257	8.47E+06	74750.	-.001595	780.502	8.74E+11	0.000
280.000	.081795	9.22E+06	74750.	-.001493	849.349	8.71E+11	0.000
290.000	.067393	9.97E+06	74750.	-.001383	918.197	8.69E+11	0.000
300.000	.054137	1.07E+07	74750.	-.001264	987.044	8.67E+11	0.000
310.000	.042119	1.15E+07	74750.	-.001136	1055.892	8.65E+11	0.000
320.000	.031426	1.22E+07	74750.	-.000999	1124.739	8.65E+11	0.000
330.000	.022145	1.30E+07	74750.	-.000853	1193.587	8.64E+11	0.000
340.000	.014363	1.37E+07	74750.	-.000699	1262.434	8.64E+11	0.000
350.000	.008169	1.45E+07	74750.	-.000536	1331.281	8.63E+11	0.000
360.000	.003649	1.52E+07	-1.08E+05	-.000364	1400.129	8.63E+11	-36489.
370.000	.000891	1.23E+07	-3.35E+05	-.000205	1132.899	8.65E+11	-8908.836
380.000	-.000444	8.51E+06	-3.57E+05	-8.48E-05	783.616	8.73E+11	4444.234
390.000	-.000806	5.16E+06	-2.95E+05	-1.98E-05	475.265	1.58E+12	8055.813
400.000	-.000840	2.62E+06	-2.12E+05	4.82E-06	241.112	1.59E+12	8399.446
410.000	-.000709	9.15E+05	-1.35E+05	1.60E-05	84.320	1.59E+12	7091.677
420.000	-.000521	-77678.	-73270.	1.86E-05	7.154	1.59E+12	5209.353
430.000	-.000338	-5.50E+05	-30345.	1.66E-05	50.649	1.59E+12	3375.780
440.000	-.000189	-6.85E+05	-4029.044	1.27E-05	63.051	1.59E+12	1887.327
450.000	-8.29E-05	-6.30E+05	9550.113	8.61E-06	58.071	1.59E+12	828.505
460.000	-1.65E-05	-4.94E+05	14520.	5.08E-06	45.459	1.59E+12	165.375
470.000	1.88E-05	-3.40E+05	14406.	2.47E-06	31.325	1.59E+12	-187.995
480.000	3.28E-05	-2.05E+05	11827.	7.55E-07	18.922	1.59E+12	-327.919
490.000	3.39E-05	-1.04E+05	8492.685	-2.15E-07	9.539	1.59E+12	-338.911
500.000	2.85E-05	-35586.	5373.601	-6.52E-07	3.278	1.59E+12	-284.905
510.000	2.09E-05	3905.012	2906.242	-7.51E-07	.359666	1.59E+12	-208.566
520.000	1.35E-05	22539.	1190.021	-6.68E-07	2.076	1.59E+12	-134.678
530.000	7.49E-06	27705.	141.958	-5.10E-07	2.552	1.59E+12	-74.935
540.000	3.26E-06	25378.	-395.614	-3.44E-07	2.337	1.59E+12	-32.580

550.000	6.15E-07	19793.	-589.269	-2.02E-07	1.823	1.59E+12	-6.151
560.000	-7.85E-07	13593.	-580.752	-9.74E-08	1.252	1.59E+12	7.855
570.000	-1.33E-06	8178.110	-474.828	-2.91E-08	.753234	1.59E+12	13.330
580.000	-1.37E-06	4096.328	-339.814	9.43E-09	.377287	1.59E+12	13.673
590.000	-1.14E-06	1381.830	-214.226	2.66E-08	.127272	1.59E+12	11.445
600.000	-8.35E-07	-188.186	-115.254	3.04E-08	.017333	1.59E+12	8.350
610.000	-5.37E-07	-923.243	-46.643	2.69E-08	.085034	1.59E+12	5.372
620.000	-2.97E-07	-1121.055	-4.908	2.05E-08	.103253	1.59E+12	2.975
630.000	-1.28E-07	-1021.394	16.369	1.37E-08	.094074	1.59E+12	1.281
640.000	-2.27E-08	-793.676	23.909	8.04E-09	.073100	1.59E+12	.227439
650.000	3.28E-08	-543.214	23.408	3.85E-09	.050032	1.59E+12	-.327591
660.000	5.42E-08	-325.511	19.062	1.12E-09	.029981	1.59E+12	-.541705
670.000	5.52E-08	-161.979	13.596	-4.10E-10	.014919	1.59E+12	-.551531
680.000	4.60E-08	-53.600	8.539	-1.09E-09	.004937	1.59E+12	-.459700
690.000	3.34E-08	8.810	4.570	-1.23E-09	.000811	1.59E+12	-.334230
700.000	2.14E-08	37.796	1.827	-1.08E-09	.003481	1.59E+12	-.214290
710.000	1.18E-08	45.353	.165374	-8.20E-10	.004177	1.59E+12	-.118069
720.000	5.03E-09	41.103	-.676529	-5.49E-10	.003786	1.59E+12	-.050312
730.000	8.35E-10	31.822	-.969837	-3.20E-10	.002931	1.59E+12	-.008350
740.000	-1.36E-09	21.706	-.943388	-1.52E-10	.001999	1.59E+12	.013640
750.000	-2.20E-09	12.955	-.765152	-4.30E-11	.001193	1.59E+12	.022007
760.000	-2.22E-09	6.403	-.543894	1.77E-11	.000590	1.59E+12	.022244
770.000	-1.85E-09	2.077	-.340358	4.43E-11	.000191	1.59E+12	.018463
780.000	-1.34E-09	-.403766	-.181154	4.96E-11	3.72E-05	1.59E+12	.013378
790.000	-8.55E-10	-1.546	-.071533	4.35E-11	.000142	1.59E+12	.008546
800.000	-4.69E-10	-1.834	-.005374	3.29E-11	.000169	1.59E+12	.004685
810.000	-1.98E-10	-1.654	.027932	2.19E-11	.000152	1.59E+12	.001976
820.000	-3.04E-11	-1.276	.039330	1.27E-11	.000118	1.59E+12	.000304
830.000	5.67E-11	-.867287	.038015	5.99E-12	7.99E-05	1.59E+12	-.000567
840.000	8.94E-11	-.515493	.030710	1.65E-12	4.75E-05	1.59E+12	-.000894
850.000	8.97E-11	-.253083	.021756	-7.62E-13	2.33E-05	1.59E+12	-.000897
860.000	7.41E-11	-.080381	.013563	-1.81E-12	7.40E-06	1.59E+12	-.000741
870.000	5.35E-11	.018172	.007178	-2.00E-12	1.67E-06	1.59E+12	-.000535
880.000	3.41E-11	.063180	.002797	-1.75E-12	5.82E-06	1.59E+12	-.000341
890.000	1.86E-11	.074107	.000164	-1.32E-12	6.83E-06	1.59E+12	-.000186
900.000	7.74E-12	.066451	-.001152	-8.76E-13	6.12E-06	1.59E+12	-7.74E-05
910.000	1.06E-12	.051060	-.001592	-5.07E-13	4.70E-06	1.59E+12	-1.06E-05
920.000	-2.41E-12	.034610	-.001524	-2.39E-13	3.19E-06	1.59E+12	2.41E-05
930.000	-3.71E-12	.020573	-.001218	-6.55E-14	1.89E-06	1.59E+12	3.71E-05
940.000	-3.72E-12	.010251	-.000846	3.12E-14	9.44E-07	1.59E+12	3.72E-05
950.000	-3.09E-12	.003653	-.000505	7.49E-14	3.36E-07	1.59E+12	3.09E-05
960.000	-2.23E-12	.000144	-.000240	8.68E-14	1.32E-08	1.59E+12	2.23E-05
970.000	-1.35E-12	-.001139	-6.06E-05	8.37E-14	1.05E-07	1.59E+12	1.35E-05
980.000	-5.53E-13	-.001068	3.48E-05	7.67E-14	9.83E-08	1.59E+12	5.53E-06
990.000	1.81E-13	-.000443	5.34E-05	7.20E-14	4.08E-08	1.59E+12	-1.81E-06
1000.	8.87E-13	0.000	0.000	7.06E-14	0.000	1.59E+12	-8.87E-06

Please note that because this analysis makes computations of ultimate moment capacity and pile response using nonlinear bending stiffness that the above values of total stress due to combined axial stress and bending may not be representative of actual conditions.

Output Verification:

Computed forces and moments are within specified convergence limits.

Output Summary for Load Case No. 1:

Pile-head deflection	=	.61752962 in
Computed slope at pile head	=	-.00201054
Maximum bending moment	=	15201667. lbs-in

48 in shaft on 60 in centers 20 ft stage.lpo

```
Maximum shear force                 =   -357007.55169 lbs
Depth of maximum bending moment     =       360.00000 in
Depth of maximum shear force        =       380.00000 in
Number of iterations                =              12
Number of zero deflection points    =               8
```

Summary of Pile-Head Response(s)

Definition of Symbols for Pile-Head Loading Conditions:

```
Type 1 = Shear and Moment,             y = pile-head displacment in
Type 2 = Shear and Slope,              M = pile-head moment lbs-in
Type 3 = Shear and Rot. Stiffness,     V = pile-head shear force lbs
Type 4 = Deflection and Moment,        S = pile-head slope, radians
Type 5 = Deflection and Slope,         R = rotational stiffness of pile-headin-lbs/rad
```

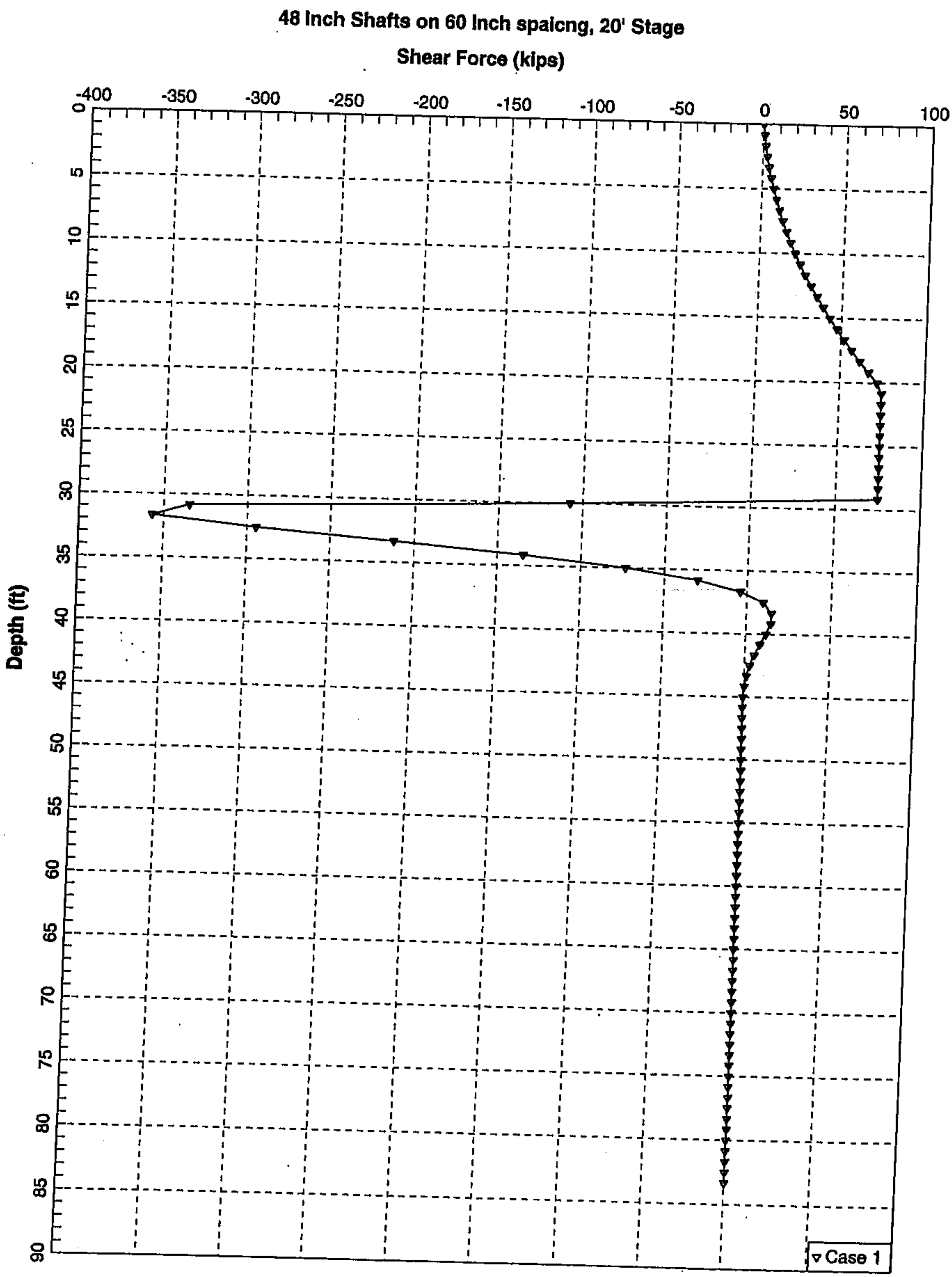
Load Type	Boundary Condition 1	Boundary Condition 2	Axial Load lbs	Pile Head Deflection in	Pile-Head Moment in-lbs	Pile Head Shear lbs
1	V= 0.000	M= 0.000	0.0000	.6175296	1.5202E+07	-357008.

Pile-head Deflection vs. Pile Length

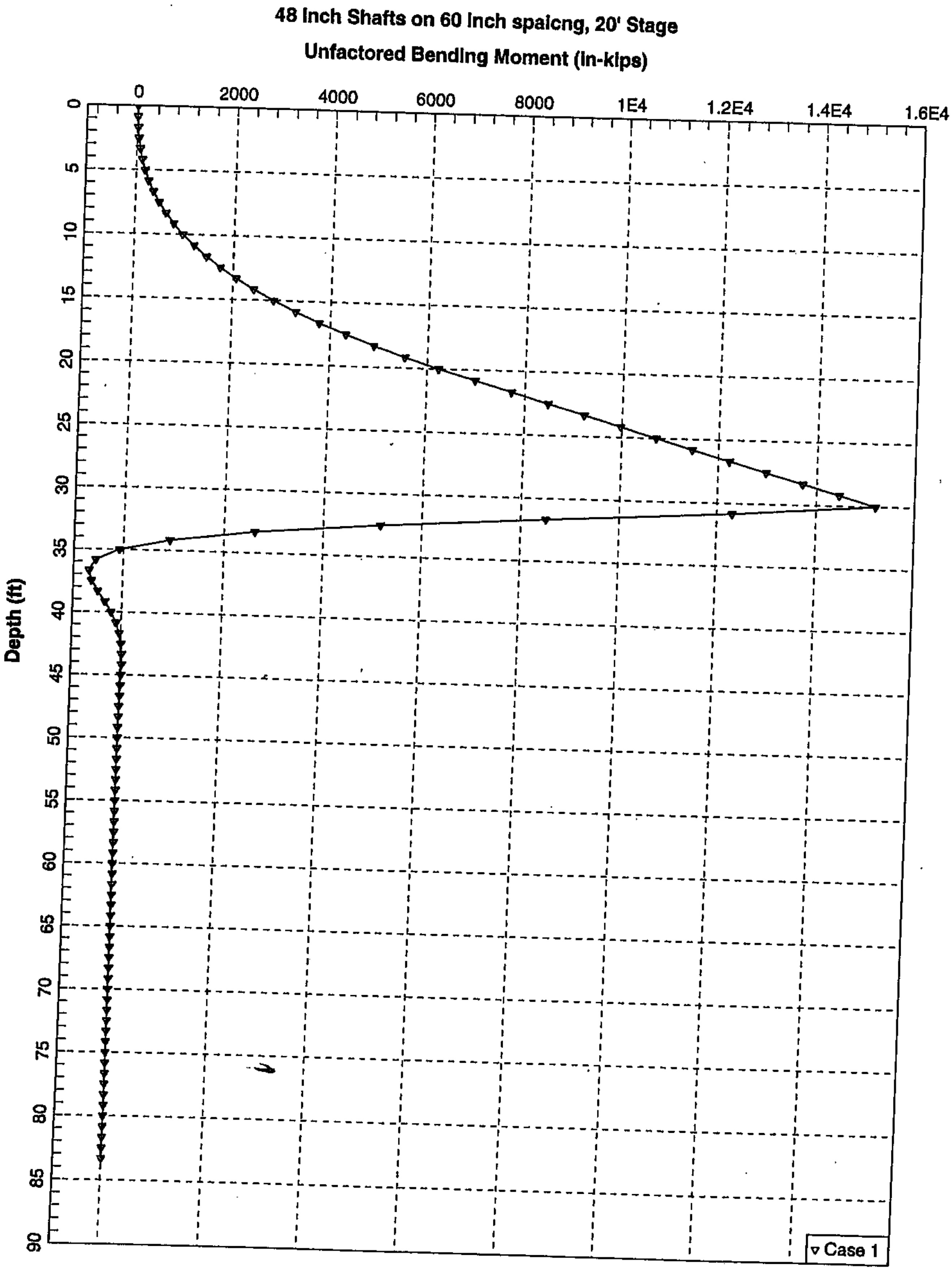
Boundary Condition Type 1, Shear and Moment

```
Shear       =     0. lbs
Moment      =     0. in-lbs
Axial Load  =     0. lbs
```

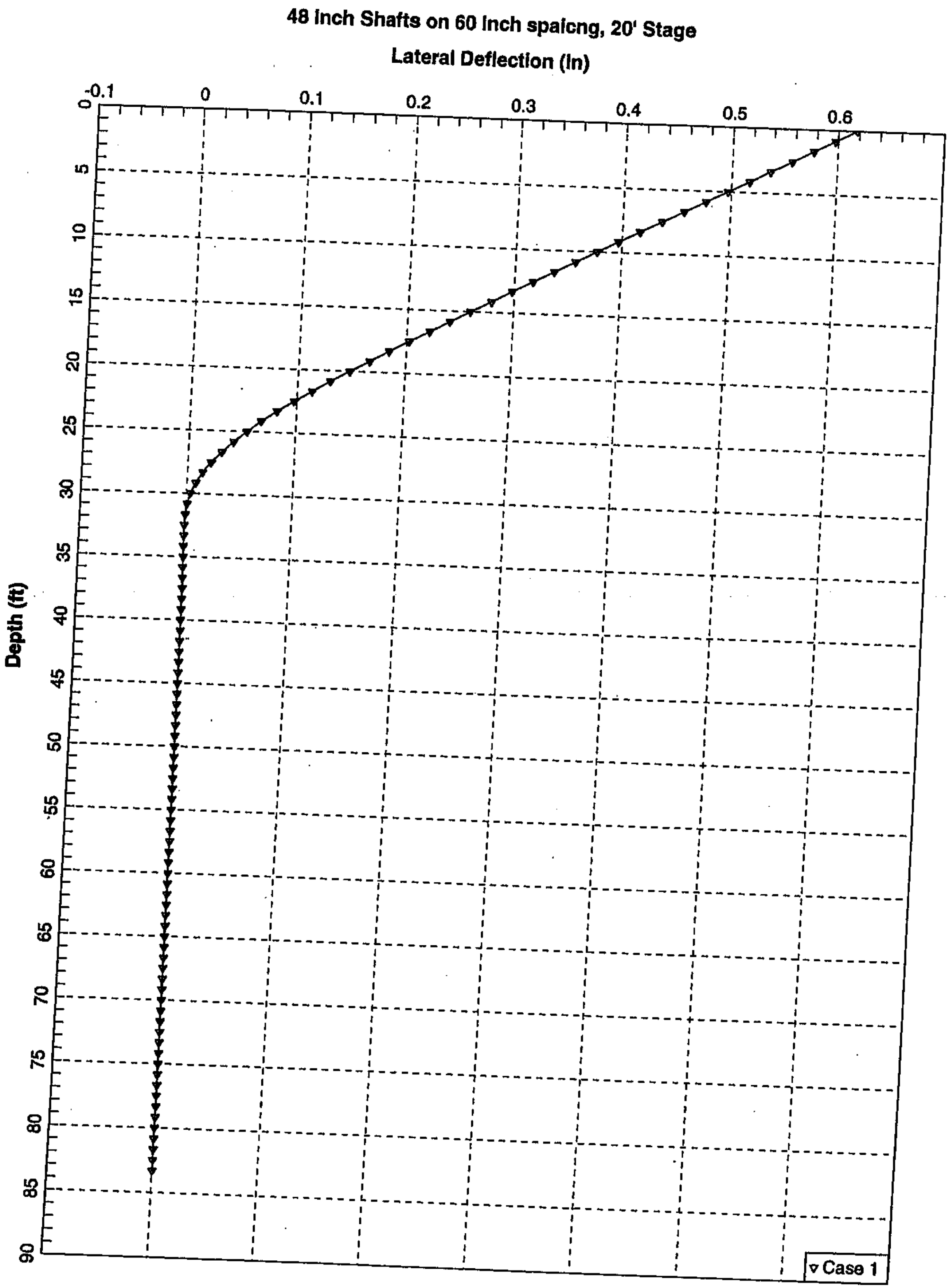
Pile Length in	Pile Head Deflection in	Maximum Moment in-lbs	Maximum Shear lbs
1000.000	.61752962	15201667.	-357007.55169
950.000	.62353982	15096851.	-356094.98536
900.000	.67037387	15417337.	-363864.33029
850.000	.65346184	15427181.	-362004.51737
800.000	.67988158	15732267.	-378337.37727
750.000	.67847649	15678281.	-378684.44173
700.000	.64872405	15267685.	-361007.69175
650.000	.63826064	14977361.	-355191.33770
600.000	.67200128	15516900.	-378042.03786
550.000	.63787495	15043766.	-362617.81619



48 Inch Shafts on 60 Inch spaicng, 20' Stage
Shear Force (kips)
-400
-350
-300
-250
-200
-150
-100
-50
0
50
100
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
Case 1



48 Inch Shafts on 60 Inch spaicng, 20' Stage
Unfactored Bending Moment (in-kips)
0
2000
4000
6000
8000
1E4
1.2E4
1.4E4
1.6E4
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
Case 1



48 inch Shafts on 60 inch spaicng, 20' Stage
Lateral Deflection (in)
-0.1
0
0.1
0.2
0.3
0.4
0.5
0.6
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
Case 1

G. ESTIMATED CONSTRUCTION COST & ESTIMATED QUANTITIES (DOES NOT INCLUDE RAILROAD SPECIAL PROVISIONS)

Estimate SCI-234-0122

Estimated Cost: $1,344,945.18

Contingency: 0.00%

Estimated Total: $1,344,945.18

SCI-823-0.00 (PID 19415) -- Structure SCI-TR234-0122 - SFN 7336934 - Shumway Hollow Road over CSXT Railroad

Base Date: 12/10/10

Spec Year: 08

Unit System: E

Work Type: BRIDGE REPLACEMENT

Highway Type: 448

Urban/Rural Type: RURAL CLASS

Season: SPRING

County: SCIOTO

Midpoint of Latitude: 385030

Midpoint of Longitude: 0825100

District: 9

Federal/State Project Number: PID 19415

Prepared by RBK on 07/24/09

Line #	Item Number	Description / *Supplemental Description*	Quantity	Units	Unit Price	Exten

Group 9000: STRUCTURE OVER 20 FOOT SPAN

Line #	Item Number	Description / *Supplemental Description*	Quantity	Units	Unit Price	Exten
0001	503E21100	UNCLASSIFIED EXCAVATION	818.000	CY	$29.19398	$23,88
0002	509E10000	EPOXY COATED REINFORCING STEEL	166,759.000	LB	$0.95765	$159,69
0003	512E10100	SEALING OF CONCRETE SURFACES (EPOXY-URETHANE)	919.000	SY	$12.31152	$11,31
0004	515E15051	DRAPED STRAND PRESTRESSED CONCRETE BRIDGE I-BEAM MEMBERS, LEVEL 3, TYPE 4 MOD. (72"), AS PER PLAN	8.000	EACH	$27,057.76173	$216,46
0005	515E20000	INTERMEDIATE DIAPHRAMS	28.000	EACH	$917.03771	$25,67
0006	516E13900	2" PREFORMED EXPANSION JOINT FILLER	163.000	SF	$8.92171	$1,45
0007	516E14015	INTEGRAL ABUTMENT EXPANSION JOINT SEAL, AS PER PLAN	62.000	FT	$30.00000	$1,86
0008	516E14021	SEMI-INTEGRAL ABUTMENT EXPANSION JOINT SEAL, AS PER PLAN	62.000	FT	$35.00000	$2,17
0009	516E44200	ELASTOMERIC BEARING WITH INTERNAL LAMINATES AND LOAD PLATE (NEOPRENE) *24"x16"x4"*	16.000	EACH	$903.52884	$14,45
0010	518E21200	POROUS BACKFILL WITH FILTER FABRIC	323.000	CY	$50.49894	$16,31
0011	518E40000	6" PERFORATED CORRUGATED PLASTIC PIPE	214.000	FT	$9.09720	$1,94
12	518E40010	6" NON-PERFORATED CORRUGATED PLASTIC PIPE, INCLUDING SPECIALS	18.000	FT	$8.60383	$15
0013	524E94702	DRILLED SHAFTS, 36" DIAMETER, ABOVE BEDROCK	75.000	FT	$239.13482	$17,93
0014	524E94704	DRILLED SHAFTS, 36" DIAMETER, INTO BEDROCK	300.000	FT	$331.33129	$99,39
0015	840E20000	MECHANICALLY STABILIZED EARTH WALL	2,492.000	SF	$35.00000	$87,22
0016	840E21000	WALL EXCAVATION	1,033.000	CY	$5.00000	$5,16
0017	840E22000	FOUNDATION PREPARATION	518.000	SY	$15.00000	$7,77
0018	840E23000	SELECT GRANULAR BACKFILL	691.000	CY	$30.00000	$20,73
0019	840E23050	NATURAL SOIL	6,649.000	CY	$20.00000	$132,98
0020	840E25010	6" DRAINAGE PIPE, PERFORATED	385.000	FT	$5.00000	$1,92
0021	840E25020	6" DRAINAGE PIPE, NON-PERFORATED	40.000	FT	$5.00000	$20
0022	840E26000	CONCRETE COPING	197.000	FT	$100.00000	$19,70
0023	840E27000	ON-SITE ASSISTANCE	12.000	DAY	$600.00000	$7,20
0024	840E28000	SGB INSPECTION AND COMPACTION TESTING	1.000	LS	$10,000.00000	$10,00
25	898E10201	QC/QA CONCRETE, CLASS QSC2, SUPERSTRUCTURE (DECK), AS PER PLAN	340.000	CY	$606.96812	$206,36
0026	898E10709	QC/QA CONCRETE, CLASS QSC2, SUPERSTRUCTURE (APPROACH SLAB), (T=17"), AS PER PLAN	500.000	SY	$200.00000	$100,00
0027	898E11000	QC/QA CONCRETE, CLASS QSC2, SUPERSTRUCTURE (PARAPET)	36.000	CY	$496.87351	$17,88
0028	898E20150		143.000	CY	$520.47951	$74,42

Line #	Item Number	Quantity	Units	Unit Price	Exten
Description					
Supplemental Description					
QC/QA CONCRETE, CLASS QSC1, SUBSTRUCTURE (ABUTMENT)					
0029	898E20300	154.000	CY	$393.83824	$60,65
QC/QA CONCRETE, CLASS QSC1, SUBSTRUCTURE (FOOTING)					

Total for Group 9000: $1,344,945.18

Company: KZFDESIGN
Structure : SCI-234-0122 | Design : DAT | Date : 7/23/2009
Subject: Quantities (Stage 2) | Checked : | Date :

ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
503	21100	818	CU YD	UNCLASSIFIED EXCAVATION	818				
509	10000	166759	POUND	EPOXY COATED REINFORCING STEEL	35667		131092		
512	10100	919	SQ YD	SEALING OF CONCRETE SURFACES (EPOXY-URETHANE)	374		544		
515	15051	8	EACH	DRAPED STRAND PRESTRESSED CONCRETE BRIDGE I-BEAM, AS PER PLAN			8		
				MEMBERS, LEVEL 3, TYPE 4 MOD (72")					
515	20000	28	EACH	INTERMEDIATE DIAPHRAGMS			28		
516	13900	163	SQ FT	2" PREFORMED EXPANSION JOINT FILLER	163				
516	14015	62	FT	INTEGRAL ABUTMENT EXPANSION JOING SEAL, AS PER PLAN	62				
516	14021	62	FT	SEMI-INTEGRAL ABUTMENT EXPANSION JOING SEAL, AS PER PLAN	62				
516	44200	16	EACH	ELASTOMERIC BEARING WITH INTERNAL LAMINATES AND LOAD PLATE (NEOPRENE)	16				
				24"X16"X3.96" LAMINATED ELASTOMERIC PAD WITH 16"X16"X1" LOAD PLATE					
518	21200	323	CU YD	POROUS BACKFILL WITH FILTER FABRIC	323				
518	40000	214	FT	6" PERFORATED CORRUGATED PLASTIC PIPE	214				
518	40012	18	FT	6" NON-PERFORATED CORRUGATED PLASTIC PIPE	18				
524	94702	75	FT	DRILLED SHAFTS, 36" DIAMETER, ABOVE BEDROCK	75				
524	94704	300	FT	DRILLED SHAFTS, 36" DIAMETER, INTO BEDROCK	300				
840	20000	2492	SQ FT	Mechanically Stabilized Earth Wall	2492				
840	21000	1033	CU YD	Wall Excavation	1033				
840	22000	518	SQ YD	Foundation Preparation	518				
840	23000	691	CU YD	Select Granular Backfill	691				
840	23050	6649	CU YD	Natural Soil	6649				
840	25010	385	FT	6" Drainage Pipe, Perforated	385				
840	25020	40	FT	6" Drainage Pipe, Non-Perforated	40				
840	26000	197	FT	Concrete Coping	197				
840	27000	12	DAY	On-Site Assistance	12				
840	28000	SUM	LUMP	SGB Inspection and Compaction Testing	SUM				
898	10200	340	CU YD	QC/QA CONCRETE, CLASS QCS2, SUPERSTRUCTURE (DECK)			340		
898	10708	500	SQ YD	QC/QA CONCRETE, CLASS QCS2, SUPERSTRUCTURE (APPROACH SLAB), (T=17")			500		
898	11000	36	CU YD	QC/QA CONCRETE, CLASS QCS2, SUPERSTRUCTURE (PARAPET)			36		
898	20150	143	CU YD	QC/QA CONCRETE, CLASS QCS1, SUBSTRUCTURE (ABUTMENT)	143				
898	20300	154	CU YD	QC/QA CONCRETE, CLASS QCS1, SUBSTRUCTURE (FOOTING)	154				

Company:	KZF DESIGN								
Structure :	SCI-234-0122			Design :	DAT	Date :	7/23/2009		
Subject:	Quantities (Stage 2)			Checked :		Date :			
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
503	21100	817.7778	CU YD	UNCLASSIFIED EXCAVATION	817.7778				
at forward abutment:									
L (ft)	W (ft)	H (ft)	Volume						
120	16	11.5	22080	ft^3					
			817.778	cu yd	817.7778				
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
509	10000	166759.3	POUND	EPOXY COATED REINFORCING STEEL	35667.01		131092.3		
Note: Calculate the amount of reinforcing steel per Cu.Yd. of concrete and multiply by the concrete volume to obtain the total									
weight of reinforcing steel (in LBS).									
Note: Laps are not included in these calculations. The approach slab & drilled shaft reinforcing is not included in these calculations.									
>>>>>	QC/QA CONCRETE, CLASS QCS2, SUPERSTRUCTURE (DECK)								
		Note: The abutment and pier diaphragm concrete is included in the pay item for deck volume.							
L (ft)	spacing (in.)	bar size	wt (per ft)	transverse (top)					
58.67	5.5	5	133.512						
L (ft)		bar size	wt (per ft)	transverse (bottom)					
58.67	5.5	5	133.512						
	# bars	bar size	wt (per ft)	top bars (temp & shrinkage)					
	88	4	58.784						
	# bars	bar size	wt (per ft)	top bars (additional)					
	0	0	0						
	# bars	bar size	wt (per ft)	bottom bars					
	88	5	91.784						
		total =	417.591	lbs					
	deck volume								
L (ft)	W (ft)	T (ft)	Volume						
58.67	1	0.729167	44.0745	ft^3					
			1.63239	cu yd					
		total =	255.816	lbs/cu yd			87054.87		
>>>>>	QC/QA CONCRETE, CLASS QCS2, SUPERSTRUCTURE (PARAPET)								
L (ft)	spacing (in.)	bar size	wt (per ft)	transverse (dowels)					
4	12	6	6.008						
L (ft)		bar size	wt (per ft)	transverse (main)					
6.8333333	12	5	7.12717						
	# bars	bar size	wt (per ft)	longitudinal bars					
	7	5	7.301						
		total =	20.4362	lbs					
	parapet volume								
	W (ft)	A (ft^2)	Volume						
	1	4.263889	4.26389	ft^3					
			0.15792	cu yd					
		total =	129.407	lbs/cu yd			44037.46		
>>>>>	QC/QA CONCRETE, CLASS QCS1, SUBSTRUCTURE (ABUTMENT)								
	take the average of the rear and forward abutment reinforcing per cu. yd.								
	at rear abutment:								
L (ft)	spacing (in.)	bar size	wt (per ft)	stirrups					
5.8333333	12	5	6.08417						
	# bars	bar size	wt (per ft)	longitudinal bars					
	5	8	13.35						
	# bars	bar size	wt (per ft)	longitudinal bars					
	2	6	3.004						
		total =	22.4382	lbs					
	abutment volume								
H (ft)	W (ft)	L (ft)	Volume						
1.75	3	1	5.25	ft^3					
			0.19444	cu yd					
		total =	115.396	lbs/cu yd					
	at forward abutment:								
L (ft)	spacing (in.)	bar size	wt (per ft)	stirrups					
21	12	6	31.542						
L (ft)	spacing (in.)	bar size	wt (per ft)	stirrups (main from shaft)					
10	12	8	26.7						
L (ft)	spacing (in.)	bar size	wt (per ft)	ties					
23.333333	24	4	7.79333						
	# bars	bar size	wt (per ft)	longitudinal bars					
	4	8	10.68						
	# bars	bar size	wt (per ft)	longitudinal bars					
	18	6	27.036						
		total =	103.751	lbs					
	abutment volume								
H (ft)	W (ft)	L (ft)	Volume						
9.3333333	3	1	28	ft^3					
			1.03704	cu yd					
		total =	100.046	lbs/cu yd					
		avg. total =	107.721	lbs/cu yd	15372.43				
>>>>>	QC/QA CONCRETE, CLASS QCS1, SUBSTRUCTURE (FOOTING)								
	take the average of the rear and forward abutment reinforcing per cu. yd.								
	at rear abutment:								
L (ft)	spacing (in.)	bar size	wt (per ft)	stirrups					
34	12	6	51.068						
	# bars	bar size	wt (per ft)	main top					
	8	8	21.36						
	# bars	bar size	wt (per ft)	main side					
	0	0	0						
	# bars	bar size	wt (per ft)	main bottom					
	8	8	21.36						
		total =	93.788	lbs					
	abutment footing volume								
H (ft)	W (ft)	L (ft)	Volume						

Company:	KZFDESIGN								
Structure :	SCI-234-0122			Design :	DAT	Date :	7/23/2009		
Subject:	Quantities (Stage 2)			Checked :		Date :			
3	10.5	1	31.5	ft^3					
			1.16667	cu yd					
		total =	80.3897	lbs/cu yd					
	at forward abutment:								
L (ft)	spacing (in.)	bar size	wt (per ft)	stirrups					
24.666667	6	5	51.4547						
	# bars	bar size	wt (per ft)	main top					
	5	8	13.35						
	# bars	bar size	wt (per ft)	main side					
	2	6	3.004						
	# bars	bar size	wt (per ft)	main bottom					
	5	8	13.35						
		total =	81.1587	lbs					
	pier footing volume								
T (ft)	W (ft)	L (ft)	Volume						
3	4	1	12	ft^3					
			0.44444	cu yd					
		total =	182.607	lbs/cu yd					
		avg. total =	131.498	lbs/cu yd	20294.58				
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
512	10100	918.7436	SQ YD	SEALING OF CONCRETE SURFACES (EPOXY-URETHANE)	374.455		544.2885		
at superstructure (transverse section):									
W (ft)	L (ft)	Area							
40.31767	121.5	4898.597	ft^2						
		544.2885	sq yd				544.2885		
at abutments:		take average heights at each abutment							
H (ft)	L (ft)	Area							
10.125	171.67	3370.095	ft^2						
		374.455	sq yd		374.455				
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
515	15051	8	EACH	DRAPED STRAND PRESTRESSED CONCRETE BRIDGE I-BEAM, AS PER PLAN			8		
	# spans =	1							
	beams per span =	8							
	# beams =	8							
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
515	20000	28	EACH	INTERMEDIATE DIAPHRAGMS			28		
	# spans =	1							
	diaphragms per span =	4							
	# bays (btw. beams) =	7							
	# diaphragms =	28							
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
516	13900	163.26	SQ FT	2" PREFORMED EXPANSION JOINT FILLER	163.26				
at abutment:									
H (ft)	W (ft)	Area							
9.83	3.00	58.98	ft^2	btw. wingwall and abutment diaphragm (rear abut.)					
17.38	3.00	104.28	ft^2	btw. wingwall and abutment diaphragm (fwd. abut.)					
	total =	163.26	ft^2						
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
516	14015	62.00	FT	INTEGRAL ABUTMENT EXPANSION JOING SEAL, AS PER PLAN	62.00				
	at abutment:								
		L							
		62.00	ft	forward abutment					
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
516	14021	62.00	FT	SEMI-INTEGRAL ABUTMENT EXPANSION JOING SEAL, AS PER PLAN	62.00				
	at abutment:								
		L							
		62.00	ft	rear abutment					
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
516	44200	16	EACH	ELASTOMERIC BEARING WITH INTERNAL LAMINATES AND LOAD PLATE (NEOPRENE)	16				
				24"X16"X3.96" LAMINATED ELASTOMERIC PAD WITH 16"X16"X1" LOAD PLATE					
	at abutments:								
	# bearings =	16	each	2 abutments					
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
518	21200	323.3	CU YD	POROUS BACKFILL WITH FILTER FABRIC	323.3				
H (ft)	W (ft)	L (ft)	Volume						
12.708333	2	171.7	8728.1	ft^3					
			323.3	cu yd					
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
518	40000	214	FT	6" PERFORATED CORRUGATED PLASTIC PIPE	214.0				
		L (ft)							
		214							
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
518	40012	18	FT	6" NON-PERFORATED CORRUGATED PLASTIC PIPE	18				
		L (ft)							
		18							
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
524	94702	75.00	FT	DRILLED SHAFTS, 36" DIAMETER, ABOVE BEDROCK	75.00				
	top elev	bedrock	shaft						
shaft #	elevation	elevation	length (ft)						
1	640.00	636.00	4.00						
2	640.00	635.86	4.14						

Company:	KZFDESIGN								
Structure :	SCI-234-0122			Design :	DAT	Date :	7/23/2009		
Subject:	Quantities (Stage 2)			Checked :		Date :			
3	640.00	635.71	4.29						
4	640.00	635.57	4.43						
5	640.00	635.43	4.57						
6	640.00	635.29	4.71						
7	640.00	635.14	4.86						
8	640.00	635.00	5.00						
9	640.00	634.86	5.14						
10	640.00	634.71	5.29						
11	640.00	634.57	5.43						
12	640.00	634.43	5.57						
13	640.00	634.29	5.71						
14	640.00	634.14	5.86						
15	640.00	634.00	6.00						
		total =	75.00	ft.					
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
524	94704	300.00	FT	DRILLED SHAFTS, 36" DIAMETER, INTO BEDROCK	300.00				
	bedrock	tip	shaft						
shaft #	elevation	elevation	length (ft)						
1	636.00	615.00	21.00						
2	635.86	615.00	20.86						
3	635.71	615.00	20.71						
4	635.57	615.00	20.57						
5	635.43	615.00	20.43						
6	635.29	615.00	20.29						
7	635.14	615.00	20.14						
8	635.00	615.00	20.00						
9	634.86	615.00	19.86						
10	634.71	615.00	19.71						
11	634.57	615.00	19.57						
12	634.43	615.00	19.43						
13	634.29	615.00	19.29						
14	634.14	615.00	19.14						
15	634.00	615.00	19.00						
		total =	300.00	ft.					
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
840	20000	2491.5	SQ FT	Mechanically Stabilized Earth Wall	2491.50				
L (ft)	H (ft)	Area (ft^2)							
40	20	400.0	LHS						
110	19.5	1072.5	Center						
38	19	361.0	RHS						
188	7	658.0	bottom						
	total =	2491.5	ft^2						
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
840	21000	1032.746	CU YD	Wall Excavation	1032.75				
L (ft)	W (ft)	H (ft)	Volume						
198	23.55	6.0	27884.1	ft^3					
			1032.75	cu yd					
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
840	22000	518.1	SQ YD	Foundation Preparation	518.10				
	L (ft)	W (ft)	area						
	198	23.55	4662.9	sq ft					
			518.1	sq yd					
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
840	23000	690.8	CU YD	Select Granular Backfill	690.80				
L (ft)	W (ft)	H (ft)	Volume						
198	23.55	4.0	18651.6	ft^3					
			690.8	cu yd					
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
840	23050	6649.36	CU YD	Natural Soil	6649.36				
L (ft)	W (ft)	H (ft)	Volume						
198	18.55	24.3	89067.8	(ft^3) area 1 (from top footing to top of wall excavation), rectangular shape					
			3298.81	cu yd					
198	24.3	24.3	58218.2	(ft^3) area 2 (from top footing to top of wall excavation), triangular shape					
			2156.23	cu yd					
99	28.0	9.0	24929.5	(ft^3) area 3 (from top footing to bottom of approach slab), rectangular shape					
			923.316	cu yd					
99	7.1	10.4	7317.19	(ft^3) area 4 (from top footing to limits of roadway quantities), rectangular shape					
			271.007	cu yd					
		total =	6649.36	cu yd					
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
840	25010	385.4	FT	6" Drainage Pipe, Perforated	385.44				
	L1 =	188.0	ft						
	L2 =	108.0	ft						
	L3 =	89.4	ft						
	total =	385.4	ft						
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
840	25020	40.0	FT	6" Drainage Pipe, Non-Perforated	40.00				
		40.0	ft						
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
840	26000	197.2	FT	Concrete Coping	197.21				
	top =	110.0	ft						
	slope =	87.2	ft						
	total =	197.2	ft						

Company:	KZFDESIGN								
Structure :	SCI-234-0122			Design :	DAT	Date :	7/23/2009		
Subject:	Quantities (Stage 2)			Checked :		Date :			
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
840	27000	12.0	DAY	On-Site Assistance	12.00				
		12.0	day						
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
840	28000	SUM	LUMP	SGB Inspection and Compaction Testing	SUM				
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
898	10200	340.3024	CU YD	QC/QA CONCRETE, CLASS QCS2, SUPERSTRUCTURE (DECK)			340.3		
note: Includes Deck and Abutment Diaphragm concrete.									
deck:									
L (ft)	W (ft)	T (ft)	Volume						
121.5	59	0.708333	5077.69	ft^3					
			188.063	cu yd					
deck overhang:									
L (ft)	W (ft)	T (ft)	Volume						
121.5	0.875	0.25	53.1563	ft^3					
			1.96875	cu yd					
beam haunches:									
L (ft)	W (ft)	T (ft)	Volume						
121.5	24	0.290833	1696.14	ft^3					
			62.82	cu yd					
abutment diaphragms:									
H (ft)	W (ft)	L (ft)	Volume						
6.67	3	118	2361.18	ft^3					
			87.4511	cu yd					
		total =	340.302	cu yd					
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
898	10708	500	SQ YD	QC/QA CONCRETE, CLASS QCS2, SUPERSTRUCTURE (APPROACH SLAB), (T=17")			500.0		
	L (ft)	W (ft)	Area						
	30	59	1770	sq ft (at rear abutment)					
			196.667	sq yd					
	30	91	2730	sq ft (at forward abutment)					
			303.333	sq yd					
		total =	500	sq yd					
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
898	11000	36.09311	CU YD	QC/QA CONCRETE, CLASS QCS2, SUPERSTRUCTURE (PARAPET)			36.1		
main parapet:									
	Area (ft^2)	L (ft)	Volume						
	8.5278	93.5	797.347	ft^3					
			29.5314	cu yd					
transition parapet:									
	Area (ft^2)	L (ft)	Volume						
	6.9028	20	138.056	ft^3					
			5.11317	cu yd					
end parapet:									
	Area (ft^2)	L (ft)	Volume						
	4.8889	8	39.1111	ft^3					
			1.44856	cu yd					
		total =	36.0931	cu yd					
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
898	20150	142.71	CU YD	QC/QA CONCRETE, CLASS QCS1, SUBSTRUCTURE (ABUTMENT)	142.71				
rear abutment:									
at beam seat:									
H (ft)	W (ft)	L (ft)	Volume						
1.75	3	59	309.75	ft^3					
			11.4722	cu yd					
at wingwalls:									
H (ft)	W (ft)	L (ft)	Volume						
4.455	2.5	35.66	397.163	ft^3					
			14.7098	cu yd					
		subtotal =	26.182	cu yd	26.18197				
forward abutment:									
at beam seat:									
H (ft)	W (ft)	L (ft)	Volume						
9.3	3	59	1646.1	ft^3					
			60.9667	cu yd					
at wingwalls:									
H (ft)	W (ft)	L (ft)	Volume						
13.355	3	16	641.04	ft^3 (left wall)					
			23.7422	cu yd					
10.605	3	27	859.005	ft^3 (right wall)					
			31.815	cu yd					
		subtotal =	116.524	cu yd	116.5239				
		total =	142.706	cu yd					
ITEM	EXTENSION	TOTAL	UNIT	DESCRIPTION	ABUT	PIER	SUPER	GEN	REF
898	20300	154.33	CU YD	QC/QA CONCRETE, CLASS QCS1, SUBSTRUCTURE (FOOTING)	154.33				
at rear abutment:									
H (ft)	W (ft)	L (ft)	Volume						
3.00	10.50	90	2835	ft^3					
			105	cu yd					
at forward abutment:									
H (ft)	W (ft)	L (ft)	Volume						
3.00	4.00	111	1332	ft^3					
			49.3333	cu yd					
		total =	154.333	cu yd	154.3333				