STRUCTURE GEOTECHNICAL REPORT

054-0516

Existing SN 054-0002

FR I-55 (Frontage Road) over Kickapoo Creek
Section 21 ACB
Logan County

D-96-008-09

Contract 72C33

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Date: September 21, 2016

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Date: September 21, 2016
(Rev.)
April 27, 2015

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Attachments: Preliminary TSL
Subsurface Profile
Boring Logs
Special Provisions

This Report has been prepared based on a preliminary TSL from March 2013. Contact the author if there are any questions regarding this Report or if there are modifications to structure location, size, geometry, or vertical alignment.

Electronic copies of boring logs are available upon request for inclusion in the plans. Calculations are also available upon request.

This Report has been prepared according to the 2012 IDOT Bureau of Bridges and Structures Bridge Manual and AASHTO LRFD Bridge Design Specifications 7th Edition – 2014 with 2015, 2016 Interims.

Project Description and Proposed Structure Information

The project includes replacing an existing 306’-10” long and 36’-0” wide five-span structure (SN 054-0002) with a new 311’-10” long and 39’-2” wide, three-span structure (SN 054-0516). The proposed structure includes integral or semi-integral abutments and solid wall encased piers. Work will be completed under road closure.

Site Investigation

The project is located approximately 3.4 miles Southwest of Atlanta (0.4 miles Northeast of Lawndale). It carries a frontage road from Lincoln to Atlanta over Kickapoo Creek. The primary land use within the project area is agriculture with intermittent locations of timber. Approximately 150 ft. downstream to the west is a Union Pacific three span bridge. Approximately 90 ft. and 210 ft. upstream to the east are dual three span bridges carrying I-55.

The original structure was built in 1922, as a 282’- 0” four-span structure founded on timber piles. It was replaced be the existing structure (SN 054-0002) built in 1953, as a 306’-10” long and 36’-0” wide five-span structure. The piers are founded on timber piles and the abutments are founded on concrete and metal shell pile. From the existing plans the pier piles appear to be approximately +/-15’-0” in length and the abutment piles appear to be +/- 28’-0” in length. No pile records are available for the existing structure to verify the actual driven lengths.

The existing roadway is located on approximately 12 ft. of fill with 6H:1V or slightly flatter slopes on east side and 2H:1V on the west side of the frontage road . There are ditches on either side of the structure. No embankment slope stability problems have been observed, and there is no evidence of approach settlement problems.

Borings obtained in 1950's were not used because of the lack of information. Borings were advanced by the District 6 drill crew using hollow stem auger methods according to AASHTO T 206 and the IDOT Geotechnical Manual. Borings were filled with cuttings immediately after drilling to allow traffic on the roadway. The boring data indicates mostly Silty Clay Loam and Sand over Sandy Gravel and Clay Loam (Till). The hard glacial (Till) strengths range from 9.0 – 13.4 tsf with blow counts ranging from 50 -100 blow per 6" of penetration, and were encountered at elevation 553.50 to 557.10

Geotechnical Evaluation

Settlement: No change in grade is proposed. No settlement problems are anticipated

Slope Stability: There is no evidence of any slope stability problems with the existing cross slopes. No slope stability analysis is needed due to the project being constructed under a road closure.

Seismic Considerations: The following table shows recommended seismic design data based on a 1000 year return period event.

Table #1	
Seismic Performance Zone (SPZ)	1
Spectral Acceleration at 1 second (S_{D1})	0.132g
Design Spectral Acceleration at 0.2 Seconds (S_{DS})	0.221g
Soil Site Class	D

Scour: Scour elevations for a 100 and 500 year event was determined by the District 6 Hydraulics unit. The following table shows recommended design scour elevations at each substructure unit. The design scour elevation at abutments is equal to the proposed bottom of abutment elevation. Some adjustment to bottom of abutment elevation may be made during final design.

Table #2					
Event/Limit State	Design Scour Elevation (ft)				Item 113
	South Abut.	Pier 1	Pier 2	North Abut.	
Q_{100}	594.73	579.06	579.00	594.76	5
Q_{500}	594.73	574.02	573.96	594.76	
Design	594.73	575.45	575.45	594.76	
Check	594.73	574.02	573.96	594.76	

Mining Activity: ISGS records indicate no mines in the proposed project area.

Foundation Evaluation

Vertical Loading

Preliminary maximum factored loads, provided by the structure designer, are approximately 1649 kips vertical at the abutments and 2407 kips vertical at the piers. The analysis included steel H-pile, metal shell pile and drilled shafts. From the analysis, only H-piles (min HP 12x53) are feasible if an integral abutment is selected. Metal shell piles and drilled shafts should be used for semi-integral abutment only. Spread footings will not be evaluated because of inadequate bearing capacity.

Piles

A pile supported substructure is feasible for all substructure locations given the preliminary axial loads provided by the structural designer. With the soil conditions present, the controlling factor in the pile design is skin friction. No bedrock was encountered during drilling.

As mention above, the proposed structure will be 39'-2" wide. It will be on a 22 degree right ahead skew, making the skew length 42'-3". Based on 3' and 8' center spacing, the approximate factored loading applied per pile are as follows:

Table #3				
Factored Loading Applied Per Pile				
Row of Piles	Abutment Loads (1,649 kips)		Pier Loads (2,407 kips)	
	Spacing		Spacing	
	3 ft.	8 ft.	3 ft.	8 ft.
1	117.09 kips	312.24 kips	170.91 kips	455.76 kips

Due to the far-off letting date for this project, IDOT BBS would like to use our new Supplement (not-yet-published) to the 2012 IDOT Integral Abutment Bridge Policy, ABD 12.3 for this structure, which it will allow the use of Metal Shell piles with Integral Abutment. This new Supplement to the ABD 12.3 will replace the current ABD Memorandum 12.3, and it is anticipated to arrive later this year. Attached is a draft of the new policy's Integral Abutment Pile

Selection Chart. Metal Shell piles are preferred at this location, because H-pile lengths are very difficult to predict when bedrock is not encountered in the boring logs."

The cutoff elevation for both abutments is 596.70' (Integral). Ground elevation during driving is 594.70' for both abutments.

Table #4							
Pile Design Table -- South Abutment (Boring 1SE Abut.)							
Est. Pile Tip Elev.	Est. Pile Length (ft.)	Metal Shell 12" w/0.25" Wall Thickness, *Max R_{NRB} = **282** kips		Metal Shell 14" w/0.25" Wall Thickness, *Max R_{NRB} = **330** kips		Metal Shell 14" w/0.312" Wall Thickness, *Max R_{NRB} = **410** kips	
		R_{NRB} (kips)	R_{FRA} (kips)	R_{NRB} (kips)	R_{FRA} (kips)	R_{NRB} (kips)	R_{FRA} (kips)
575.70	21	99	55	125	69	125	69
570.70	26	118	65	148	81	148	81
566.70	30	149	82	182	100	182	100
565.70	31	164	90	200	110	200	110
564.70	32	174	96	212	117	212	117
563.70	33	**282**	**155**	**330**	**182**	**410**	**226**
* Max R_{NRB} was reduced by 20% to prevent pile damage during driving.							

Table #5							
Pile Design Table -- North Abutment (Boring 2 NW Abut.)							
Est. Pile Tip Elev.	Est. Pile Length (ft.)	Metal Shell 12" w/0.25" Wall Thickness, *Max R_{NRB} = **282** kips		Metal Shell 14" w/0.25" Wall Thickness, *Max R_{NRB} = **330** kips		Metal Shell 14" w/0.312" Wall Thickness, *Max R_{NRB} = **410** kips	
		R_{NRB} (kips)	R_{FRA} (kips)	R_{NRB} (kips)	R_{FRA} (kips)	R_{NRB} (kips)	R_{FRA} (kips)
572.70	24	82	45	108	59	108	59
571.70	25	89	49	116	64	116	64
570.70	26	102	56	124	68	124	68
569.70	27	243	134	314	173	314	173
568.70	28	251	138	323	178	323	178
567.70	29	259	142	332	183	332	183
566.70	30	267	147	341	188	341	188
565.70	31	274	151	350	193	350	193
564.70	32	249	137	314	173	314	173
563.70	33	254	140	321	176	321	176
562.70	34	260	143	327	180	327	180
561.70	35	266	146	**330**	**182**	334	184
560.70	36	272	149			341	187
559.70	37	**282**	**155**			369	203
558.70	38					383	210
557.70	39					396	218
556.70	40					**410**	**226**
* Max R_{NRB} was reduced by 20% to prevent pile damage during driving.							

R_{NRB} = Nominal Required Bearing R_{FRA} = Factored Resistance Available

The cutoff elevation for both piers is 596.50'. Ground elevation during driving is 575.5' for both piers.

Table #6					
Pile Design Table -- Pier #1 (Boring 1A S. Pier)					
Est. Pile Tip Elev.	Est. Pile Length (ft.)	Metal Shell 14" w/0.25" Wall Thickness, *Max R_{NRB} = **330** kips		Metal Shell 14" w/0.312" Wall Thickness, *Max R_{NRB} = **410** kips	
		R_{NRB} (kips)	R_{FRA} (kips)	R_{NRB} (kips)	R_{FRA} (kips)
560.50	36	283	156	283	156
559.50	37	295	162	295	162
558.50	38	307	169	307	169
557.50	39	319	175	319	175
556.50	40	**330**	**182**	**410**	**226**
* Max R_{NRB} was reduced by 20% to prevent pile damage during driving.					

Table #7					
Pile Design Table -- Pier #1 (Boring 1B S. Pier)					
Est. Pile Tip Elev.	Est. Pile Length (ft.)	Metal Shell 14" w/0.25" Wall Thickness, *Max R_{NRB} = **330** kips		Metal Shell 14" w/0.312" Wall Thickness, *Max R_{NRB} = **410** kips	
		R_{NRB} (kips)	R_{FRA} (kips)	R_{NRB} (kips)	R_{FRA} (kips)
568.50	28	128	70	128	70
565.50	31	144	79	144	79
564.50	32	155	85	155	85
563.50	33	166	91	166	91
562.50	34	**330**	**182**	371	204
561.50	35			402	221
560.50	36			**410**	**226**
* Max R_{NRB} was reduced by 20% to prevent pile damage during driving.					

R_{NRB} = Nominal Required Bearing
R_{FRA} = Factored Resistance Available

Table #8					
Pile Design Table -- Pier #2 (Boring 2A N. Pier)					
Est. Pile Tip Elev.	Est. Pile Length (ft.)	Metal Shell 14" w/0.25" Wall Thickness, *Max R_{NRB} = **330** kips		Metal Shell 14" w/0.312" Wall Thickness, *Max R_{NRB} = **410** kips	
		R_{NRB} (kips)	R_{FRA} (kips)	R_{NRB} (kips)	R_{FRA} (kips)
556.50	40	212	116	212	116
555.50	41	224	123	224	123
554.50	42	**330**	**181**	**410**	**226**
* Max R_{NRB} was reduced by 20% to prevent pile damage during driving.					

Table #9					
Pile Design Table -- Pier #2 (Boring 3B N. Pier)					
Est. Pile Tip Elev.	Est. Pile Length (ft.)	Metal Shell 14" w/0.25" Wall Thickness, *Max R_{NRB} = **330** kips		Metal Shell 14" w/0.312" Wall Thickness, *Max R_{NRB} = **410** kips	
		R_{NRB} (kips)	R_{FRA} (kips)	R_{NRB} (kips)	R_{FRA} (kips)
556.50	40	209	115	209	115
555.50	41	221	122	221	122
554.50	42	233	128	233	128
553.50	43	**330**	**182**	**410**	**226**
* Max R_{NRB} was reduced by 20% to prevent pile damage during driving.					

R_{NRB} = Nominal Required Bearing
R_{FRA} = Factored Resistance Available

Drilled Shafts

A drilled shaft supported substructure is feasible for all substructure locations given the preliminary axial loads provided by the structural designer. As mentioned earlier, the boring data indicates mostly Silty Slay Loam and Sand over Sandy Gravel and Clay Loam (Till). The hard glacial (Till) strengths range from 9.0 – 13.4 tsf with blow counts ranging from 50 -100 blow per 6” of penetration, and were encountered at elevation 553.50’ to 557.10’.

Based on the strengths, the glacial Tills fall under the category of Intermediate Geomaterial (IGM). IGMs are transition materials between soils and rock. The distinction of IGMs from soils or rocks for geotechnical engineering purposes is made purely on the basis of strength. Strengths range from 5.0 tsf to 50.0 tsf for IGMs. Because the IGMs from the borings are in the lower strength range, their calculated nominal unit side resistance and nominal end bearing values more closely correspond to high strength clays. From this similarity and to error on the conservative side, it was decided to use the resistance factors of 0.45 (side resistance) and 0.40 (end bearing) for clay.

Axial Design, Service Limit State

The development of side and tip resistances is dictated by the amount of movement a shaft experiences. Side resistance is mobilized much earlier than that mobilized at the bottom of the shaft. The amount of displacement needed for full mobilization of side resistance is about 0.2% to 0.8% of the shaft diameter in cohesive soils. The amount of displacement required for mobilizing full tip resistance is function of base dimension which is about 5.0%. "See Figs. 10.8.2.2.2-1 and 10.8.2.2.2-2, p. 10-131 of 2014 AASHTO for Load Transfer Computations for Service Limit Design Check."

Appreciable side resistance is typically developed before significant load can be transferred to the base, especially in long slender shafts. The settlement required for mobilizing the full base capacity is usually too large, therefore, only a fraction of the available tip capacity is relied upon in design. Because the maximum displacement of the shafts is not known, a load-transfer analysis was not studied.

Axial Design, Strength Limit State

Per AASHTO 10.8.3.5, the failure criterion for the Strength Limit State is established at a base deflection of 5% of the base diameter. Accordingly, based on Fig. 10.8.2.2.2-2 and the 5% deflection, all of the end bearing will be mobilized for the Strength Limit State. Therefore, the nominal axial resistance will be composed of 100% of the side resistance and 100% of the end bearing resistance being added together. The Strength Limit State and Service Limit State must both be satisfied.

The resistance of a drilled shaft group to vertical load is not necessarily the sum of the axial resistance of the individual shafts within the group. The zone of influence from an individual drilled shaft may intersect with other shafts, depending on the shaft spacing. Because the spacing of the drilled shafts is not known, group settlement and block failure was not analyzed.

If the structural designer decides to utilize drilled shafts, then a more detail analysis would need to be performed, specifically on what the maximum allowed displacement for load-transfer and shaft spacing for group effects.

Table #10

Unit Side and End Bearing Resistance
South Abutment (Boring 1 SE Abut.)

Layer	Top	Bottom	Nominal q_s (ksf)	Nominal q_p (ksf)	Factored q_s (ksf)	Factored q_p (ksf)	Description
1	594.90	591.40	0.58	28.00	0.32	14.00	Sandy Gravel
2	591.40	586.40	0.91	14.85	0.41	5.94	Clay
3	586.40	575.40	0.58	9.60	0.32	4.80	Sandy Gravel
4	575.40	570.40	1.31	18.00	0.72	9.00	Sandy Gravel
5	570.40	563.40	2.40	52.20	1.08	20.88	Till
6	563.40	556.40	1.52	30.40	0.84	15.20	Sandy Gravel
7	556.40	549.40	2.40	80.00	1.08	32.00	Till
8	549.40	544.40	2.40	80.00	1.08	32.00	Till
9	544.40	539.40	2.40	80.00	1.08	32.00	Till
10	539.40	535.40	2.40	80.00	1.08	32.00	Till

q_s = Unit Side Resistance in ksf; q_p = End Bearing Resistance in ksf

Table #11							
Unit Side and End Bearing Resistance North Abutment (Boring 2 NW Abut.)							
Layer	Top	Bottom	Nominal q_s (ksf)	Nominal q_p (ksf)	Factored q_s (ksf)	Factored q_p (ksf)	Description
1	594.80	583.30	0.61	9.90	0.27	3.96	Clay
2	583.30	574.30	0.61	9.60	0.34	4.80	Sandy Gravel
3	574.30	569.30	1.38	22.50	0.62	9.00	Clay
4	569.30	554.30	2.24	41.85	1.01	16.74	Clay
5	554.30	540.80	2.39	80.00	1.08	32.00	Till
q_s = Unit Side Resistance in ksf				q_p = End Bearing Resistance in ksf			

Table #12							
Unit Side and End Bearing Resistance Pier #1 (Boring 1A S. Pier)							
Layer	Top	Bottom	Nominal q_s (ksf)	Nominal q_p (ksf)	Factored q_s (ksf)	Factored q_p (ksf)	Description
1	573.70	568.70	1.13	18.45	0.51	7.38	Clay
2	568.70	565.70	2.03	35.10	0.91	14.04	Clay
3	565.70	562.20	0.88	14.40	0.40	5.76	Clay
4	562.20	556.70	1.98	45.60	1.09	22.80	Sandy Gravel
5	556.70	550.70	2.40	80.00	1.08	32.00	Till
6	550.70	545.70	2.40	80.00	1.08	32.00	Till
7	545.70	540.70	2.40	80.00	1.08	32.00	Till
8	540.70	535.70	2.40	80.00	1.08	32.00	Till
9	535.70	530.70	2.40	80.00	1.08	32.00	Till
10	530.70	525.70	2.01	35.10	0.90	14.04	Clay
11	525.70	521.20	2.40	80.00	1.08	32.00	Till
q_s = Unit Side Resistance in ksf				q_p = End Bearing Resistance in ksf			

Table #13							
Unit Side and End Bearing Resistance Pier #2 (Boring 2A N. Pier)							
Layer	Top	Bottom	Nominal q_s (ksf)	Nominal q_p (ksf)	Factored q_s (ksf)	Factored q_p (ksf)	Description
1	574.10	569.60	1.16	18.90	0.52	7.56	Clay
2	569.60	567.10	2.40	59.40	1.08	23.76	Till
3	567.10	564.60	2.30	44.10	1.04	17.64	Clay
4	564.60	559.60	0.22	3.60	0.10	1.44	Clay
5	559.60	554.60	2.40	62.10	1.08	24.84	Clay
6	554.60	549.60	2.40	80.00	1.08	32.00	Till
7	549.60	544.60	2.40	80.00	1.08	32.00	Till
8	544.60	539.60	2.40	80.00	1.08	32.00	Till
9	539.60	536.10	1.84	30.60	0.83	12.24	Clay
q_s = Unit Side Resistance in ksf				q_p = End Bearing Resistance in ksf			

Table #14							
Unit Side and End Bearing Resistance Pier #2 (Boring 3B N. Pier)							
Layer	Top	Bottom	Nominal q_s (ksf)	Nominal q_p (ksf)	Factored q_s (ksf)	Factored q_p (ksf)	Description
1	575.50	571.50	0.80	12.00	0.44	6.00	Sandy Gravel
2	571.50	569.00	1.00	16.20	0.45	6.48	Clay
3	569.00	564.50	2.40	59.40	1.08	23.76	Till
4	564.50	558.00	1.22	48.00	0.67	24.00	Sandy Gravel
5	558.00	554.00	2.40	48.60	1.08	19.44	Till
6	554.00	540.50	2.40	80.00	1.08	32.00	Till
q_s = Unit Side Resistance in ksf				q_p = End Bearing Resistance in ksf			

Lateral Loading

Soil inputs have been provided to facilitate a more detailed analysis as requested by the structural designer.

Soil Parameters										
Substructue Unit	Layer	Elevation		Unit Weight		Cohesion	ϕ	k	e_{50}	Description
		Top	Bottom	(pcf)	(pci)	(psi)	(deg)	(pci)		
South Abutment Boring (1 SE Abut.)	1	594.7	592.4	115	0.0666		30	29		Sand
	2	592.4	586.4	120	0.0694	11.46		550	0.0068	Silty Clay
	3	586.4	575.4	115	0.0666		30	20		Sand
	4	575.4	570.4	120	0.0694		33	33		Sand
	5	570.4	563.4	125	0.0723	38.89		1865	0.0041	Clay Till
	6	563.4	556.4	130	0.0752		40	125		Sandy Gravel
	7	556.4	535.4	130	0.0752	73.3		2000	0.003	Clay Till
	8	535.4	531.4	130	0.0752		40	125		Sandy Gravel
	9	531.4	520.9	130	0.0752	75.3		200	0.003	Clay Till
Pier 1 Boring (1A S. Pier)	1	575.5	573.1	120	0.0694		33	25		Sand
	2	573.1	562.2	120	0.0694	16.7		800	0.0058	Clay Till
	3	562.2	556.7	125	0.0723		36	80		Sandy Gravel
	4	556.7	535.7	130	0.0752	77.1		2000	0.003	Clay Till
	5	535.7	527.7	125	0.0723	33.7		1615	0.0044	Clay Till
	6	527.7	523.7	130	0.0752	77.1		2000	0.003	Clay Till
	7	523.7	514.2	130	0.0752		40	125		Sandy Gravel
Pier 1 Boring (1B S. Pier)	1	575.5	573.8	120	0.0694		33	39		Sandy Gravel
	2	573.8	562.8	125	0.0723	30.6		1465	0.0045	Clay Till
	3	562.8	557.1	130	0.0752		40	112		Sandy Gravel
	4	557.1	550.6	130	0.0752	66.7		2000	0.003	Clay Till
	5	550.6	545.6	125	0.0723	37.5		1799	0.0042	Clay Till
	6	545.6	532.1	130	0.0752	53.2		200	0.003	Clay Till
	7	532.1	529.1	115	0.0666		30	23		Sand
Pier 2 Boring (2A N. Pier)	1	575.5	569.6	120	0.0694	14.6		700	0.0062	Clay
	2	569.6	564.6	125	0.0723	39.9		1915	0.0041	Clay Till
	3	564.6	559.6	115	0.0666	2.8		35	0.019	Clay
	4	559.6	554.6	125	0.0723	47.9		2000	0.0038	Clay Till
	5	554.6	539.6	130	0.0752	82.4		2000	0.003	Clay Till
	6	539.6	536.1	125	0.0723	23.6		1132	0.005	Clay Till
	7	536.1	523.1	130	0.0752		40	125		Sandy Gravel
Pier 2 Boring (3B N. Pier)	1	575.5	571.5	115	0.0666		30	23		Sand
	2	571.5	569.0	120	0.0694	12.5		600	0.0066	Clay
	3	569.0	564.5	125	0.0723	45.5		200	0.004	Clay Till
	4	564.5	558.0	125	0.0723		36	60		Sand
	5	558.0	553.5	125	0.0723	37.5		1799	0.0042	Clay Till
	6	553.5	540.5	130	0.0752	77		200	0.003	Clay Till
	7	540.5	537.5	125	0.0723		36	60		Sand
North Abutment Boring (3B N. Pier)	1	594.8	583.3	120	0.0694	7.6		287	0.0086	Clay
	2	583.3	574.3	115	0.0666		30	20		Sandy Gravel
	3	574.3	569.3	125	0.0723	17.4		833	0.0057	Clay Till
	4	569.3	554.3	125	0.0723	31.9		1532	0.0045	Clay Till
	5	554.3	540.8	130	0.0752	68.75		2000	0.003	Clay Till
	6	540.8	517.8	130	0.0752		40	125		Sand

ϕ = phi angle
k = subgrade modulus
E_{50} = strain at 50% deflection in p-y curve

Losses

Because there is no change in the roadway profile grade, there are no Downdrag (DD) losses.

There are no scour losses at the abutments. For the piers, the driving elevation of the piles is 575.50' with the 500 year scour elevation is +/-574.00. This 1.50' of scour loss is minor and was therefore disregarded in the analysis.

For drilled shafts at the piers, the drilling elevation would be +/-585.80' for both piers with the same 500 year scour elevation of 574.00'. For this analysis, the unit side resistance and end bearing was calculated starting at elevation +/-575.50' and below. This 1.50' of scour loss is minor and was therefore disregarded in the analysis.

Liquefaction losses were not analyzed for Seismic Performance Zone (SPZ) 1.

Approach Pavement

Foundation conditions beneath proposed approach pavement footings have been reviewed, based on available boring data, the available bearing capacity is greater than required. For structure replacement projects, experience indicates approach pavement footings do not experience excessive settlement when there is no new fill beneath the footing, and it is constructed on undisturbed soil. No remedial action is required.

Construction Considerations

Stage Construction: This project will be constructed under a road closure.

Ground Improvement: No ground improvement is required.

Foundation Construction: If piles are utilized, a test pile is recommended at each abutment and pier, located farthest from the boring locations. Hard driving will be encountered for Metal Shell Pile at elevations +/-553.50' to -/+557.10'. The maximum Nominal Required Bearing for metal shell pile should be reduced to prevent damage during driving, and pile shoes are required.

The Estimated Water Surface Elevation (EWSE) is 584.70' the existing ground elevation is +/-588.00' for Pier #1 and #2. The elevation at which the piling will be driven is +/-575.50', pier borings indicate a Sandy Gravel layer at the driving elevation making it difficult to dewater through reasonable pumping efforts. Based on this information a Cofferdam Type 2 is warranted for both piers if founded on piling, see 2012 BBS manual section 2.3.6.4.2.

If drilled shafts are anticipated to be constructed, then temporary casings should be utilized. Permanent casing will reduce the unit side resistance of the shaft and are not recommended. There will likely be some seepage during drilling and construction of the shafts. Drilled shafts would allow top down construction, thus eliminating the need for a cofferdam. There is no evidence of debris build up under the existing bridge; thus eliminating the need for web walls. This foundation option would greatly reduce construction time.

It is recommend moving the proposed abutment out an additional 5.0 ft. to miss any potential obstructions from the existing structure (typical District 6 recommendation).

The following is a list of spreadsheets and software programs that were used in the geotechnical analysis:

- Seismic Site Class Determination Spreadsheet by BBS (Modified 12/10/10)
- AASHTO Guide Specifications for LRFD Seismic Bridge Design 2007
- *AllPile* by Civil Tech
- New Supplement (not-yet-published) to the 2012 IDOT Integral Abutment Bridge Policy, ABD 12.3

Benchmarks: BM #13 Chiseled "□" on Northwesterly parapet wall of I-55 Southbound Lane Structure over Kickapoo Creek, Station 10283+65.73/65.48' RT., NAVD 88 Elevation = 607.00.
BM #14 Chiseled "□" on Southeast wingwall of Existing Structure No. 054-0002, Station 10280+36.48/16.69' RT., NAVD 88 Elevation = 605.90.
BM #14A Chiseled "□" on Southwest wingwall of Railroad Bridge over Kickapoo Creek, Station 10280+15.72/153.65' LT., NAVD 88 Elevation = 605.62.

Existing Structure: Structure No. 054-0002, originally built in 1954 as FAP 5, Section 21, RB2. In 1989, the expansion joints and parts of the abutments were replaced as FAP 5, Section 21RB-21. The superstructure consists of a continuous five-span, haunched, reinforced concrete girder bridge with a 7" concrete slab. The substructure consists of concrete pile bent abutments supported by precast concrete piles and solid wall pile bent piers supported by untreated timber piles. The back-to-back of abutments dimension measures 306'-10" and the out-to-out dimension measures 34'-4". The span lengths are 50'-2", 67'-0", 68'-0", 67'-0" and 50'-2" (℄ bearing to ℄ bearing) with a 22° right forward skew. Traffic will be detoured during construction.

No Salvage.

HIGHWAY CLASSIFICATION

F.A.S. Route 1773 (FR I-55 West)
Functional Class: Major Collector, Non Urban
A.D.T.: 1700 (2011), 1707 (2032)
D.H.V.: 86
A.D.T.T.: 7%
Design Speed: 60 M.P.H.
Posted Speed: 55 M.P.H.
Two Way Traffic
Directional Distribution: 50/50

DESIGN SPECIFICATIONS

2012 AASHTO LRFD Bridge Design Specifications, Customary U.S. Units, 6th Edition, with 2013 Interims

DESIGN STRESSES

FIELD UNITS:

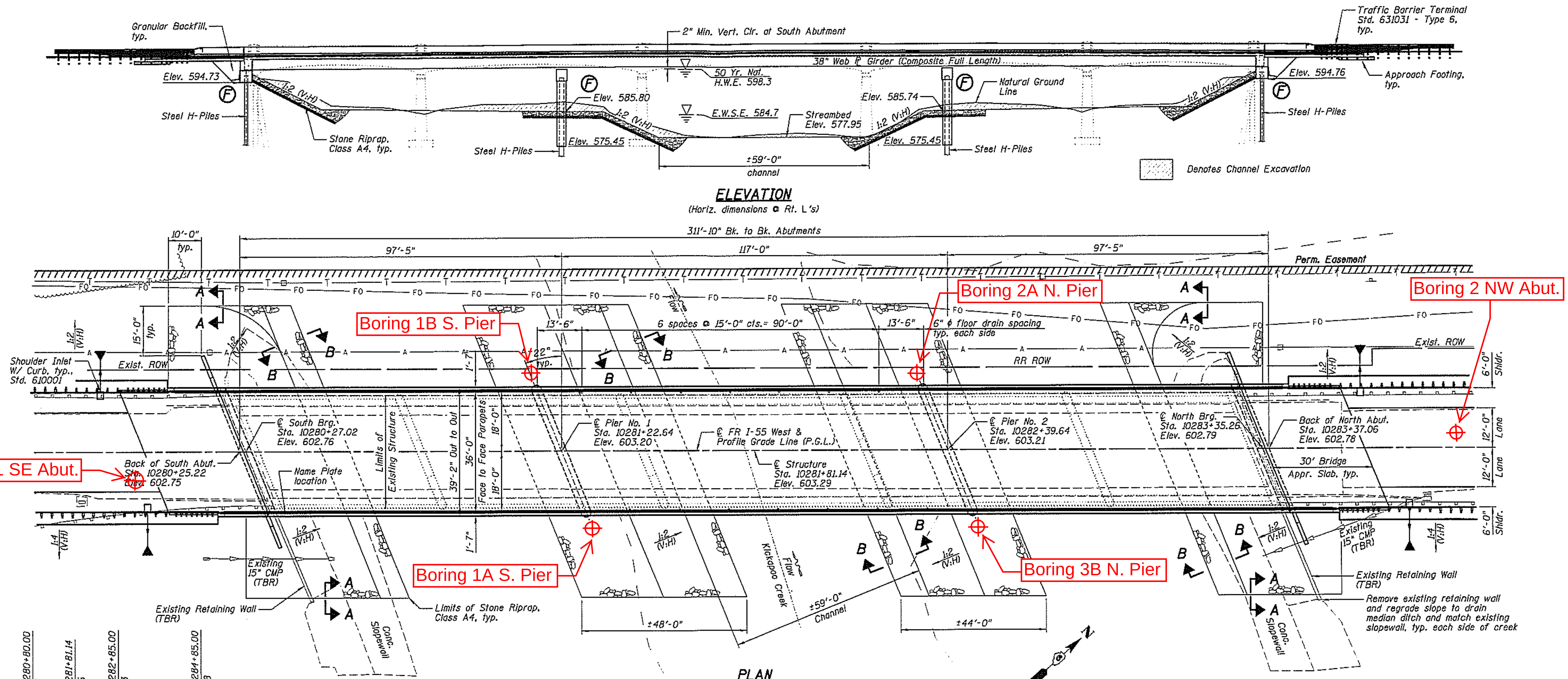
f'c = 3,500 psi
fy = 60,000 psi (Reinforcement)
fy = 50,000 psi (AASHTO M270 Grade 50W)

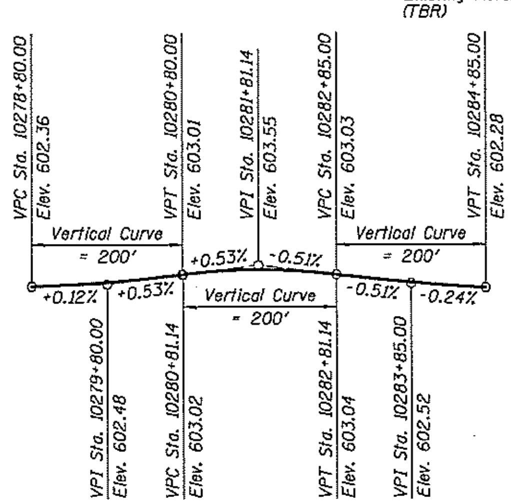
LOADING HL-93

Allow 50#/sq. ft. for future wearing surface.

SEISMIC DATA

Seismic Performance Zone (SPZ) =
Design Spectral Acceleration at 1.0 sec. (SD1) = g
Design Spectral Acceleration at 0.2 sec. (SDs) = g
Soil Site Class =





PROFILE GRADE
(Along ℄ Roadway)

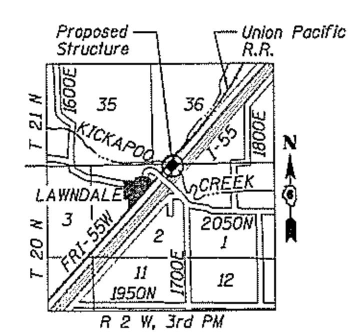
WATERWAY INFORMATION

Drainage Area = 284 Sq. Mi.	Existing Low Grade Elev. 602.4 @ Sta. 10283+00.00 Proposed Low Grade Elev. 602.7 @ Sta. 10283+36.14								
Flood	Freq. Yr.	Q C.F.S.	Opening Sq. Ft. Exist.	Opening Sq. Ft. Prop.	Nat. H.W.E.	Head - Ft. Exist.	Head - Ft. Prop.	Headwater El. Exist.	Headwater El. Prop.
	10	10,460	2,890	3,119	595.7	0.3	0.3	596.0	596.0
Design	50	16,360	3,911	3,900	598.3	0.4	0.5	598.7	598.8
Base	100	19,010	3,911	3,961	599.2	0.4	0.6	599.6	599.7
Overtopping									
Max. Calc.	500	25,310	3,911	3,961	600.8	0.0	0.7	600.5	601.5

10 Yr. Velocity = 2.9 ft/sec. (Existing)
10 Yr. Velocity = 2.9 ft/sec. (Proposed)

DESIGN SCOUR ELEVATION TABLE

Design Scour Elevations (ft.)				
	S. Abut.	Pier 1	Pier 2	N. Abut.
Q100	594.73	579.06	579.00	594.76
Q500	594.73	574.02	573.96	594.76



LOCATION SKETCH

NOTE:
See Sheet 2 for Section A-A and B-B.

GENERAL PLAN
FR I-55 WEST OVER KICKAPOO CREEK
F.A.S. 1773 - SECTION 21ACB
LOGAN COUNTY
STATION 10281+81.14
STRUCTURE NO. 054-0516

Farnsworth GROUP, INC.
2709 McGraw Drive
Bloomington, Illinois 61704
309/663-8435, 309/663-1571 fax

DESIGNED - JCZ	REVISED	
CHECKED - JML	REVISED	
DRAWN - DJM/JWK	REVISED	
DATE - 02/07/14	CHECKED - MSW	REVISED

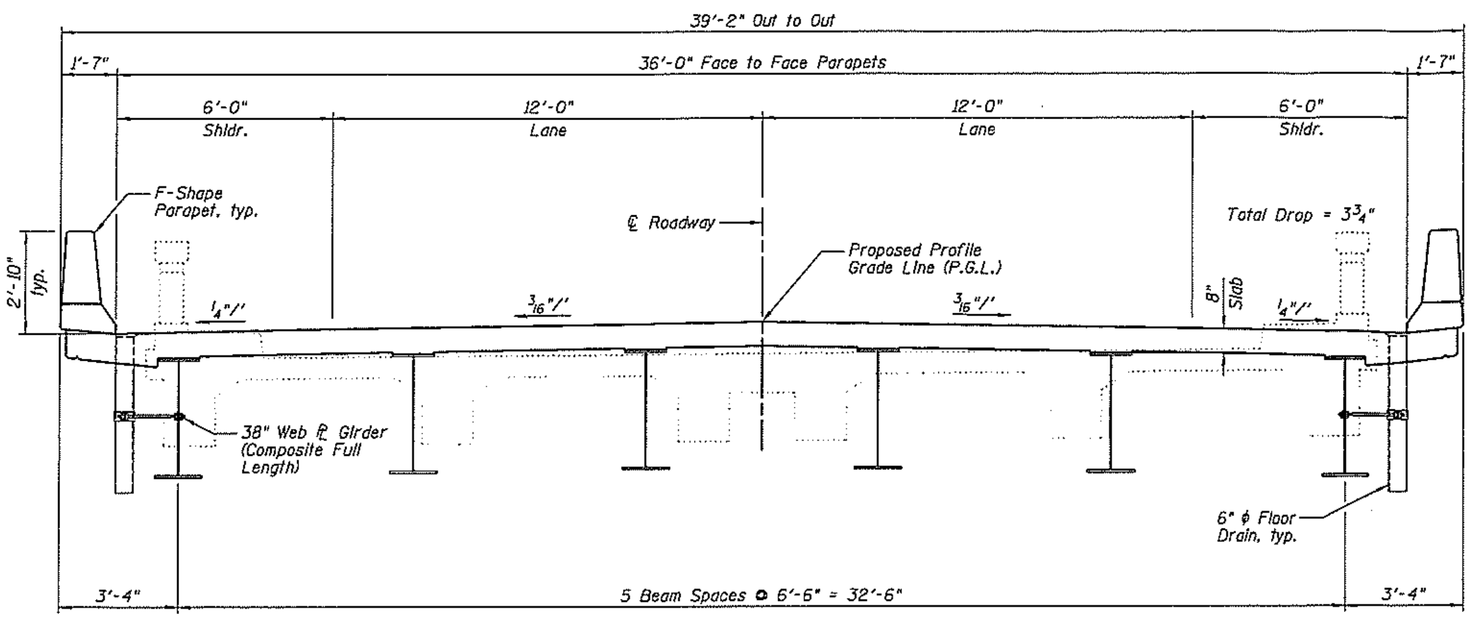
STATE OF ILLINOIS
DEPARTMENT OF TRANSPORTATION

SHEET NO. OF SHEETS

F.A.S. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
1773	21ACB	LOGAN	2	1
		CONTRACT NO. 72C33		

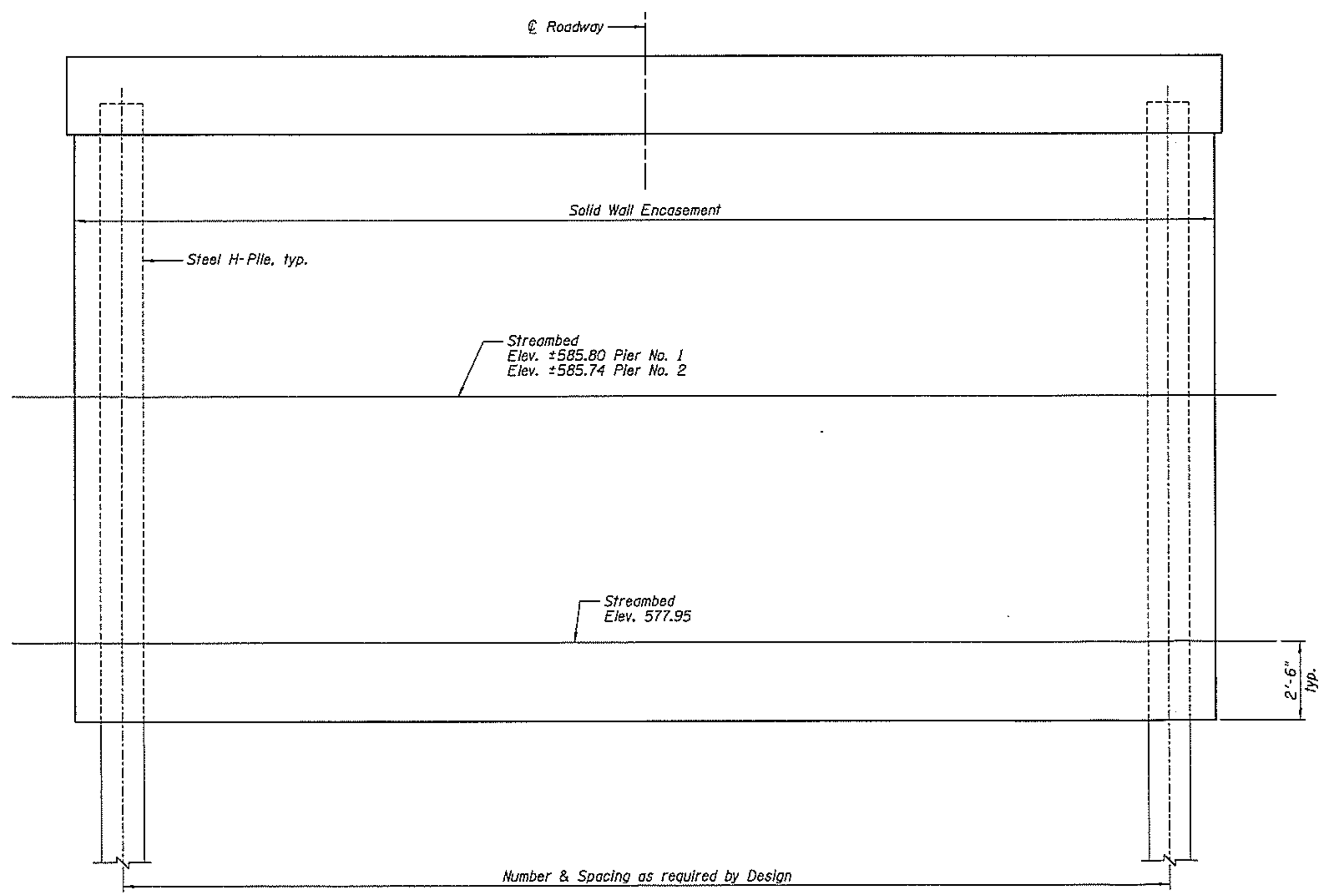
ILLINOIS FED. AID PROJECT

24-8346

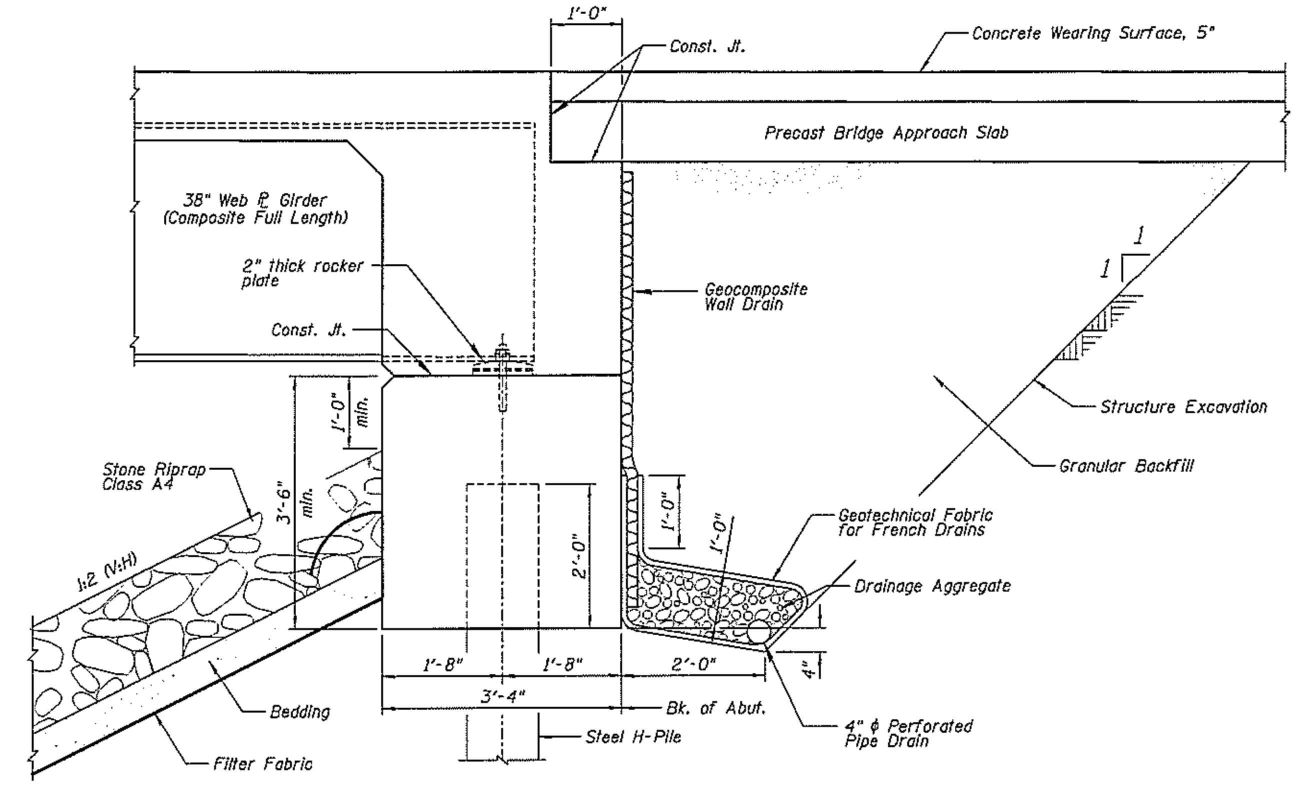


CROSS SECTION

(Looking North @ ℄ of Bridge)

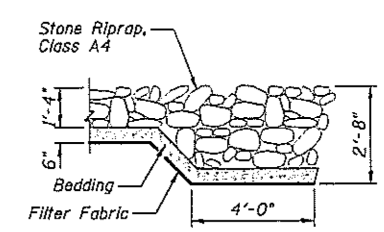


PIER SKETCH



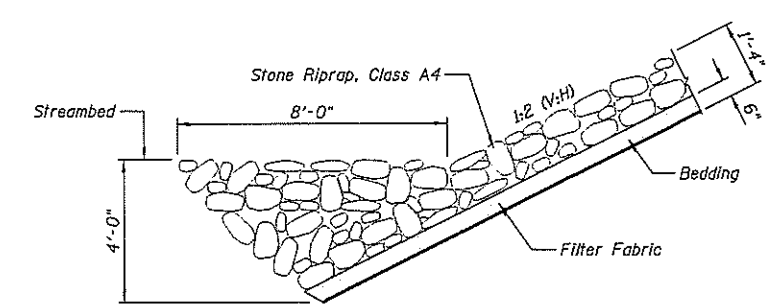
SECTION THRU ABUTMENT

(Horiz. dimensions @ Rt. ∟'s)



SECTION A-A

(Horiz. dimensions @ Rt. ∟'s)



SECTION B-B

(Horiz. dimensions @ Rt. ∟'s)

DETAILS
STRUCTURE NO. 054-0516

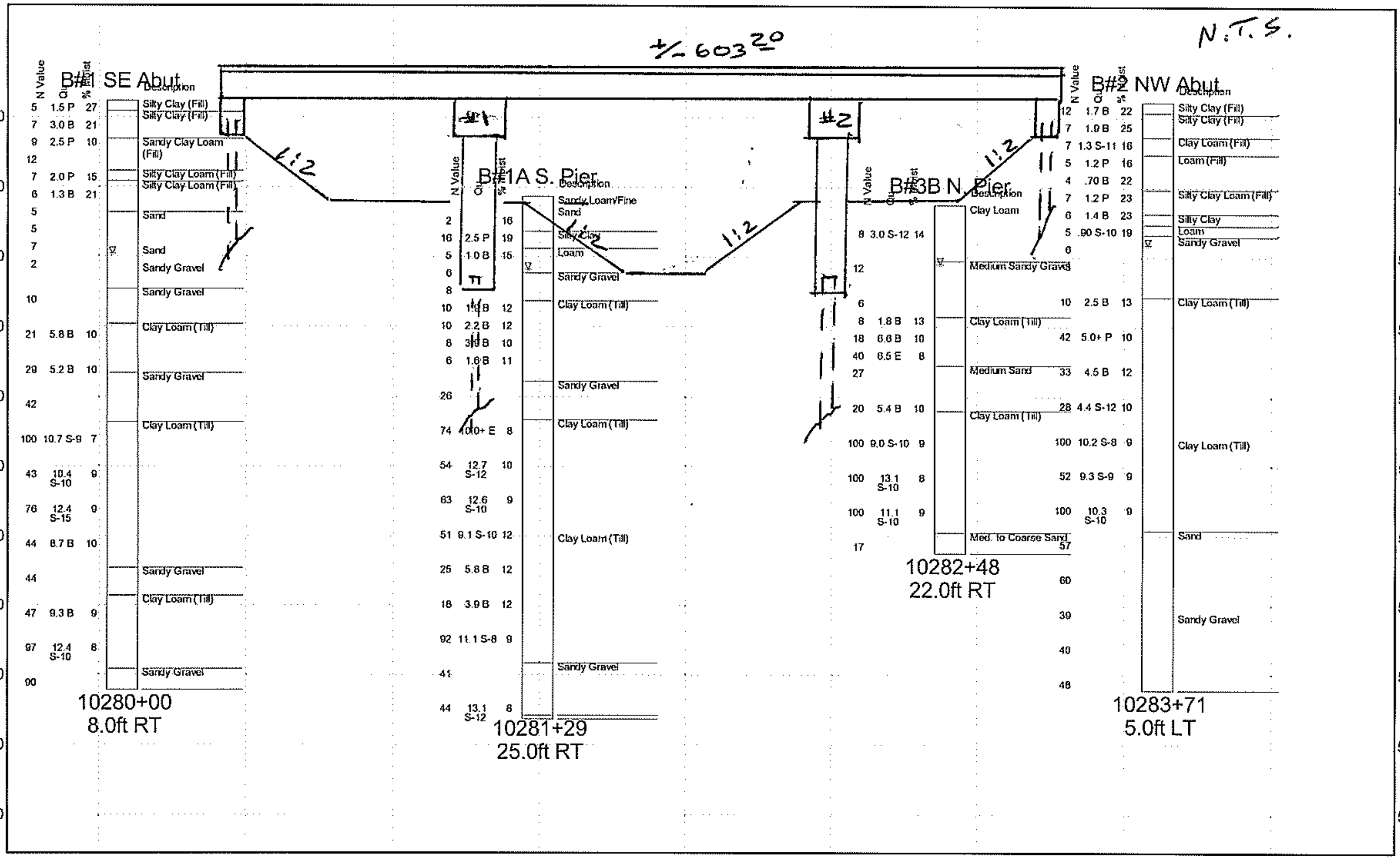
Farnsworth GROUP, INC. 2709 McGraw Drive Bloomington, Illinois 61704 309/663-8435, 309/663-1571 fax		DESIGNED - JCZ	REVISED	STATE OF ILLINOIS DEPARTMENT OF TRANSPORTATION
		CHECKED - JML	REVISED	
		DRAWN - DJM/JWK	REVISED	
	DATE - 02/07/14	CHECKED - MSW	REVISED	

SHEET NO. OF SHEETS

F.A.S. RTE.	SECTION	COUNTY	TOTAL SHEETS	SHEET NO.
1773	21ACB	LOGAN	2	2
		CONTRACT NO. 72C33		
	ILLINOIS FED. AID PROJECT			

24-834

Structure Number EX SN 054-0002   PR SN 054-0516 Over Kickapoo Creek
Located in the NE 1/4 of Section 2, Township 20N, Range 2W of the 3 P.M.



NOT TO HORIZONTAL SCALE

Illinois Department of Transportation
Division of Highways
IDOT

VARIATIONS IN SUBSURFACE CONDITIONS MAY EXIST BETWEEN BORINGS

Groundwater
First Encounter
Completion
after (refer to log) hours

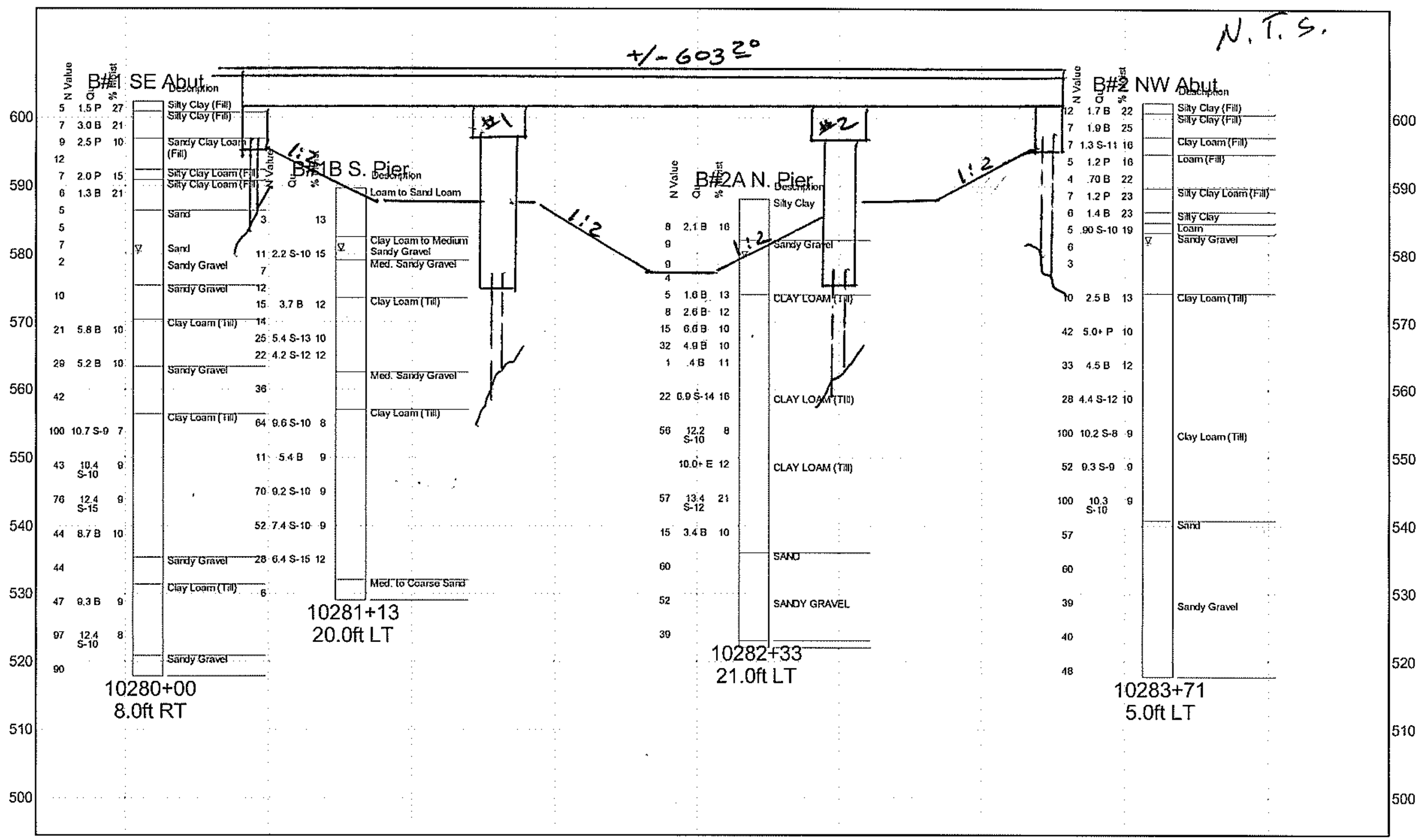
Abbreviations
WOH - Sampler Advanced by Weight of Hammer, WOP - Weight of Pipe
B.S. - Before Seating

SUBSURFACE DATA PROFILE

Route: FR I-55
Section: 21 ACB
County: Logan

SUBSURFACE DATA PROFILE 054-0516.GPJ D6TEMPLT.GDT 4/24/15

Structure Number EX SN 054-0002   PR SN 054-0516 Over Kickapoo Creek
Located in the NE 1/4 of Section 2, Township 20N, Range 2W of the 3 P.M.



NOT TO HORIZONTAL SCALE

VARIATIONS IN SUBSURFACE CONDITIONS MAY EXIST BETWEEN BORINGS

Groundwater
First Encounter
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Abbreviations
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Illinois Department of Transportation
Division of Highways
IDOT

SUBSURFACE DATA PROFILE

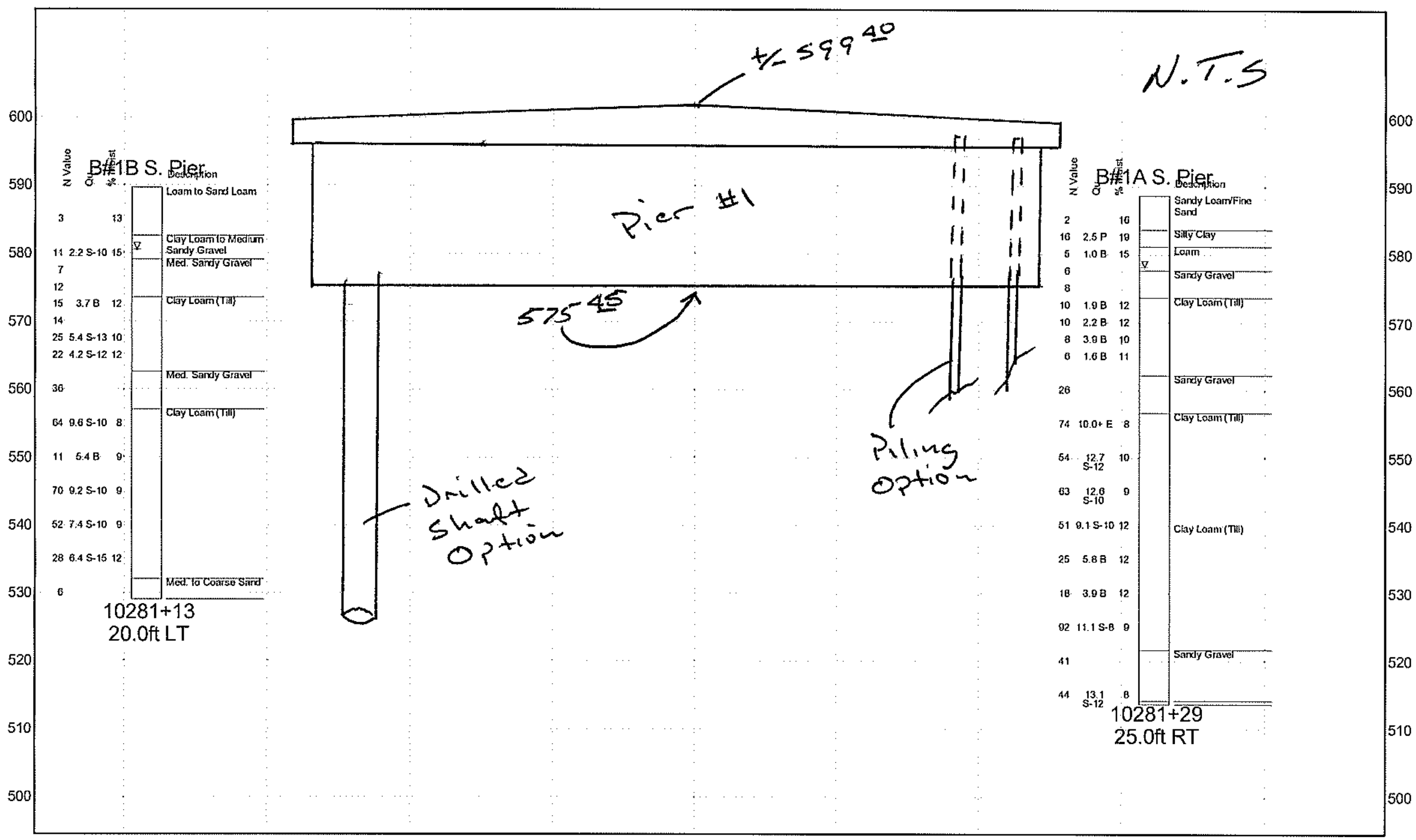
Route: FR I-55

Section: 21 ACB

County: Logan

SUBSURFACE DATA PROFILE 054-0516.GPJ D6TEMPLT.GDT 4/24/15

Structure Number EX SN 054-0002   PR SN 054-0516 Over Kickapoo Creek
Located in the NE 1/4 of Section 2, Township 20N, Range 2W of the 3 P.M.



NOT TO HORIZONTAL SCALE

Illinois Department of Transportation
Division of Highways
IDOT

VARIATIONS IN SUBSURFACE CONDITIONS MAY EXIST BETWEEN BORINGS

Groundwater
∇ First Encounter
∇ Completion
∇ after (refer to log) hours

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WOH - Sampler Advanced by Weight of Hammer, WOP - Weight of Pipe
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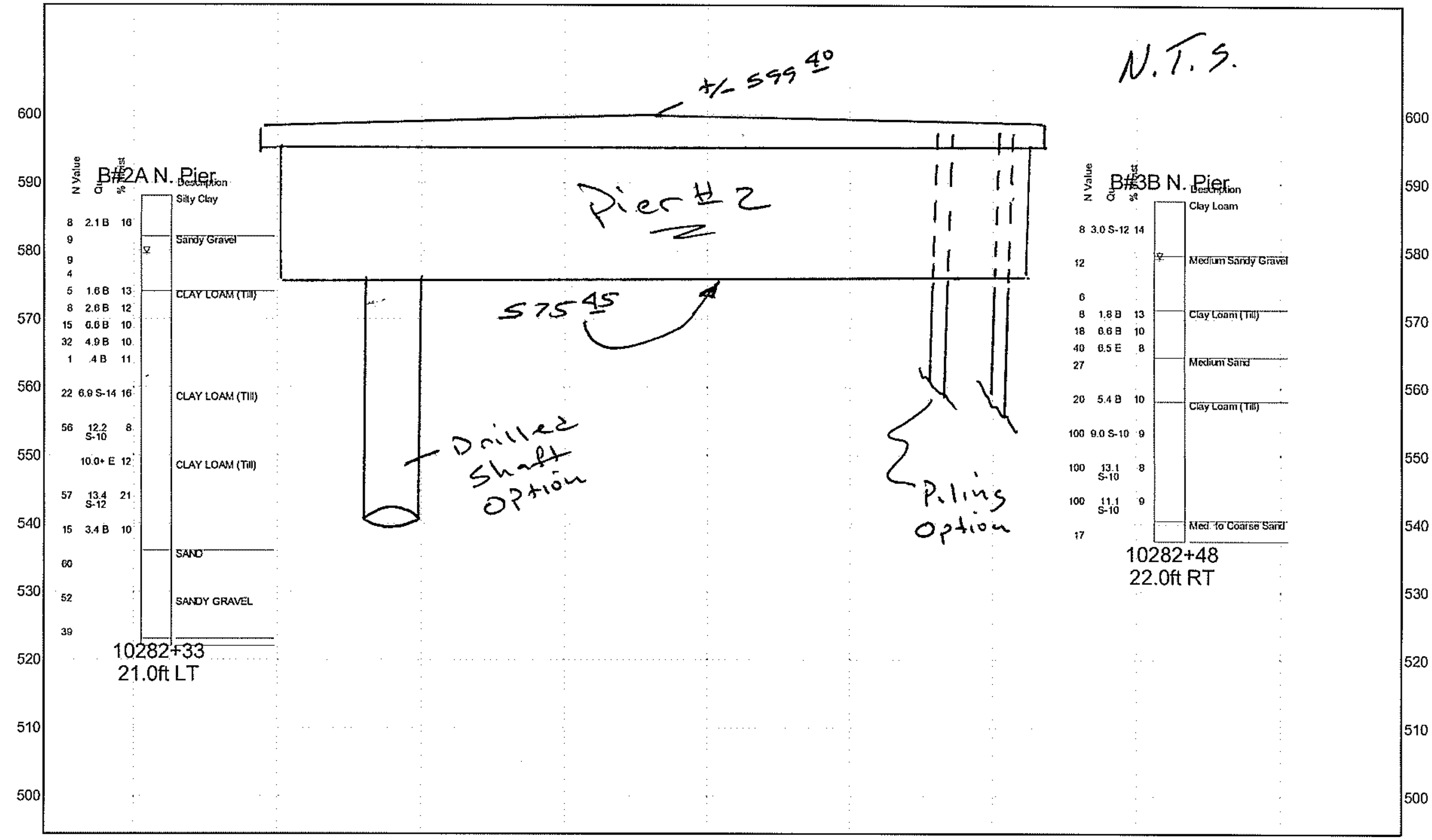
SUBSURFACE DATA PROFILE

Route: FR I-55
Section: 21 ACB
County: Logan

SUBSURFACE DATA PROFILE 054-0516.GPJ D6TEMPLT.GDT 4/24/15

Structure Number EX SN 054-0002   PR SN 054-0516 Over Kickapoo Creek
Located in the NE 1/4 of Section 2, Township 20N, Range 2W of the 3 P.M.



NOT TO HORIZONTAL SCALE

Illinois Department of Transportation
Division of Highways
IDOT

VARIATIONS IN SUBSURFACE CONDITIONS MAY EXIST BETWEEN BORINGS

Groundwater
First Encounter
Completion
after (refer to log) hours

Abbreviations
WOH - Sampler Advanced by Weight of Hammer, WOP - Weight of Pipe
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SUBSURFACE DATA PROFILE

Route: FR I-55

Section: 21 ACB

County: Logan

SUBSURFACE DATA PROFILE 054-0516.GPJ D6TEMPLT.GDT 4/24/15

Illinois Department of Transportation
Division of Highways
IDOT

SOIL BORING LOG

Date 2/16/12

ROUTE FR I-55 **DESCRIPTION** Over Kickapoo Creek **LOGGED BY** M. Tappan

SECTION 21 ACB **LOCATION** NE 1/4, **SEC.** 2, **TWP.** 20N, **RNG.** 2W, 3 **PM**

COUNTY Logan **DRILLING METHOD** HSA **HAMMER TYPE** 140 # AUTO

STRUCT. NO. EX SN 054-0002 / PR SN 054-0516
Station 10281+81

BORING NO. 1 SE Abut.
Station 10280+00
Offset 8.0ft RT
Ground Surface Elev. 602.4 ft

Surface Water Elev. 580.2 ft
Stream Bed Elev. 579.1 ft

Groundwater Elev.:
First Encounter 580.4 ft
Upon Completion Washed ft
After Plugged **Hrs.** ft

Description	Elev. (ft)	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Brown and Gray Moist CA-6 to Dark Gray Moist SILTY CLAY (Fill) Poor Recovery	600.90		1		
			3	1.5	27
Light Olive Gray Moist SILTY CLAY (Fill)			2	P	
			1		
			3	3.0	21
			4	B	
	596.90	-5	Silty Clay		
Brown and Gray Moist SANDY CLAY LOAM (Fill) Sample Broken			3		
			5	2.5	10
			4	P	
			3		
			6		
	592.40		6		
Brown and Olive Gray Moist LOAM to Very Dark Gray Moist SILTY CLAY LOAM (Fill)		-10	1		
	590.90		3	2.0	15
Brown and Dark Gray Moist SILTY CLAY LOAM (Fill)			4	P	
			1		
			3	1.3	21
			3	B	
	586.40	-15	1		
Brown Moist Medium SAND			2		
			3		
			1		
Tan			2		
		-20	3		

Description	Elev. (ft)	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Brown Moist Medium SAND *(continued)*			1		
Gray Very Moist Medium to Coarse SAND			4		
			3		
			1		
Gray Fine SANDY GRAVEL			1		
			1		
	575.40	-25			
Gray Medium SANDY GRAVEL					
Washed			3		
			4		
			6		
	570.40	-30			
Gray Moist CLAY LOAM (Till) 6" Seam Gray Medium SANDY GRAVEL at 39' Washed			5		
			10	5.8	10
			11	B	
Washed		-35	7		
	563.40		11	5.2	10
Gray Medium SANDY GRAVEL			18	B	
		-40			

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer, E-Estimated)
Abbreviations W.O.H - Sampler Advanced By Weight of Hammer, W.O.P - Advanced by Weight of Pipe, B.S. - Before Seating
The SPT (N value) is the sum of the last two blow values in each sampling zone (AASHTO T206) BBS, from 137 (Rev. 8-99)

Illinois Department of Transportation
Division of Highways
IDOT

SOIL BORING LOG

Date 2/16/12

ROUTE FR I-55 **DESCRIPTION** Over Kickapoo Creek **LOGGED BY** M. Tappan

SECTION 21 ACB **LOCATION** NE 1/4, **SEC.** 2, **TWP.** 20N, **RNG.** 2W, 3 **PM**

COUNTY Logan **DRILLING METHOD** HSA **HAMMER TYPE** 140 # AUTO

STRUCT. NO. EX SN 054-0002 / PR SN 054-0516
Station 10281+81

BORING NO. 1 SE Abut.
Station 10280+00
Offset 8.0ft RT
Ground Surface Elev. 602.4 ft

Surface Water Elev. 580.2 ft
Stream Bed Elev. 579.1 ft

Groundwater Elev.:
First Encounter 580.4 ft
Upon Completion Washed ft
After Plugged **Hrs.** ft

Description	DEPTH (ft)	BLOWS /6"	UCS Qu (tsf)	MOIST (%)
Gray Medium SANDY GRAVEL *(continued)*				
Washed		11		
		19		
		23		
	-45			
556.40				
Gray Dry CLAY LOAM (Till) Drilled Hard at 46.0'				
Washed		28		
		100	10.7	7
		6"	S-9	
	-50			
Washed		22		
		43	10.4	9
		57/6"	S-10	
	-55			
Washed		9		
		29	12.4	9
		47	S-15	
	-60			

Description	DEPTH (ft)	BLOWS /6"	UCS Qu (tsf)	MOIST (%)
Gray Dry CLAY LOAM (Till) Drilled Hard at 46.0' *(continued)*				
Washed		11		
		22	8.7	10
		22	B	
	-65			
535.40				
Gray Medium SANDY GRAVEL				
Washed		21		
		22		
		22		
	-70			
531.40				
Gray Moist CLAY LOAM (Till)				
Washed		5		
		20	9.3	9
		27	B	
	-75			
Washed		4		
		28	12.4	8
		69	S-10	
	-80			

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer, E-Estimated)
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Illinois Department of Transportation
Division of Highways
IDOT

SOIL BORING LOG

Date 2/16/12

ROUTE FR I-55 **DESCRIPTION** Over Kickapoo Creek **LOGGED BY** M. Tappan

SECTION 21 ACB **LOCATION** NE 1/4, **SEC.** 2, **TWP.** 20N, **RNG.** 2W, 3 **PM**

COUNTY Logan **DRILLING METHOD** HSA **HAMMER TYPE** 140 # AUTO

STRUCT. NO. EX SN 054-0002 / PR SN 054-0516
Station 10281+81

BORING NO. 1 SE Abut.
Station 10280+00
Offset 8.0ft RT
Ground Surface Elev. 602.4 ft

Surface Water Elev. 580.2 ft
Stream Bed Elev. 579.1 ft

Groundwater Elev.:
First Encounter 580.4 ft
Upon Completion Washed ft
After Plugged **Hrs.** ft

Description	DEPTH (ft)	BLOWS /6"	UCS Qu (tsf)	MOIST (%)
Gray Moist CLAY LOAM (Till) *(continued)*				
520.90				
Gray Medium SANDY GRAVEL Drilled Easy at 81.5				
Washed		21		
		42		
517.90		48		
Boring Completed	-85			
Ref. Sta. to Centerline of Ex. Structure=1028+81 Sta. Increase to North				
Ref. Elev. to BM 14=605.9				
	-90			
	-95			
	-100			

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer, E-Estimated)
Abbreviations W.O.H - Sampler Advanced By Weight of Hammer, W.O.P - Advanced by Weight of Pipe, B.S. - Before Seating
The SPT (N value) is the sum of the last two blow values in each sampling zone (AASHTO T206) BBS, from 137 (Rev. 8-99)

File Name S:\SOILS\GINT FILES\054 LOGAN\054-0516.GPJ Data Template D6TEMPLT.GDT Date Printed 8/27/14
Latitude 40.13.303N Longitude 89.16.636W Datum NAD83 Job Number D-96-008-09

Illinois Department of Transportation
Division of Highways
IDOT

SOIL BORING LOG

Date 5/28/14

ROUTE FR I-55 **DESCRIPTION** Over Kickapoo Creek **LOGGED BY** M. Tappan

SECTION 21 ACB **LOCATION** NE 1/4, **SEC.** 2, **TWP.** 20N, **RNG.** 2W, 3 **PM**

COUNTY Logan **DRILLING METHOD** HSA **HAMMER TYPE** 140 # AUTO

STRUCT. NO. EX SN 054-0002 / PR SN 054-0516
Station 10281+81

BORING NO. 1A S. Pier
Station 10281+29
Offset 25.0ft RT
Ground Surface Elev. 588.7 ft

Surface Water Elev. 580.2 ft
Stream Bed Elev. 579.1 ft

Groundwater Elev.:
▽ **First Encounter** 578.2 ft
▼ **Upon Completion** Washed ft
▼ **After** Plugged **Hrs.** ft

Description	Elev. (ft)	DEPTH (ft)	BLOWS /6"	UCS Qu (tsf)	MOIST (%)
Dark Gray Moist SANDY LOAM to Fine Dirty SAND			0 / 1 / 1		16
	583.70	-5			
Brown & Gray Moist SILTY CLAY			2 / 6 / 10	2.5 P	19
	581.20				
Very Dark Gray Moist LOAM			1 / 3 / 2	1.0 B	15
		-10			
▽	577.70				
Gray Medium SANDY GRAVEL -- Free H2O			1 / 3 / 3		
Gray Medium SANDY GRAVEL Washed			3 / 4 / 4		
	573.70	-15			
Gray Moist CLAY LOAM (Till)			1 / 4 / 6	1.9 B	12
			1 / 4 / 6	2.2 B	12
		-20			

Description	Elev. (ft)	DEPTH (ft)	BLOWS /6"	UCS Qu (tsf)	MOIST (%)
Gray Moist CLAY LOAM (Till) *(continued)*			1 / 3 / 5	3.9 B	10
			1 / 3 / 3	1.6 B	11
		-25			
	562.20				
Gray Medium to Coarse SANDY GRAVEL (Sandy Gravel in Augers 4' Washed)			10 / 10 / 16		
		-30			
	556.70				
Gray Dry CLAY LOAM (Till)			12 / 36 / 38	10.0+ E	8
		-35			
			8 / 22 / 32	12.7 S-12	10
		-40			

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer, E-Estimated)
Abbreviations W.O.H - Sampler Advanced By Weight of Hammer, W.O.P - Advanced by Weight of Pipe, B.S. - Before Seating
The SPT (N value) is the sum of the last two blow values in each sampling zone (AASHTO T206) BBS, from 137 (Rev. 8-99)

Illinois Department of Transportation
Division of Highways
IDOT

SOIL BORING LOG

Date 5/28/14

ROUTE FR I-55 **DESCRIPTION** Over Kickapoo Creek **LOGGED BY** M. Tappan

SECTION 21 ACB **LOCATION** NE 1/4, **SEC.** 2, **TWP.** 20N, **RNG.** 2W, 3 **PM**

COUNTY Logan **DRILLING METHOD** HSA **HAMMER TYPE** 140 # AUTO

STRUCT. NO. EX SN 054-0002 / PR SN 054-0516
Station 10281+81

BORING NO. 1A S. Pier
Station 10281+29
Offset 25.0ft RT
Ground Surface Elev. 588.7 ft

Surface Water Elev. 580.2 ft
Stream Bed Elev. 579.1 ft

Groundwater Elev.:
First Encounter 578.2 ft
Upon Completion Washed ft
After Plugged **Hrs.** ft

Description	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Gray Dry CLAY LOAM (Till) *(continued)*				
		8		
Gray Dry CLAY LOAM (Till) -- Rained Out Continued on 05/29/14		23	12.6	9
		40	S-10	
	-45			
		8		
Gray Dry CLAY LOAM (Till) -- Resumed Drilling on 05/29/14		22	9.1	12
		29	S-10	
	-50			
		4		
Gray Dry CLAY LOAM (Till)		11	5.8	12
		14	B	
	-55			
		4		
Gray Dry CLAY LOAM (Till) Washed		5	3.9	12
		13	B	
	-60			

Description	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Gray Dry CLAY LOAM (Till) *(continued)*				
		9		
Gray Dry CLAY LOAM (Till)		36	11.1	9
		56	S-8	
	-65			
521.70				
Gray Dirty Medium SANDY GRAVEL Washed (Drilled Easy at 67')		4		
		10		
		31		
	-70			
		15		
Gray Medium SANDY GRAVEL with Gray Dry CLAY LOAM (Till) at 74'-74.5'		44	13.1	8
514.20		56/4"	S-12	
513.70	-75			
Boring Complete				
	-80			

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer, E-Estimated)
Abbreviations W.O.H - Sampler Advanced By Weight of Hammer, W.O.P - Advanced by Weight of Pipe, B.S. - Before Seating
The SPT (N value) is the sum of the last two blow values in each sampling zone (AASHTO T206) BBS, from 137 (Rev. 8-99)

Illinois Department of Transportation
Division of Highways
IDOT

SOIL BORING LOG

Date 7/9/14

ROUTE FR I-55 **DESCRIPTION** Over Kickapoo Creek **LOGGED BY** M. Tappan

SECTION 21 ACB **LOCATION** NE 1/4, **SEC.** 2, **TWP.** 20N, **RNG.** 2W, 3 **PM**

COUNTY Logan **DRILLING METHOD** HSA **HAMMER TYPE** 140 # AUTO

STRUCT. NO. EX SN 054-0002 / PR SN 054-0516
Station 10281+81

BORING NO. 1B S. Pier
Station 10281+13
Offset 20.0ft LT
Ground Surface Elev. 589.6 ft

Surface Water Elev. 580.2 ft
Stream Bed Elev. 579.1 ft

Groundwater Elev.:
First Encounter 580.6 ft
Upon Completion Washed ft
After Plugged **Hrs.** ft

Description	DEPTH (ft)	BLOWS /6"	UCS Qu (tsf)	MOIST (%)
Gray Moist LOAM to SAND LOAM (Sample Broken)				
		0		
	-5	1		13
		2		
582.60				
Dark Gray Moist CLAY LOAM with Dark Gray Dirty Medium SANDY GRAVEL at 10.5' Free Water ▽				
		4		
	-10	5	2.2	15
579.10		6	S-10	
Gray Medium SANDY GRAVEL				
		3		
		4		
		3		
Hit Limestone Cobble/Boulder at 15.5'. Moved boring to East Washed		3		
	-15	4		
		8		
573.60				
Gray Moist CLAY LOAM (Till)				
		4		
		6	3.7	12
		9	B	
No Recovery		4		
	-20	6		
Gray Moist CLAY LOAM (Till) *(continued)*		8		
		5		
		11	5.4	10
		14	S-13	
		5		
	-25	10	4.2	12
		12	S-12	
562.60				
Gray Medium SANDY GRAVEL Washed				
		10		
	-30	18		
		18		
557.10				
Gray Dry CLAY LOAM (Till) Drilled Hard at 32.5' Washed				
		6		
	-35	25	9.6	8
		39	S-10	
Gray Dry CLAY LOAM (Till) with Gray Medium SANDY GRAVEL		8		
	-40	6	5.4	9

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer, E-Estimated)
Abbreviations W.O.H - Sampler Advanced By Weight of Hammer, W.O.P - Advanced by Weight of Pipe, B.S. - Before Seating
The SPT (N value) is the sum of the last two blow values in each sampling zone (AASHTO T206) BBS, from 137 (Rev. 8-99)

Illinois Department of Transportation
Division of Highways
IDOT

SOIL BORING LOG

Date 7/9/14

ROUTE FR I-55 DESCRIPTION Over Kickapoo Creek LOGGED BY M. Tappan

SECTION 21 ACB LOCATION NE 1/4, SEC. 2, TWP. 20N, RNG. 2W, 3 PM

COUNTY Logan DRILLING METHOD HSA HAMMER TYPE 140 # AUTO

STRUCT. NO. EX SN 054-0002 PR SN 054-0516
Station 10281+81

BORING NO. 1B S. Pier
Station 10281+13
Offset 20.0ft LT
Ground Surface Elev. 589.6 ft

Surface Water Elev. 580.2 ft
Stream Bed Elev. 579.1 ft

Groundwater Elev.:
First Encounter 580.6 ft
Upon Completion Washed ft
After Plugged Hrs. ft

Description	DEPTH (ft)	BLOWS /6"	UCS Qu (tsf)	MOIST (%)
Seam from 40-42.5 Washed Gray Dry CLAY LOAM (Till) Drilled Hard at 32.5' Washed *(continued)*		5	B	
Washed		12		
	-45	28	9.2	9
		42	S-10	
Gray Dry CLAY LOAM (Till) Washed		10		
	-50	25	7.4	9
		27	S-10	
Gray Dry CLAY LOAM (Till) Washed		5		
	-55	13	6.4	12
		15	S-15	
532.10				
Gray Medium to Coarse SAND Washed		4		
	-60	3		

Description	DEPTH (ft)	BLOWS /6"	UCS Qu (tsf)	MOIST (%)
529.10		3		
Boring Completed				
	-65			
	-70			
	-75			
	-80			

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer, E-Estimated)
Abbreviations W.O.H - Sampler Advanced By Weight of Hammer, W.O.P - Advanced by Weight of Pipe, B.S. - Before Seating
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File Name S:\SOILS\GINT FILES\054 LOGAN\054-0516.GPJ Data Template D6TEMPLT.GDT Date Printed 8/27/14
Latitude No Data Longitude No Data Datum NAD83 Job Number D-96-008-09

Illinois Department of Transportation

Division of Highways
IDOT

SOIL BORING LOG

Date 5/28/14

ROUTE FR I-55 **DESCRIPTION** Over Kickapoo Creek **LOGGED BY** M. Tappan

SECTION 21 ACB **LOCATION** NE 1/4, **SEC.** 2, **TWP.** 20N, **RNG.** 2W, 3 **PM**

COUNTY Logan **DRILLING METHOD** HSA **HAMMER TYPE** 140 # AUTO

STRUCT. NO. EX SN 054-0002 / PR SN 054-0516
Station 10281+81

BORING NO. 2A N. Pier
Station 10282+33
Offset 21.0ft LT
Ground Surface Elev. 588.1 ft

Surface Water Elev. 580.2 ft
Stream Bed Elev. 579.1 ft

Groundwater Elev.:
▽ **First Encounter** 579.6 ft
▼ **Upon Completion** Washed ft
▼ **After** Plugged **Hrs.** ft

Description	DEPTH (ft)	BLOWS /6"	UCS Qu (tsf)	MOIST (%)
Black Moist SILTY CLAY		2		
		3	2.1	16
	-5	5	B	
582.10				
Brown Wet Medium SANDY GRAVEL		1		
		3		
		6		
▽				
Brown Wet Medium SANDY GRAVEL		2		
	-10	4		
		5		
		1		
Gray Wet Medium SANDY GRAVEL		2		
		2		
574.10		1		
Gray Moist CLAY LOAM (Till) Washed		2	1.6	13
	-15	3	B	
		2		
		3	2.6	12
		5	B	
		3		
		6	6.6	10
	-20	9	B	
Gray Moist CLAY LOAM (Till) Washed *(continued)*		4		
		11	4.9	10
		21	B	
		0		
Gray Wet CLAY LOAM (Till) Washed		1	.4	11
	-25	0	B	
		7		
Gray Moist CLAY LOAM (Till) Washed with 6" gray Medium SANDY GRAVEL From 28.5' to 29.5'		7	6.9	16
	-30	15	S-14	
		17		
Gray Dry CLAY LOAM (Till) Washed (Stopped Drilling Due To Rain)		56	12.2	8
	-35	44/4"	S-10	
		44		
Gray Dry CLAY LOAM (Till) Washed -- Poor Recovery		100/4"	10.0+	12
	-40		E	

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer, E-Estimated)
Abbreviations W.O.H - Sampler Advanced By Weight of Hammer, W.O.P - Advanced by Weight of Pipe, B.S. - Before Seating
The SPT (N value) is the sum of the last two blow values in each sampling zone (AASHTO T206) BBS, from 137 (Rev. 8-99)

Illinois Department of Transportation
Division of Highways
IDOT

SOIL BORING LOG

Date 5/28/14

ROUTE FR I-55 **DESCRIPTION** Over Kickapoo Creek **LOGGED BY** M. Tappan

SECTION 21 ACB **LOCATION** NE 1/4, **SEC.** 2, **TWP.** 20N, **RNG.** 2W, 3 **PM**

COUNTY Logan **DRILLING METHOD** HSA **HAMMER TYPE** 140 # AUTO

STRUCT. NO.	EX SN 054-0002 PR SN 054-0516
Station	10281+81
BORING NO.	2A N. Pier
Station	10282+33
Offset	21.0ft LT
Ground Surface Elev.	588.1 ft
Surface Water Elev.	580.2 ft
Stream Bed Elev.	579.1 ft
Groundwater Elev.:	
First Encounter	579.6 ft
Upon Completion	Washed ft
After Plugged **Hrs.**	ft

Description	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
05/30/14 Gray Moist CLAY LOAM (Till) Washed *(continued)*		29		
Gray Dry CLAY LOAM (Till) Washed -- Poor Recovery	-45	57 43//3"	13.4 S-12	21
		4		
Gray Dry CLAY LOAM (Till) Washed -- Poor Recovery	-50	5 10	3.4 B	10
536.10				
Gray Fine SAND Drilled Easy at 52'		9		
	-55	22 38		
		15		
Gray Medium SANDY GRAVEL Washed	-60	20 32		

Description	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Gray Fine SAND Drilled Easy at 52' *(continued)*		23		
523.10	-65	39 61/5"		
Boring Complete 522.10				
	-70			
	-75			
	-80			

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer, E-Estimated)
Abbreviations W.O.H - Sampler Advanced By Weight of Hammer, W.O.P - Advanced by Weight of Pipe, B.S. - Before Seating
The SPT (N value) is the sum of the last two blow values in each sampling zone (AASHTO T206) BBS, from 137 (Rev. 8-99)

Illinois Department of Transportation
Division of Highways
IDOT

SOIL BORING LOG

Date 6/27/14

ROUTE FR I-55 **DESCRIPTION** Over Kickapoo Creek **LOGGED BY** M. Tappan

SECTION 21 ACB **LOCATION** NE 1/4, **SEC.** 2, **TWP.** 20N, **RNG.** 2W, 3 **PM**

COUNTY Logan **DRILLING METHOD** HSA **HAMMER TYPE** 140 # AUTO

STRUCT. NO. EX SN 054-0002 / PR SN 054-0516
Station 10281+81

BORING NO. 3B N. Pier
Station 10282+48
Offset 22.0ft RT
Ground Surface Elev. 587.5 ft

Surface Water Elev. 580.2 ft
Stream Bed Elev. 579.1 ft

Groundwater Elev.:
First Encounter 579.0 ft
Upon Completion Washed ft
After Plugged **Hrs.** ft

Description	DEPTH (ft)	BLOWS /6"	UCS Qu (tsf)	MOIST (%)
Brown and Dark Gray Moist CLAY LOAM (Disturbed)		2		
		2	3.0	14
	-5	6	S-12	
579.50				
Gray Dirty Medium SANDY GRAVEL		2		
		6		
	-10	6		
		2		
		3		
	-15	3		
571.50				
Gray Moist CLAY LOAM (Till) Washed		2		
		3	1.8	13
		5	B	
Washed		2		
		7	6.6	10
	-20	11	B	

Description	DEPTH (ft)	BLOWS /6"	UCS Qu (tsf)	MOIST (%)
Gray Moist CLAY LOAM (Till) Washed *(continued)*				
Gray Moist CLAY LOAM (Till) Washed Sample Broken		5		
		19	6.5	8
		21	E	
564.50				
Gray Medium SAND with 1/4" - 1/2" Pea Gravel. Washed. Sand Blew in Augers 7'.		6		
		12		
	-25	15		
Gray Medium SAND with 1/4" Pea Gravel. Gray Dry CLAY LOAM (Till) at 29.5 Washed		10		
558.00		7	5.4	10
	-30	13	B	
Gray Dry CLAY LOAM (Till) Washed		11		
		75	9.0	9
Washed	-35	25	S-10	
		1"		
Washed		19		
		65	13.1	8
	-40	35	S-10	

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer, E-Estimated)
Abbreviations W.O.H - Sampler Advanced By Weight of Hammer, W.O.P - Advanced by Weight of Pipe, B.S. - Before Seating
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Latitude 40.13.329N Longitude 89.16.596W Datum NAD83 Job Number D-96-008-09

Illinois Department of Transportation
Division of Highways
IDOT

SOIL BORING LOG

Date 6/27/14

ROUTE FR I-55 **DESCRIPTION** Over Kickapoo Creek **LOGGED BY** M. Tappan

SECTION 21 ACB **LOCATION** NE 1/4, **SEC.** 2, **TWP.** 20N, **RNG.** 2W, 3 **PM**

COUNTY Logan **DRILLING METHOD** HSA **HAMMER TYPE** 140 # AUTO

STRUCT. NO. EX SN 054-0002 / PR SN 054-0516
Station 10281+81

BORING NO. 3B N. Pier
Station 10282+48
Offset 22.0ft RT
Ground Surface Elev. 587.5 ft

Surface Water Elev. 580.2 ft
Stream Bed Elev. 579.1 ft

Groundwater Elev.:
First Encounter 579.0 ft
Upon Completion Washed ft
After Plugged **Hrs.** ft

Description	Elev.	DEPTH (ft)	BLOWS (/6")	UCS Qu (tsf)	MOIST (%)
Gray Dry CLAY LOAM (Till) Washed *(continued)*			3"		
Washed			28		
			63	11.1	9
		-45	37	S-10	
			4"		
	540.50				
Gray Medium to Coarse SAND with 1/4" Pea Gravel Drilled Easy at 47' Washed			3		
			5		
	537.50	-50	12		
Boring Completed					
		-55			
		-60			

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer, E-Estimated)
Abbreviations W.O.H - Sampler Advanced By Weight of Hammer, W.O.P - Advanced by Weight of Pipe, B.S. - Before Seating
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Latitude 40.13.329N Longitude 89.16.596W Datum NAD83 Job Number D-96-008-09

Illinois Department of Transportation
Division of Highways
IDOT

SOIL BORING LOG

Date 2/9/12

ROUTE FR I-55 **DESCRIPTION** Over Kickapoo Creek **LOGGED BY** M. Tappan

SECTION 21 ACB **LOCATION** NE 1/4, **SEC.** 2, **TWP.** 20N, **RNG.** 2W, 3 **PM**

COUNTY Logan **DRILLING METHOD** HSA **HAMMER TYPE** 140 # AUTO

STRUCT. NO. EX SN 054-0002 / PR SN 054-0516
Station 10281+81

BORING NO. 2 NW Abut.
Station 10283+71
Offset 5.0ft LT
Ground Surface Elev. 602.3 ft

Surface Water Elev. 580.2 ft
Stream Bed Elev. 579.1 ft

Groundwater Elev.:
First Encounter 581.8 ft
Upon Completion Washed ft
After Plugged **Hrs.** ft

Description	Elev.	DEPTH (ft)	BLOWS /6"	UCS Qu (tsf)	MOIST (%)
Brown Dirty Moist CA-6 to Black Moist SILTY CLAY (Fill)			4		
			8	1.7	22
	600.80				
Very Dark Gray Moist SILTY CLAY (Fill)			4	B	
			2		
			3	1.9	25
			4	B	
	597.30	-5			
Gray Moist LOAM to CLAY LOAM (Fill)			2		
			3	1.3	16
			4	S-11	
	594.80				
Gray Moist LOAM (Fill)			1		
			2	1.2	16
			3	P	
		-10			
			1		
			2	.70	22
			2	B	
	589.80				
Black Moist SILTY CLAY LOAM (Fill)			2		
			3	1.2	23
			4	P	
		-15			
			1		
	586.30				
Light Brown and Gray Moist SILTY CLAY			2	1.4	23
			4	B	
	584.80				
Brown and Gray Moist LOAM with Gray Moist Medium to Coarse Sand at 19'			1		
			2	.90	19
	583.30				
Brown to Gray Dirty Medium SANDY GRAVEL			3	S-10	
		-20			

Description	Elev.	DEPTH (ft)	BLOWS /6"	UCS Qu (tsf)	MOIST (%)
Brown to Gray Dirty Medium SANDY GRAVEL *(continued)*			2		
			3		
			3		
			1		
Gray Dirty Fine to Medium SANDY GRAVEL			1		
			2		
		-25			
	574.30				
Gray Moist CLAY LOAM (Till)			2		
			4	2.5	13
Washed			6	B	
		-30			
			9		
Poor Recovery. Rock in Sampler.			17	5.0+	10
Washed			25	P	
		-35			
			9		
			16	4.5	12
Washed			17	B	
		-40			

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer, E-Estimated)
Abbreviations W.O.H - Sampler Advanced By Weight of Hammer, W.O.P - Advanced by Weight of Pipe, B.S. - Before Seating
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Illinois Department of Transportation

Division of Highways
IDOT

SOIL BORING LOG

Date 2/9/12

ROUTE FR I-55 **DESCRIPTION** Over Kickapoo Creek **LOGGED BY** M. Tappan

SECTION 21 ACB **LOCATION** NE 1/4, **SEC.** 2, **TWP.** 20N, **RNG.** 2W, 3 **PM**

COUNTY Logan **DRILLING METHOD** HSA **HAMMER TYPE** 140 # AUTO

STRUCT. NO. EX SN 054-0002 / PR SN 054-0516
Station 10281+81

BORING NO. 2 NW Abut.
Station 10283+71
Offset 5.0ft LT
Ground Surface Elev. 602.3 ft

Surface Water Elev. 580.2 ft
Stream Bed Elev. 579.1 ft

Groundwater Elev.:
First Encounter 581.8 ft
Upon Completion Washed ft
After Plugged **Hrs.** ft

Description	DEPTH (ft)	BLOWS /6"	UCS Qu (tsf)	MOIST (%)
Gray Moist CLAY LOAM (Till) *(continued)*				
		8		
Washed		9	4.4	10
6" Seam Gray Medium SANDY GRAVEL		19	S-12	
	-45			
		21		
Gray Dry CLAY LOAM (Till)		100	10.2	9
Washed		5"	S-8	
	-50			
		15		
		26	9.3	9
Washed		26	S-9	
	-55			
		18		
		46	10.3	9
Washed		54	S-10	
	-60	5"		

Description	DEPTH (ft)	BLOWS /6"	UCS Qu (tsf)	MOIST (%)
Gray Moist CLAY LOAM (Till) *(continued)*				
540.80				
Gray Fine SAND Drilled Easier at 61.5'				
		21		
		25		
Washed		32		
	-65			
		12		
Gray Dirty Medium Sand		31		
Washed		29		
	-70			
		14		
Gray Medium SANDY GRAVEL		15		
Washed		24		
	-75			
		20		
		19		
Washed		21		
	-80			

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer, E-Estimated)
Abbreviations W.O.H - Sampler Advanced By Weight of Hammer, W.O.P - Advanced by Weight of Pipe, B.S. - Before Seating
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Illinois Department of Transportation
Division of Highways
IDOT

SOIL BORING LOG

Date 2/9/12

ROUTE FR I-55 DESCRIPTION Over Kickapoo Creek LOGGED BY M. Tappan

SECTION 21 ACB LOCATION NE 1/4, **SEC.** 2, **TWP.** 20N, **RNG.** 2W, 3 **PM**

COUNTY Logan DRILLING METHOD HSA HAMMER TYPE 140 # AUTO

STRUCT. NO. EX SN 054-0002 PR SN 054-0516
Station 10281+81

BORING NO. 2 NW Abut.
Station 10283+71
Offset 5.0ft LT
Ground Surface Elev. 602.3 ft

Surface Water Elev. 580.2 ft
Stream Bed Elev. 579.1 ft

Groundwater Elev.:
First Encounter 581.8 ft
Upon Completion Washed ft
After Plugged **Hrs.** ft

Description	Elev.	DEPTH (ft)	BLOWS /6"	UCS Qu (tsf)	MOIST (%)
Gray Fine SAND Drilled Easier at 61.5' *(continued)*					
			25		
Washed			22		
	517.80		26		
Boring Completed		-85			
Ref. Sta. to Centerline of Ex. Structure=1028+81 Sta. Increase to North Ref. Elev. to BM 14=605.9					
		-90			
		-95			
		-100			

The Unconfined Compressive Strength (UCS) Failure Mode is indicated by (B-Bulge, S-Shear, P-Penetrometer, E-Estimated)
Abbreviations W.O.H - Sampler Advanced By Weight of Hammer, W.O.P - Advanced by Weight of Pipe, B.S. - Before Seating
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Latitude 40.13.349N Longitude 89.16.588W Datum NAD83 Job Number D-96-008-09

COFFERDAMS

Effective: October 15, 2011

Replace Article 502.06 with the following.

502.06 Cofferdams. A Cofferdam shall be defined as a temporary structure, consisting of engineered components, designed to isolate the work area from water to enable construction under dry conditions based on either the Estimated Water Surface Elevation (EWSE) or Cofferdam Design Water Elevation (CDWE) shown on the contract plans as specified below. When cofferdams are not specified in the contract documents and conditions are encountered where the excavation for the structure cannot be kept free of water for prosecuting the work by pumping and/or diverting water, the Contractor, with the written permission of the Engineer, will be permitted to construct a cofferdam.

The Contractor shall submit a cofferdam plan for each cofferdam to the Engineer for approval prior to the start of construction. Cofferdams shall not be installed or removed without the Engineer's approval. Work shall not be performed in flowing water except for the installation and removal of the cofferdam. The cofferdam plan shall address the following:

(a) Cofferdam (Type 1). The Contractor shall submit a cofferdam plan which addresses the proposed methods of construction and removal; the construction sequence including staging; dewatering methods; erosion and sediment control measures; disposal of excavated material; effluent water control measures; backfilling; and the best management practices to prevent reintroduction of excavated material into the aquatic environment. The design and method of construction shall provide, within the measurement limits specified in Article 502.12, necessary clearance for forms, inspection of exterior of the forms, pumping, and protection of fresh concrete from water. For Type 1 cofferdams, it is anticipated the design will be based on the EWSE shown on the contract plans. The Contractor shall assume all liability, financial or otherwise for a Type 1 cofferdam designed for an elevation lower than the EWSE.

(b) Cofferdam (Type 2). In addition to the requirements of Article 502.06(a), the Contractor’s submittal shall include detailed drawings and design calculations, prepared and sealed by an Illinois Licensed Structural Engineer. For Type 2 cofferdams it is anticipated the design will be based on the CDWE shown on the contract plans. The Contractor shall assume all liability, financial or otherwise for a Type 2 cofferdam designed for an elevation lower than the CDWE.

(c) Seal Coat. The seal coat concrete, when shown on the plans, is based on design assumptions in order to establish an estimated quantity. When seal coat is indeed utilized, it shall be considered an integral part of the overall cofferdam system and, therefore, its design shall be included in the overall cofferdam design submittal. If a seal coat was not specified but determined to be necessary, it shall be added to the contract by written permission of the Engineer. The seal coat concrete shall be constructed according to Article

503.14. After the excavation within the cofferdam has been completed and the piles have been driven (if applicable), and prior to placing the seal coat, the elevation of the bottom of the proposed seal coat shall be verified by soundings. The equipment and methods used to conduct the soundings shall meet the approval of the Engineer. Any material within the cofferdam above the approved bottom of the seal coat elevation shall be removed.

No component of the cofferdam shall extend into the substructure concrete or remain in place without written permission of the Engineer. Removal shall be according to the previously approved procedure. Unless otherwise approved in writing by the Engineer, all components of the cofferdam shall be removed.

Revise the first paragraph of 502.12(b) to read as follows.

(b) Measured Quantities. Structure excavation, when specified, will be measured for payment in its original position and the volume computed in cubic yards (cubic meters). Horizontal dimensions will not extend beyond vertical planes 2 ft (600 mm) outside of the edges of footings of bridges, walls, and corrugated steel plate arches. The vertical dimension for structure excavation will be the average depth from the surface of the material to be excavated to the bottom of the footing as shown on the plans or ordered in writing by the Engineer. The volume of any unstable and/or unsuitable material removed within the structure excavation will be measured for payment in cubic yards (cubic meters).

Revise the last paragraph of 502.12(b) to read as follows.

Cofferdam excavation will be measured for payment in cubic yards (cubic meters) in its original position within the cofferdam. Unless otherwise shown on the plans, the horizontal dimensions used in computing the volume will not extend beyond vertical planes 2 ft (600 mm) outside of the edges of the substructure footings or 4 ft (1.2 m) outside of the faces of the substructure stem wall, whichever is greater. The vertical dimensions will be the average depth from the surface of the material to be excavated to the elevation shown on the plans for bottom of the footing, stem wall, or seal coat, or as otherwise determined by the Engineer as the bottom of the excavation.

Revise the first sentence of the sixth paragraph of 502.13 to read as follows.

Cofferdams, when specified, will be paid for at the contract unit price per each for COFFERDAM (TYPE 1) or COFFERDAM (TYPE 2), at the locations specified.

GRANULAR BACKFILL FOR STRUCTURES
Effective: April 19, 2012
Revised: October 30, 2012

Revise Section 586 of the Standard Specifications to read:

SECTION 586. GRANULAR BACKFILL FOR STRUCTURES

586.01 Description. This work shall consist of furnishing, transporting and placing granular backfill for abutment structures.

586.02 Materials. Materials shall be according to the following.

Item	Article/Section
(a) Fine Aggregate	1003.04
(b) Coarse Aggregates	1004.05

CONSTRUCTION REQUIREMENTS

586.03 General. This work shall be done according to Article 502.10 except as modified below. The backfill volume shall be backfilled, with granular material as specified in Article 586.02, to the required elevation as shown in the contract plans. The backfill volume shall be placed in convenient lifts for the full width to be backfilled. Unless otherwise specified in the contract plans, mechanical compaction will not be required. A deposit of gravel or crushed stone placed behind drain holes shall not be required. All drains not covered by geocomposite wall drains or other devices to prevent loss of backfill material shall be covered by sufficient filter fabric material meeting the requirements of Section 1080 and Section 282 with either 6 or 8 oz/sq yd (200 or 270 g/sq m) material allowed, with free edges overlapping the drain hole by at least 12 in. (300 mm) in all directions.

The granular backfill shall be brought to the finished grade as shown in the contract plans. When concrete is to be cast on top of the granular backfill, the Contractor, subject to approval of the Engineer, may prepare the top surface of the fill to receive the concrete as he/she deems necessary for satisfactory placement at no additional cost to the Department.

586.04 Method of Measurement. This work will be measured for payment as follows.

(a) Contract Quantities. The requirements for the use of contract quantities shall conform to Article 202.07(a).

(b) Measured Quantities. This work will be measured for payment in place and the volume computed in cubic yards (cubic meters). The volume will be determined by the method of average end areas behind the abutment.

586.05 Basis of Payment. This work will be paid for at the contract unit price per cubic yard (cubic meter) for GRANULAR BACKFILL FOR STRUCTURES.

Integral Abutment Pile Selection

Integral abutment bridges eliminate the need for joints in bridge decks and thereby provide better protection for the superstructure from water and salt damage to the superstructure. Integral abutments are the preferred abutment type when appropriate and the Department continues to strive to increase the number of structures eligible for integral design.

The behavior and displacement capacity of integral abutment piles is not only a function of the soil-structure interaction that occurs with the soil embedded pile, but also the frame action that exists between the superstructure and abutment piles. The superstructure stiffness affects the rotational restraint, or fixity, of the pile head at the abutment and subsequently the moment developed in the pile as the superstructure expands and contracts and displaces the pile head laterally. In recent years, IDOT has implemented research resulting in expanded applicability of integral abutments with established prescriptive expansion length limits for the various available pile sizes. The prescriptive expansion length limits were derived from the displacement capacity of the piles for various anticipated soil conditions and superstructure stiffnesses anticipated to envelope most scenarios. This allows for a “no-analysis” policy intended to expedite integral abutment design by avoiding the need for designers to assess the capacity of piles for combined flexure and axial loads through frame analysis models that also include soil structure interaction.

The 2015 AASHTO LRFD interims introduced improvements increasing the structural capacity of concrete filled metal shell piles. These improvements have resulted in increased expansion length limits and applicability of metal shell piles for integral abutments. In addition, a superstructure stiffness correction factor has been introduced in an effort to better align pile behavior and superstructure stiffness and economize pile selections for superstructures that are smaller and more flexible. These improvements, including background information pertaining to IDOT’s integral abutment policy, are discussed in further detail in the following sections. Calculations for the correction factors presented herein have been programmed into an Excel spreadsheet titled “Integral Abutment Pile Selection” available on the IDOT website.

Placing piles directly beneath the superstructure beams or girders is considered the most efficient method of load transfer between the superstructure and abutment piles. As such, it is generally preferred that that the piles also be designed for axial load in a manner that results with an arrangement of one pile placed under each girder line. For integral abutments, it is permissible for the maximum pile spacing along the centerline of structure to exceed 8 ft.

IDOT has on-going research that includes instrumentation of in-service bridges. It is anticipated that the future research and knowledge of gained from the field instrumentation will result in further refinements in the future.

Design Thermal Movement

IDOT ABD Memo 15.7 revised the temperature range used to assess expansion joints to more accurately reflect the thermal ranges presented in AASHTO LRFD 3.12.2.

Substructure components are typically detailed and built in construction for bridge geometry corresponding to a base or “installation” temperature of 50 ^{o}F. Expansion joints benefit from having the ability to adjust the opening of the joints to accommodate the ambient temperature at the time of installation as described in Article 520.04 of the IDOT Standard Specifications for Road and Bridge Construction. Conversely, it can be difficult, if not impossible, to make adjustments in construction for expansion or contraction of longitudinal superstructure elements of beam/slab type bridges that may occur prior to, or as the subject components are installed. This occurs due to the temperature of the longitudinal superstructure elements simply being different than the 50 ^{o}F base temperature assumed for establishing the layout of the substructure units.

Structures in Illinois tend to be built in the warmer months and it is anticipated that the average temperature is approximately 70 ^{o}F when superstructures and integral abutments become “locked together”. Conversely, it is not unusual for portions of Illinois to experience short durations of sustained temperatures in the 0 to -5 ^{o}F range in the winter. As such and in lieu of the temperature range established by ABD Memo 15.7, the BBS has continued to use an 80 ^{o}F temperature range from “normal installation” for the study of integral abutment piling for contraction, as well as expansion, realizing it is likely conservative for the latter scenario. It is worth noting that letting dates for projects can be easily moved, making it difficult to predict during the design phase the time of year and anticipated ambient air temperature likely to exist when a structure becomes integral.

Pile Orientation and Capacity

The impact of various HP orientations was also previously assessed with the final chosen orientation being web perpendicular to the centerline of roadway (i.e., weak axis bending). A single orientation was chosen for the HP's, regardless of skew, as the dominant direction of displacement is generally parallel to the longitudinal axis of the structure. Secondly, consistent with the dominant direction of displacement, the dominant flexural demand is generally about the weak axis with the weak axis flexural capacity being relatively unaffected by the axial load on the pile (when considering that the axial load will be less than or equal to the maximum geotechnical axial capacity of the pile). Lastly, recognition was given to the assumptions employed by the Department in the design and analysis of integral abutment superstructures. Designers typically assume that the superstructure is simply supported at the abutment although a certain amount of frame action exists between the superstructure and abutment piles. The ability to assume a simply supported condition at the abutment greatly simplifies the superstructure design effort and is consistent with the assumption employed by the BBS in load rating the Department's bridge inventory using the AASHTO Bridge Rating software. As such, the weak axis of the piles was aligned with the primary bending axis of the superstructure in an effort to increase flexibility and simulate the assumed simply supported boundary condition as much as possible.

With a fixed connection between the superstructure and piles, movement of the superstructure is required to be accommodated through flexure and combined bending and axial loads on the piles. AASHTO (2010) 6.15.1 indicates that "piles shall be designed as structural members capable of safely supporting all imposed loads" while AASHTO (2010) 6.15.3.2 indicates that piles subjected to axial load and flexure shall be designed according to equations in AASHTO LRFD 6.9.2.2.

The equations in AASHTO LRFD 6.9.2.2 are intended to estimate member capacity for limit states governed by excessive bending within the member (i.e., away from "bracing" points) accompanied by sideways deflection and/or twisting (i.e., lateral-torsional buckling). The AASHTO code implies that the soil surrounding fully embedded piles is sufficient to prevent Euler buckling and there are numerous research papers suggesting that soil embedment is sufficient to also prevent lateral-torsional buckling. Given that the upper portion of IAB piles will generally be installed in competent cohesive embankment material having a minimum Q_u of 1.0 tsf, this limit state is considered negligible for integral abutment piles.

A second limit state discussed in the “Guide to Stability Design Criteria for Metal Structures” by Theodore Galambos is the in-plane or local cross-sectional strength of the member. This limit state is considered to be more applicable for integral abutment piles given that the maximum bending moment in the pile typically occurs right at the abutment cap. Galambos provides the following equations for checking the ultimate cross sectional moment capacity of I-shaped members modified for the effect of axial compression:

$$\frac{P}{P_y} + 0.85\left(\frac{M_o}{M_p}\right) \le 1.0 \text{ (strong-axis bending)}$$

$$\left(\frac{P}{P_y}\right)^2 + 0.84\left(\frac{M_o}{M_p}\right) \le 1.0 \text{ (weak-axis bending)}$$

Where:

P = applied axial load

P_y = axial load at full yield

M_o = applied moment

M_p = plastic bending moment

$M_o \le M_p$

The above local cross-sectional strength equations assume that slenderness and local buckling of the flanges is not a concern. A factored version of these equations exists in Appendix H of the 3rd Edition of the AISC code as shown below:

$$\left(\frac{M_{ux}}{\varphi_b M'_{px}}\right)^{\zeta} + \left(\frac{M_{uy}}{\varphi_b M'_{py}}\right)^{\zeta} \le 1.0 \qquad \varphi_b=0.9 \quad \text{(AISC Eqn. A-H3-1)}$$

$$\zeta = 1.6 - \frac{\frac{P_u}{P_y}}{2\left[\ln\left(\frac{P_u}{P_y}\right)\right]} \quad \text{(AISC Eqn. A-H3-3)}$$

$$M'_{px} = 1.2 \text{ x } M_{px}\left[1-\left(\frac{P_u}{P_y}\right)\right] \leq M_{px} \quad \text{(AISC Eqn. A-H3-5)}$$

$$M'_{py} = 1.2 \text{ x } M_{py}\left[1-\left(\frac{P_u}{P_y}\right)^2\right] \leq M_{py} \quad \text{(AISC Eqn. A-H3-6)}$$

The above equations are noted in AISC as being a considerable liberalization over those contained within the specification and mirrored in the AASHTO code. Acknowledging the statistical and probability basis of LRFD design, it is noted that there are different load and resistance factors between the AISC and AASHTO codes for similar loads and strength checks. One difference between the two codes is that the resistance factor for flexural resistance is 0.9 in AISC and 1.0 in AASHTO. Similarly, the resistance factor for axial compression is 0.85 in AISC and 0.7 for the axial resistance of HP's in the AASHTO code.

AASHTO LRFD 10.7.1.5 indicates that long-term durability of the pile (corrosion and deterioration) shall be taken into consideration and is discussed in further detail in AASHTO LRFD 10.7.5. It's been long suspected that gaps exist beneath the abutments due to normal consolidation and long term settlement of the embankments allow air and water to come in contact with the piles. With the elimination of the concrete encasement, IDOT desired to maintain some corrosion protection of the piles or allowance for long term section loss due to corrosion. To address potential corrosion, it was decided to use a hybridized version of the AASHTO and AISC codes in assessing pile capacity by using resistance factors of 0.9 for flexure (AISC) and 0.7 for compression (AASHTO) to account for long term section loss. These resistance factors are also intended to account for additional eccentric loads that may be induced into the piles as a result of the structure being exposed to a larger temperature range due to the temperature at the time of construction, potential presence of long term shrinkage, driving tolerances for the piles, etc. Per Article 512.12 of the IDOT Standard Specifications for Road and Bridge Construction, piles are permitted to be driven out of plan position by as much as 6 inches. It is anticipated that the above resistance factors are likely conservative but were chosen for current use in lieu of performing statistical calibration and until future research is completed.

In the past, the flexural capacity of concrete-filled metal shell piles was computed using the ACI 318-05 code as it was much more liberal than the design provisions in the AASHTO code. The 2015 Interim Revisions to the AASHTO code introduced significant revisions for calculating the combined compression and flexural capacity of concrete-filled steel tubes considering composite action resulting in improved capacities. Combined flexural and axial capacity of metal shell piles for IAB's is now assessed using the interim revisions coupled with the use of an increased reinforcement cage (see metal shell piling base sheet) inside the metal shell pile and increase yield strength for the metal shell material (50 ksi). Since the reinforcement cage is explicitly relied upon for assessment of the structural capacity of the metal shell pile, a reduction in metal shell thickness of 0.06 in. is taken into account for potential corrosion as suggested by AASHTO LRFD 5.13.4.5.2.

Base Permissible Expansion Length

To assess displacement capacity and force demands on the abutment piles, 3-dimensional finite element analysis models were assembled with the following parameters:

- 63 in. plate girder with 1/2 in. webs, 1 in. x 14 in. flanges, and $\approx$ 136.75 ft spans
- 6 girders spaced at 6 ft centers
- 36 ft wide, 8 in. thick concrete deck
- 3 ft thick pile cap and concrete diaphragm
- 3'-6" tall pile cap beneath the bottom of the superstructure beam
- Plates were used to model the deck, pile cap, concrete diaphragm, and wingwalls.
- Beam elements were used to model the superstructure girders and piles. Inelastic beam elements were used for the pile segments just below the abutment cap. Rigid links were provided between the superstructure girders and deck to capture composite action.
- $\alpha_{Steel} = 6.5e^{-6}$ / ^{o}F, $\alpha_{Concrete} = 5.5e^{-6}$ / ^{o}F, $\Delta_{Temperature}$ = +/- 80 ^{o}F
- 1 ft thick "dog-ear" style wingwalls. The lengths were sized assuming soil is allowed to wrap around to the front side with a maximum length of 10 ft.
- Roller supports at the piers.

- Abutment piles were placed beneath each girder and were modeled to extend 2 ft into the pile cap.
- Steel superstructures were modeled for 0, 15, 30, and 45 degree skews.
- P-y soil springs were modeled along the length of the pile assuming soil with a Q_u of 1.5 tsf.
- P-y soil springs for the abutment backfill were modeled along the back of abutment assuming an internal friction angle of 35 degrees and placed at an angle of 15 degrees from the axis perpendicular to the abutments for skews exceeding 15 degrees to account for wall friction.

Figure 1 shows the results of the analysis models and permissible effective expansion lengths that correspond with the previously discussed methods for computing the combined axial load and bending pile capacities. As the intent of the analysis models is to assess superstructure stiffness effects on the various piles, the pile capacities are assessed and permissible effective expansion lengths are computed assuming each pile is loaded to its maximum factored geotechnical axial capacity and not necessarily the vertical reactions that correspond with the superstructure parameters.

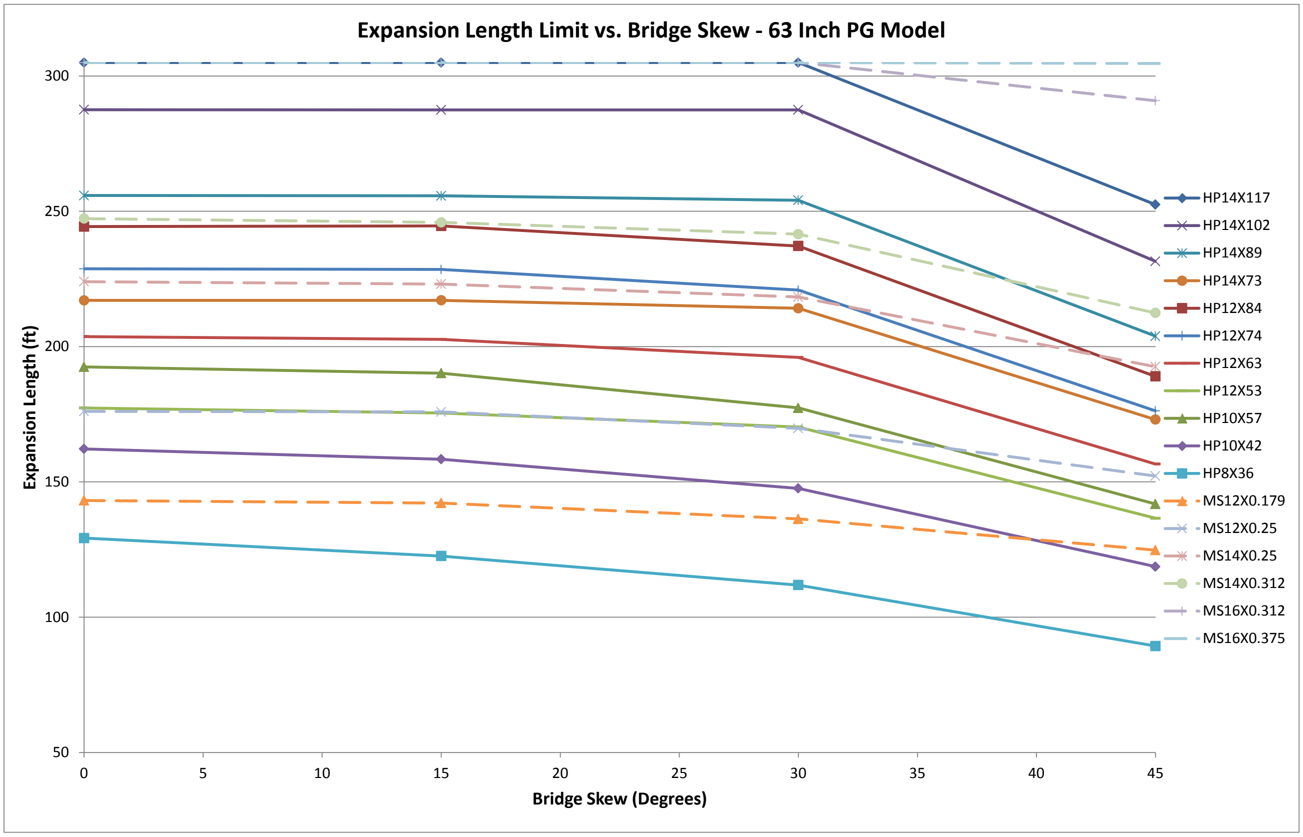


Figure 1. 63-Inch Plate Girder Results

For the piles in Figure 1 whose expansion lengths truncate at 305 ft, this is not necessarily an indication of the expansion length that corresponds to the pile capacity but rather the limits of the analysis chosen by the BBS considering limitations of the strip seal expansion joint at the ends of the bridge approach slabs.

Superstructure Stiffness Expansion Length Correction Factor

Superstructure stiffness is viewed as one of the largest factors that affect permissible expansion lengths for a given pile. The superstructure properties used to generate the results in Figure 1 were chosen from an example structure anticipated to result in a superstructure stiffness and permissible expansion lengths that are likely conservative for the majority of “garden variety” structures. To investigate the effects of varying superstructure stiffnesses, a limited number of piles have been analyzed using the same finite element model previously described for the 63-inch plate girder with the following alternate superstructure modifications:

- W30x124 Beam, 68.4 ft Spans
- 72" PPC Bulb-T, 110 ft Spans
- 36" PPC I-Beam, 59.5 ft Spans

Analysis results from the alternate superstructure properties have been analyzed against those in Figure 1 that were generated using the base superstructure properties corresponding to the 63-inch plate girder. Following is a procedure for adjusting the permissible expansion lengths in Figure 1 for alternate superstructure properties in an effort to better economize and align pile options for the superstructure stiffness of any bridge under consideration.

Figure 2 provides a qualitative depiction of the movement that occurs at an integral abutment due to thermal contraction. This movement can be summarized with the following equation:

$$\alpha_{eff}\ L_{exp}\ \Delta_T - \frac{V_p\ L_{exp}}{AE_{eff}} = \Delta_p + \Delta_\theta$$

Equivalently, thermal contraction of the superstructure minus elastic lengthening of the superstructure due to the abutment resistance equals the lateral pile displacement (Δ_p) plus the lateral movement that occurs due to rotation of the pile and superstructure (Δ_θ). The above equation can be rearranged as follows to solve for the expansion length.

$$L_{exp} = \frac{\Delta_p + \Delta_\theta}{\alpha_{eff}\ \Delta_T - \dfrac{V_p}{AE_{eff}}}$$

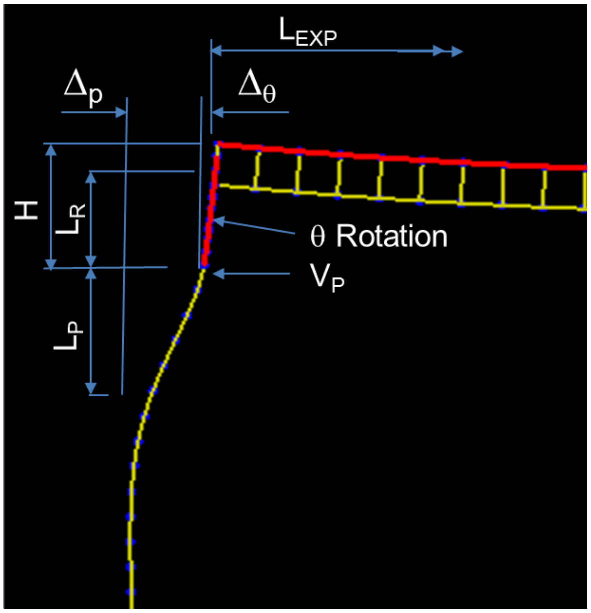


Figure 2. Illustration of Integral Abutment Movement

Where:

L_{exp} =	permissible expansion length for a given pile
Δ_p =	lateral displacement of the pile that corresponds with the maximum moment capacity of the pile
Δ_θ =	lateral displacement over the height from the bottom of the abutment cap to the mid-thickness of the deck that occurs to rotation of the superstructure and pile
=	$H^*\theta$
H =	height from the bottom of the abutment cap to the mid-thickness of the deck (in.)
θ =	rotation of the superstructure and pile
α_{eff} =	effective thermal coefficient for the superstructure
Δ_T =	temperature range over which thermal contraction is presumed to occur (taken as 80°F for the current study)
V_p =	shear force at the top of the pile that corresponds with the lateral stiffness and maximum moment capacity of the pile
AE_{eff} =	effective cross-sectional stiffness of the superstructure

The rotational stiffness of the superstructure at the abutment and the pile have a significant effect on Δ_p and Δ_θ. This rotational stiffness may be estimated using the following relationships:

k_θ = total rotational stiffness at the abutment (k*ft/rad.)

= $k_{\theta\text{-}p} + k_{\theta\text{-}s}$

$k_{\theta\text{-}p}$ = rotational stiffness of the pile (k*ft/rad.)

$$= \frac{EI_p}{144\ L_p}$$

EI_p = flexural stiffness of the pile (k*in.^{2}) (Note that for HP sections, the weak axis moment of inertia shall be used.)

L_p = approximate fixity depth of the pile for soil with Q_u equal to 1.5 tsf (ft)

= $2.2\ \mathrm{Ln}\,(EI_p) - 24$

$k_{\theta\text{-}s}$ = rotational stiffness of the superstructure (k*ft/rad.)

$$= \frac{2\ E_n\ I_{ne}\ s_p}{L_e\ s_s} \text{ (for simple spans)}$$

$$= \frac{E_n\left(\frac{3\ I_{ne}}{L_e} + \frac{2\ I_{na}}{L_a}\right) s_p}{72\left(2 + \frac{I_{na}\ L_e}{I_{ne}\ L_a}\right) s_s} \text{ (for continuous spans)}$$

s_s = superstructure beam spacing perpendicular to centerline of structure (ft)

s_p = pile spacing perpendicular to centerline of structure (ft)

I_{ne}, I_{na} = short term composite moment of inertia for the end span (I_{ne}) and adjacent interior span (I_{na}) superstructure beam using the width of the deck tributary to the beam (in.^{4})

L_e, L_a = length of the end span (L_e) and adjacent interior span (L_a) (ft) (Note: L_a shall be set to a small value, such as 0.01 ft, for 2-span structures)

E_n = modulus of elasticity used to calculate I_{ne} and I_{na} (ksi)

The above formula for calculating the rotational stiffness of the pile models the pile as a cantilever with a concentrated moment applied to the free end. The general form of this equation (EI/L) can be found in most structural analysis text books. The expression for the fixity depth of the pile, L_p, acknowledges that that this hypothetical parameter varies according to pile stiffness and was derived from the results of analysis models for the 63-in. plate girder according to the depth at which there is an inflection point in the bending moment for the pile. The fixity depths are anticipated to nominally fluctuate for a given pile as superstructure stiffness changes. However, the proposed estimated depths are considered suitable for the purposes of scaling the effects of superstructure stiffness on permissible expansion lengths.

The formulas for calculating the rotational stiffness of the superstructure assume a simply supported structure with a concentrated moment applied at the abutment and adjusts the stiffness for the ratio of the pile to superstructure beam spacing. The equation provided for the simple span condition can also be found in most structural analysis text books. The equation for the continuous span condition was derived using the "slope deflection" method of analysis for a simply supported continuous beam with the end of the adjacent span restrained for flexure but free to deflect vertically as shown in Figure 3.

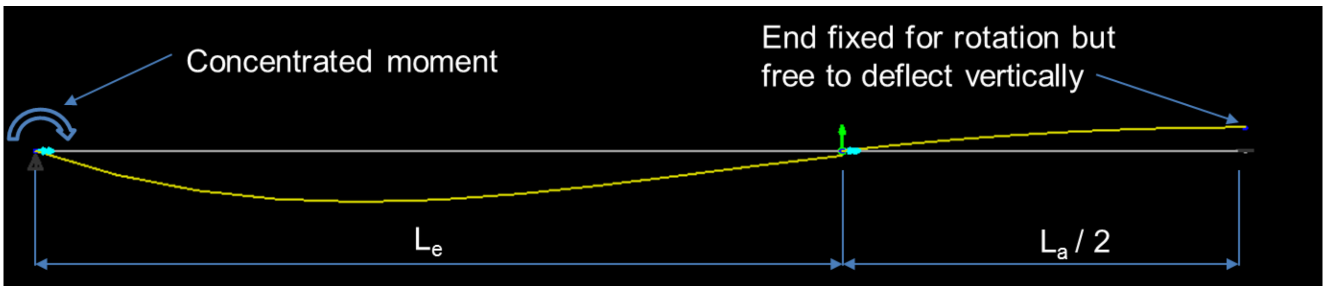


Figure 3. Continuous span model used for estimating superstructure rotational stiffness

Span configurations will affect the location of the inflection point in the adjacent span (i.e., it will not always occur at $L_a/2$ as shown in Figure 3). Estimated stiffnesses using the derived formula have been checked against values obtained from software analysis for several varying span configurations. The estimated rotational stiffness are generally within a small percentage of the values obtained from software analysis indicating that the estimated values are reasonably accurate.

In addition for steel structures, beam sections often change within the negative moment region. To investigate this impact, additional analysis was conducted with the moment of inertia of the negative moment region sections modeled as either 0.5 or 2.0 times the value in the positive moment region. The estimated rotational stiffness values were calculated using the stiffness in the positive moment regions. The comparative analysis indicated that when the moment of inertia of the negative moment region is less than the positive moment region, there is improved agreement between the values obtained from software analysis and the estimated values. This is considered favorable as the short term composite moment of inertia in the positive moment region is generally anticipated to be greater than the negative moment of inertia considering either the noncomposite steel section or the cracked composite moment of inertia. Therefore, it is deemed acceptable to use section properties in the positive moment regions of the end and adjacent spans for estimating the superstructure rotational stiffness.

The effective cross-sectional stiffness of the superstructure (AE_{eff}) is used to account for elastic lengthening of the structure and may be calculated as follows:

AE_{eff} = $AE_{eff\text{-}s}$ adjusted for the ratio of the pile to superstructure beam spacing

$$= \frac{AE_{eff\text{-}s}\, s_p}{s_s}$$

$AE_{eff\text{-}s}$ = AE of the composite superstructure beam adjacent to the abutment (single and 2-span continuous structures) (k*in.^{2})

$$= \frac{(L_e + \lambda\, L_a)\, AE_e\, AE_a}{AE_a L_e + AE_e\, \lambda\, L_a}$$ (continuous structures with more than 2 spans) (k*in.^{2})

AE_e = AE of the composite superstructure beam in the end span (k*in.^{2})

AE_a = AE of the composite superstructure beam in the adjacent interior span (k*in.^{2})

λ = span length factor

= 0.5 for 3-span structures

= 1.0 for structures with more than 3 spans

The above equation for continuous structures was derived using the axial load deformation relationship for members with variable cross-sectional areas that can be readily found in most mechanics of materials textbooks. Similar to the discussion for the rotational stiffness of the superstructure, it is acknowledged that steel structures often utilize larger sections in the negative moment region. The potential impact of larger negative moment sections was assessed with a series of previously designed structures having larger beam sections in the negative moment regions. There was generally less than a 5% difference in $AE_{eff\text{-}s}$ values between assuming the positive moment region properties over the entire span length and including the properties of the larger beams in the negative moment region. This small difference is due to the inverse relationship involved in calculating axial stiffness of members with variable cross sections and connected in series (i.e., end to end). This difference is deemed negligible considering potential effects of deck cracking, lateral stiffness of intermediate piers, resistance of expansion bearings, etc. As such, and similar to the rotational stiffness of the superstructure, it is recommended that $AE_{eff\text{-}s}$ only be calculated using the superstructure properties in the positive moment regions.

Effective coefficient of thermal expansion (α_{eff}) is an intermediate coefficient of dissimilar materials working together (i.e., steel and concrete) and is calculated according to the cross-sectional stiffness of the individual elements. α_{eff} may be calculated as indicated below:

$$\alpha_{eff} = \frac{\alpha_{Concrete}\, AE_{Concrete} + \alpha_{Steel}\, AE_{Steel}}{AE_{eff\text{-}s}}$$

$\alpha_{Concrete}$ = coefficient of thermal expansion for concrete ($5.5e^{-6}$ / °F)

α_{Steel} = coefficient of thermal expansion for steel ($6.5e^{-6}$ / °F)

$AE_{Concrete}$ = AE of the concrete slab tributary to the superstructure beam (k*in.^{2})

AE_{Steel} = $AE_{eff\text{-}s} - AE_{Concrete}$ (k*in.^{2}) (accounts for variable steel cross sections that may exist in the end and adjacent spans for continuous structures)

The lateral pile displacement, Δ_p, is difficult to predict with simple equations due to the non-linear resistance of the soil as well as the effects of the superstructure. However, the following formula has been derived in an effort to qualitatively predict the effects that the superstructure has on the relationship between pile moment and lateral pile displacement. The following formula models the pile as a “fixed-fixed” member of length L_P with a reduction in flexure at the top of the pile that is a function of the total rotational stiffness at the abutment and assumes that member “L_R” shown in Figure 2 is a rigid link.

$$\Delta_p = \frac{M_p\left(1 + \frac{4\, EI_p}{144\, k_\theta\, L_P}\right)}{k_p\left(\frac{L_P}{2} - \frac{4L_R\, EI_p}{144\, k_\theta\, L_P}\right)}$$

M_p = moment capacity of pile (k*ft)

k_p = lateral stiffness of the pile for a “fixed-fixed” condition (k/in.)

$$= \frac{12\,EI_p}{(12\,L_P)^3}$$

L_R = vertical distance from the bottom of the abutment cap to the centroid of the composite superstructure at the abutment (ft)

In addition, the estimated lateral movement that occurs due to rotation of the pile and superstructure, Δ_θ, can be further refined as a function of the pile moment and total rotational stiffness at the abutment.

$$\Delta_\theta = \frac{M_p\,H}{k_\theta}$$

Using the assorted variables described herein, regression analysis was performed in Excel to develop the following relationship to adjust permissible expansion lengths for a given pile for various superstructure properties:

ELCF = expansion length correction factor

$$= 0.9077 \times 0.9967^{R_{ps}} \times 4.345^{R_p} \times 0.9874^{R_\theta} \times 0.2674^{R_a} \times 0.9752^{R_{ea}}$$

R_{ps} = pile stiffness factor

$$= \frac{EI_p}{1168700}$$

R_p = Δ_p ratio

$$= \frac{\left[\dfrac{\left(1 + \dfrac{4\,EI_p}{144\,k_\theta\,L_P}\right)}{\left(\dfrac{L_P}{2} - \dfrac{4L_R\,EI_p}{144\,k_\theta\,L_P}\right)}\right]_{alt}}{\left[\dfrac{\left(1 + \dfrac{4\,EI_p}{144\,k_\theta\,L_P}\right)}{\left(\dfrac{L_P}{2} - \dfrac{4L_R\,EI_p}{144\,k_\theta\,L_P}\right)}\right]_{base}}$$

R_θ = Δ_θ ratio

$$= \frac{\left[\dfrac{H}{k_\theta}\right]_{alt}}{\left[\dfrac{H}{k_\theta}\right]_{base}}$$

$R_a = \alpha_{eff}$ ratio

$= \dfrac{[\alpha_{eff}]_{alt}}{[\alpha_{eff}]_{base}}$

$R_{ea} = AE_{eff}$ ratio

$= \dfrac{[AE_{eff}]_{alt}}{[AE_{eff}]_{base}}$

base = properties related to the 63-in. plate girder model

alt = properties related to an alternate superstructure configuration

For the R_p and R_θ ratios, M_p and k_p are considered constant for a given pile and cancel out of the equations for Δ_p and Δ_θ.

The width of some analysis models were also increased to investigate potential effects of varying bridge widths. The impact to the biaxial bending demands on the piles was generally small and deemed not significant enough to develop additional policy at this time considering all other variables involved.

Soil Modification Factors

Abutments are often constructed on top of manmade embankments which are typically required by IDOT policy to consist of compacted material having a minimum Q_u of 1.0 tsf. As such, the BBS chose to assume for the aforementioned analysis models that the upper portion of the piles subjected to significant bending and lateral displacement would be installed in material having a Q_u of 1.5 tsf. Assuming a Q_u of 1.5 tsf was anticipated to envelope a significant amount of soil properties typically encountered within the embankment at a nominal depth below the pile cap and should generate results that are conservative for weaker soils. Through time it has become apparent that a modest number of structures exist in which the soil strengths at shallow depths are comprised of soils having a Q_u greater than 1.5 tsf and/or contain granular soil layers. Rather than simply discount these structures from being eligible for integral abutments, additional correction factors have been developed.

“Pushover” analysis models have been used to assess the impact of soils strengths other than 1.5 tsf, and up to a maximum of 3.0 tsf, on various pile sizes. Increased soil strength results in increased pile stiffness and a decrease in lateral displacement of the pile corresponding to the pile flexural capacity, "M_p". Analysis suggests that there is approximately a 15% decrease in the displacement capacity of the piles for each 0.5 tsf increase in Q_u. As such, the permissible expansion lengths shown in Figure 1 can be reduced by the following modification factor to adjust for the effect of soils with a Q_u greater than 1.5 tsf:

$$M_{pile} = 1.45 – 0.3 \times Q_u$$

Analysis indicates that the above equation produces conservative results for soils with a Q_u less than 1.5 tsf.

The above equation unfortunately only addresses the effect of the stiffer soil on the pile itself. As soil stiffness increases, a larger lateral force is required to achieve a pile displacement that corresponds to the pile's moment capacity. IDOT's standard integral abutment reinforcement is based on a design moment at the base of the superstructure that is a function of the pile moment plus flexure caused by the lateral pile force acting over the height of the cap for displacement demands corresponding to soil with a Q_u of 1.5 tsf. As such, the following expression and reduction factor was developed for the permissible expansion lengths shown in Figure 1 to ensure that the pile demands from the stiffer soil conditions do not exceed the assumptions used in standardizing the abutment reinforcement. The following equation is more restrictive than the equation shown above for the piles. This equation does not apply for soils with a Q_u less than 1.5 tsf.

$$M_{abut} = 1.5 / Q_u$$

For soils with a Q_u other than 1.5 tsf, the formula shown for M_{pile} can also be used to provide a reasonable estimate of the lateral stiffness of a given pile relative to its lateral stiffness for soils with a Q_u equal to 1.5 tsf. To obtain the relative lateral stiffness, the reciprocal of the equation shown for M_{pile} should be used.

It is recommended that a weighted average of the soil strengths within a depth of 10 ft (considered the "critical pile depth") below the abutment cap be used when assessing the previously mentioned modification factors. Below a depth of 10 ft, pushover analysis models suggest increased soil stiffness has minimal effect on the force demands on the pile for the magnitude of displacements considered when the average Q_u within the critical pile depth is greater than or equal to 1.5 tsf. Conversely, when the average Q_u within the critical pile depth is less than 1.5 tsf, pushover analysis models suggest increased soil stiffness below 10 ft may be influential on the pile response. However, the generally conservative results for the above "M_{pile}" equation for soils less with a Q_u less than 1.5 tsf should envelope these effects in such scenarios.

While it is anticipated that the upper portion of integral abutment piles will generally be installed in embankment material consisting of cohesive soils, designers may occasionally encounter soil profiles with a combination of cohesive and granular soils within the critical pile depth. The following expression should be used for converting granular soil layers to equivalent cohesive soils for the purpose of evaluating soils within the critical pile depth.

$Q_u = 0.75*\ln(N) + 0.7$

N is the SPT blow count recorded in the soil boring logs. This expression was derived by conducting a series of lateral load pile analysis for combinations of granular and cohesive soils. The above equation is intended only for the purpose of trying to equate the lateral stiffness of shallow granular soil layers and is not intended to be used for assessing the strength of granular soil layers.

Average soil strengths within the critical pile depth of 3.0 tsf have generally been considered by the BBS as an upper limit for using integral abutments. Beyond 3.0 tsf, piles are anticipated to encounter significant resistance to lateral deflection from thermal superstructure movement that has not been investigated to date by the BBS. There are however some instances in which it may be acceptable to use integral abutment with soils having a Q_u exceeding 3.0 tsf. Such scenarios will generally include significantly different soil strengths at each abutment. As an example, if the average soil strengths at the abutments were 0.8 and 4.0 tsf, the abutment with 4.0 tsf soil is anticipated to be fairly rigid and exhibit little lateral movement with most of the thermal superstructure movement occurring at the abutment with the weaker soil. When the average soil strengths at an abutment exceed 3.0 tsf and the thermal length of structure tributary to the subject abutment are less than 20% of the overall structure length, integral abutments may be used. The 20% is based upon engineering judgement acknowledging the variability that may exist when calculating the thermal length of structure tributary to an abutment using relative stiffness and the above "M_{pile}" equation.

When average soil strengths at an abutment exceed 3.0 tsf and do not satisfy the above 20% criteria, semi-integral abutments are the next recommended option to achieving a jointless structure. Precoring holes in such situations for the 10 ft critical pile depth to increase pile flexibility and backfilling with loose sand is not recommended at this time due to potential concerns with progressive consolidation and stiffness of the sand that may occur due to cyclical pile movement. For similar reasons, integral abutments are typically not used within the select fill area of MSE retaining walls. Backfilling the precored holes with bentonite may be considered. However, bentonite is considered to be a low strength material having properties similar to cohesive soil with a Q_u of approximately 0.1 tsf and is not considered adequate to offer continuous bracing against pile buckling. Designers choosing to use bentonite should check the capacity of the pile for combined bending and axial loads according to AASHTO LRFD 6.9.2.2 (HP's) and 6.9.6.3 (metal shell piles) considering the pile to be unbraced.

<u>End Span Length Restrictions</u>

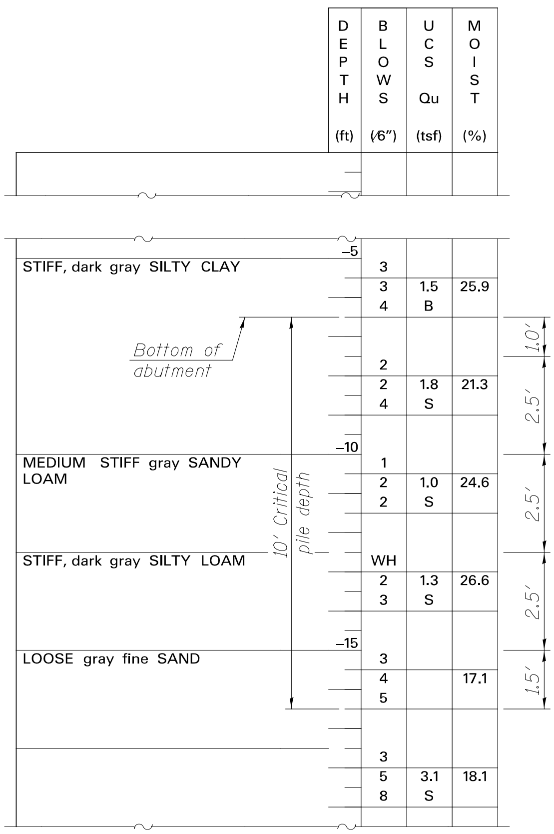
Live load that causes downward deflection in the end span typically increases flexural demand on the abutment piles for the thermal expansion scenario while decreasing the flexural demand for the contraction scenario. For the thermal loading condition, superstructure contraction generally controls the flexural demand on the piles. As such, analysis used to generate the results in Figure 1 assumed contraction controlled with live load placement to create the maximum vertical live load reaction at the abutment.

Select structures with longer end spans have been analyzed and scenarios identified where live load rotations in the end span suggest larger piles should be used at the abutments than would be otherwise specified for typical structures. As such, use of the pile selection procedure detailed herein is limited to simple span structures having a maximum length of 170 ft and continuous span structures with a maximum end span length of 200 ft. In addition, abutments adjacent to spans of 150 ft or greater shall use 14 or 16-inch metal shell piles or HP 12 x74 piles and larger.

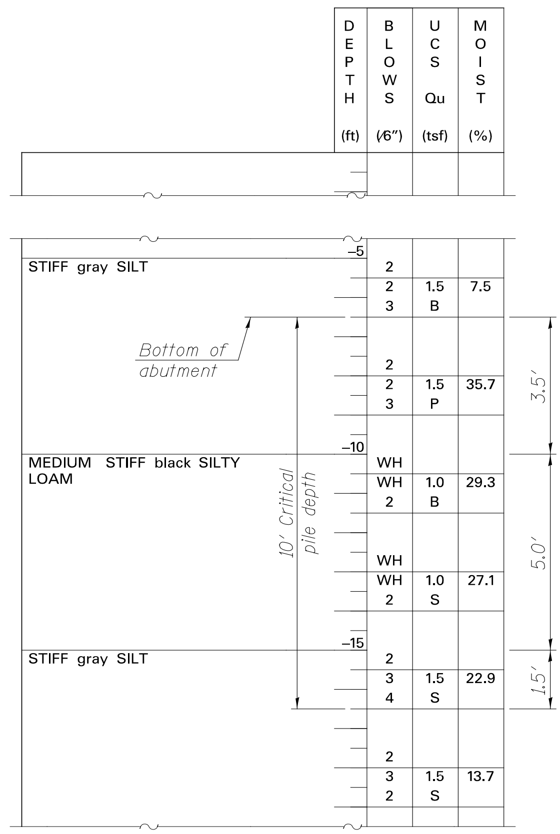
This is an item that continues to be researched, along with the effects of integral abutments on superstructures, and future refinements are expected as field instrumentation data is collected, analyzed, and analysis models are calibrated and refined.

Pile Selection Example 1

The structure is a continuous 450 ft. long structure consisting of 6 – 75 ft. spans with a zero degree skew. The superstructure consists of 5 - W36x150 beams at a 7 ft spacing with an 8 inch thick deck. The structure is the same width throughout and thus expected to have the same number of piles at each abutment. The following example determines the effective expansion length for the structure and indicates acceptable piles.



West Abutment Boring B-1



East Abutment Boring B-2

Determine the average Q_u for the critical pile depth at each abutment.

$$Q_{u\text{-}west} = \frac{(1.0)(1.5)+(2.5)(1.8)+(2.5)(1.0)+(2.5)(1.3)+(1.5)[0.75\ln(9)+0.7]}{10}$$

$$= 1.53 \text{ (say 1.5 tsf)}$$

$$Q_{u\text{-}east} = \frac{(3.5)(1.5)+(5.0)(1.0)+(1.5)(1.5)}{10} = 1.25 \text{ tsf}$$

Determine the pile stiffness modifier for the east abutment since it has an average Q_u that is not equal to 1.5 tsf.

$$M_{east} = \frac{1}{1.45-0.3(1.25)} = 0.93$$

Assume 6 beam lines in the structure with a pile placed beneath each beam and calculate the centroid of stiffness from the west abutment.

$$\Sigma_{\text{Stiff. W. Abut.}} = \frac{(6 \text{ piles})(0 \text{ ft})+(6 \text{ piles})(0.93)(450 \text{ ft})}{(6 \text{ piles})+(6 \text{ piles})(0.93)} \approx 217 \text{ ft}$$

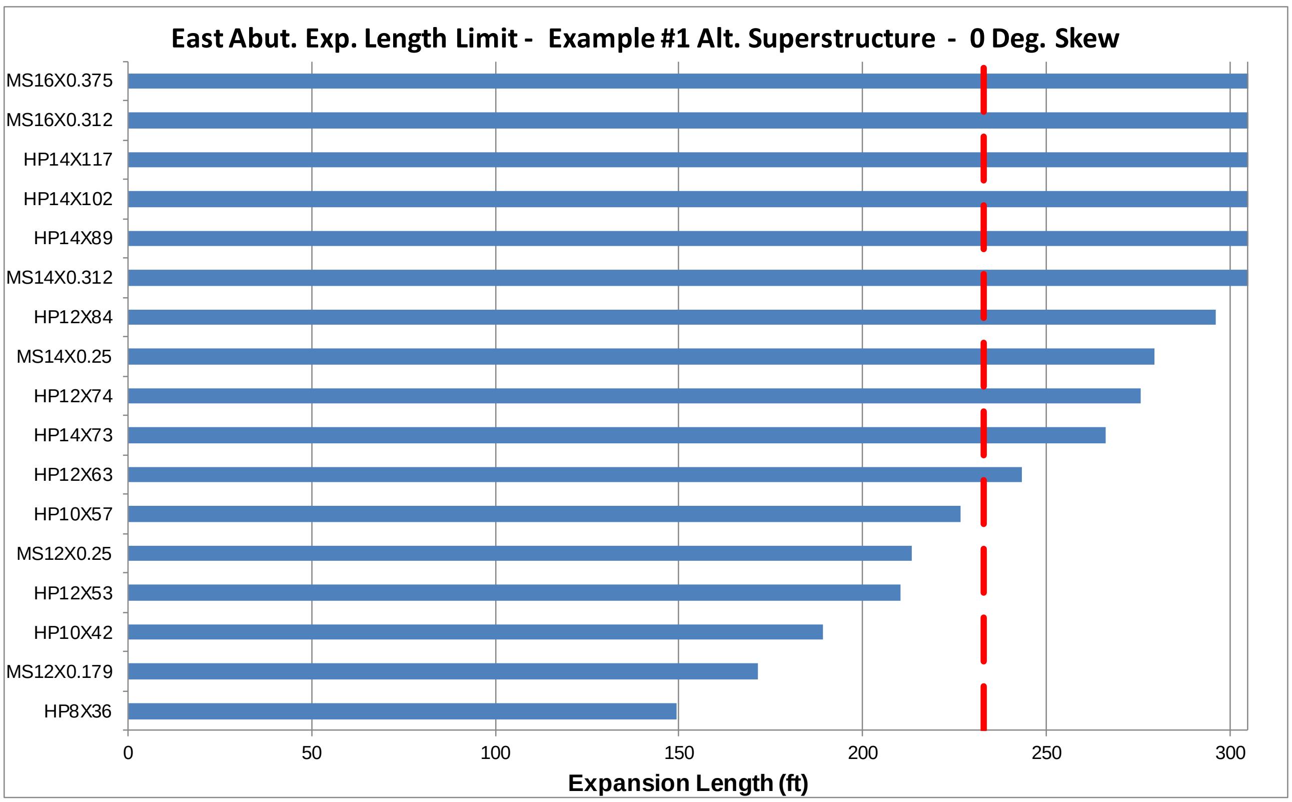
The distance from the centroid of stiffness to the East Abutment is

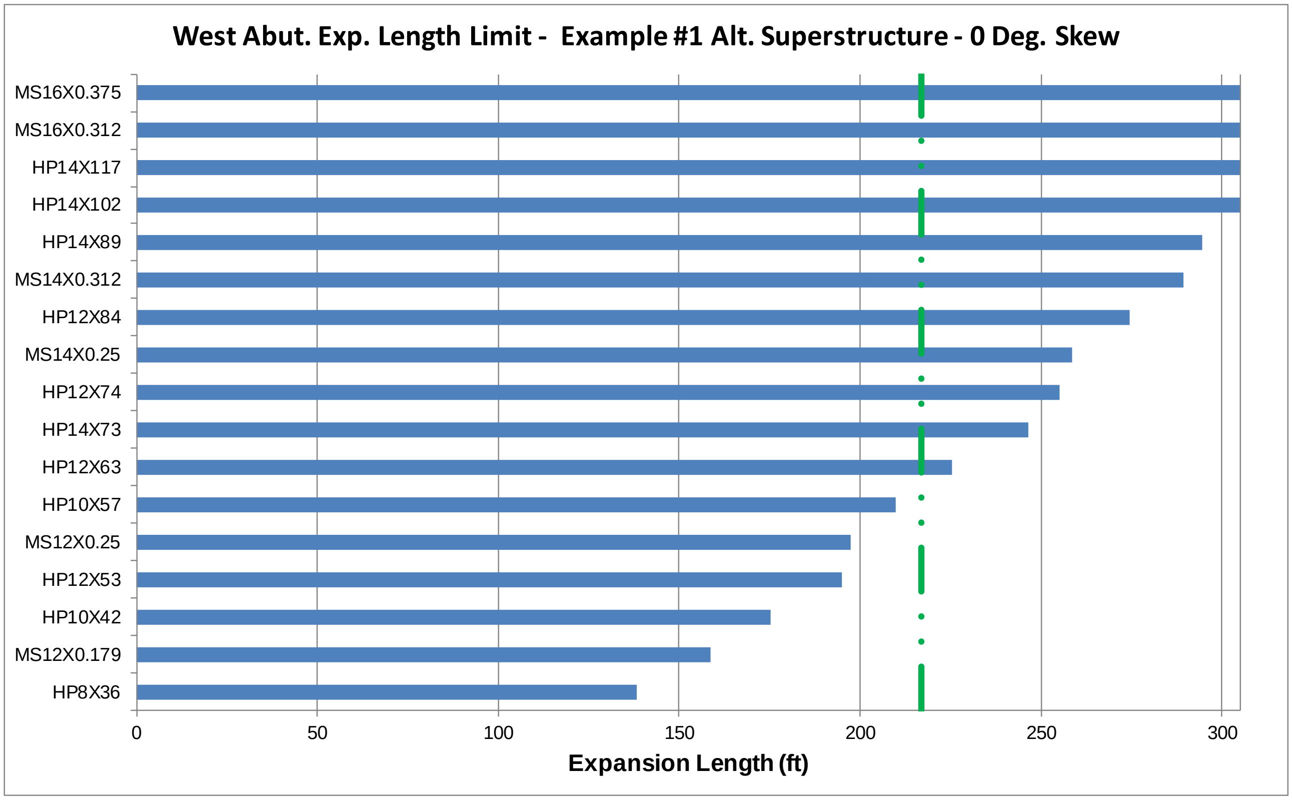
450 - 217 = 233 ft.

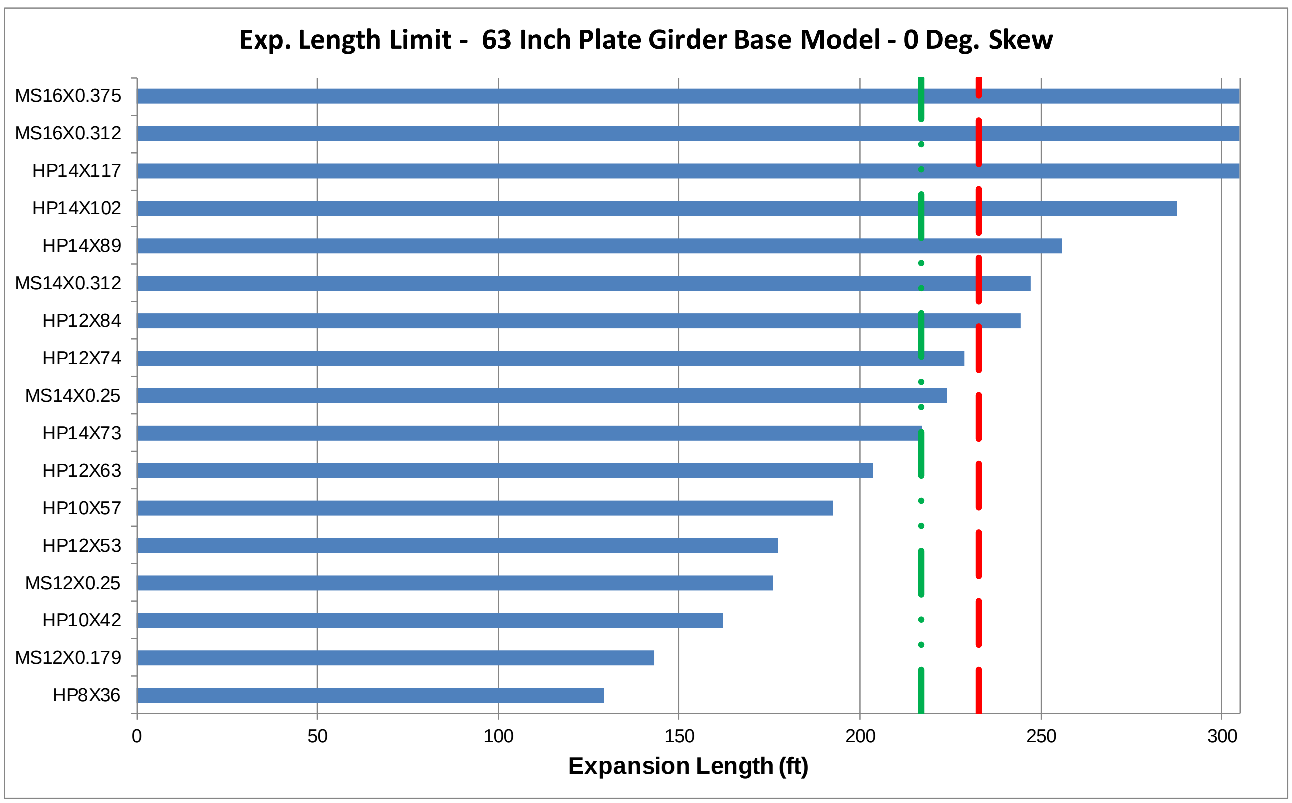
The soil strength correction factor at the east abutment for the displacement capacity and permissible expansion length of the pile is the reciprocal of "M_{east}" calculated above, or 1.08.

The table below shows the base model expansion length factors for each pile as well as the various correction factors. The superstructure stiffness correction factors have been calculated for each pile using the previously described procedure and the alternate superstructure properties for the example. Also shown are pile selection graphs for each abutment with the correction factors incorporated. Piles whose lengths exceed the tributary expansion length are suitable for use. For comparison, the tributary expansion lengths are also plotted on a graph of permissible pile expansion lengths for the base case model that assumes a Q_u of 1.5 tsf.

			EAST ABUTMENT		WEST ABUTMENT	
	BASE MODEL EXP. LENGTH (FT)	SUPERSTRUCTURE STIFFNESS CORRECTION FACTOR	SOIL STRENGTH CORRECTION FACTOR	CORRECTION FACTOR PRODUCT	SOIL STRENGTH CORRECTION FACTOR	CORRECTION FACTOR PRODUCT
HP14X117	305	1.18	1.08	1.27	1	1.18
HP14X102	288	1.16	1.08	1.26	1	1.16
HP14X89	256	1.15	1.08	1.24	1	1.15
HP14X73	217	1.14	1.08	1.23	1	1.14
HP12X84	244	1.12	1.08	1.21	1	1.12
HP12X74	229	1.12	1.08	1.20	1	1.12
HP12X63	204	1.11	1.08	1.19	1	1.11
HP12X53	177	1.10	1.08	1.19	1	1.10
HP10X57	193	1.09	1.08	1.18	1	1.09
HP10X42	162	1.08	1.08	1.17	1	1.08
HP8X36	129	1.07	1.08	1.16	1	1.07
MS12X0.179	143	1.11	1.08	1.20	1	1.11
MS12X0.25	176	1.12	1.08	1.21	1	1.12
MS14X0.25	224	1.16	1.08	1.25	1	1.16
MS14X0.312	247	1.17	1.08	1.26	1	1.17
MS16X0.312	305	1.22	1.08	1.31	1	1.22
MS16X0.375	305	1.24	1.08	1.34	1	1.24




West Abut. Exp. Length Limit - Example #1 Alt. Superstructure - 0 Deg. Skew
MS16X0.375
MS16X0.312
HP14X117
HP14X102
HP14X89
MS14X0.312
HP12X84
MS14X0.25
HP12X74
HP14X73
HP12X63
HP10X57
MS12X0.25
HP12X53
HP10X42
MS12X0.179
HP8X36
0
50
100
150
200
250
300
Expansion Length (ft)


Exp. Length Limit - 63 Inch Plate Girder Base Model - 0 Deg. Skew
MS16X0.375
MS16X0.312
HP14X117
HP14X102
HP14X89
MS14X0.312
HP12X84
HP12X74
MS14X0.25
HP14X73
HP12X63
HP10X57
HP12X53
MS12X0.25
HP10X42
MS12X0.179
HP8X36
0
50
100
150
200
250
300
Expansion Length (ft)

Pile Selection Example 2

Use the same geometric configuration from Example 1 except that the average Qu within the critical pile depth at the east abutment is increased from 1.25 to 2.5 tsf. The following example determines the effective expansion length for the structure and indicates acceptable piles.

Determine the pile stiffness modifier for the east abutment since it has an average Q_u that is not equal to 1.5 tsf.

$$M_{east} = \frac{1}{1.45\text{-}0.3(2.5)} = 1.43$$

Assume 6 beam lines in the structure with a pile placed beneath each beam and calculate the centroid of stiffness from the west abutment.

$$\Sigma_{\text{Stiff. W. Abut.}} = \frac{(6\text{ piles})(0\text{ ft})+(6\text{ piles})(1.43)(450\text{ ft})}{(6\text{ piles})+(6\text{ piles})(1.43)} \approx 265\text{ ft}$$

The distance from the centroid of stiffness to the East Abutment is

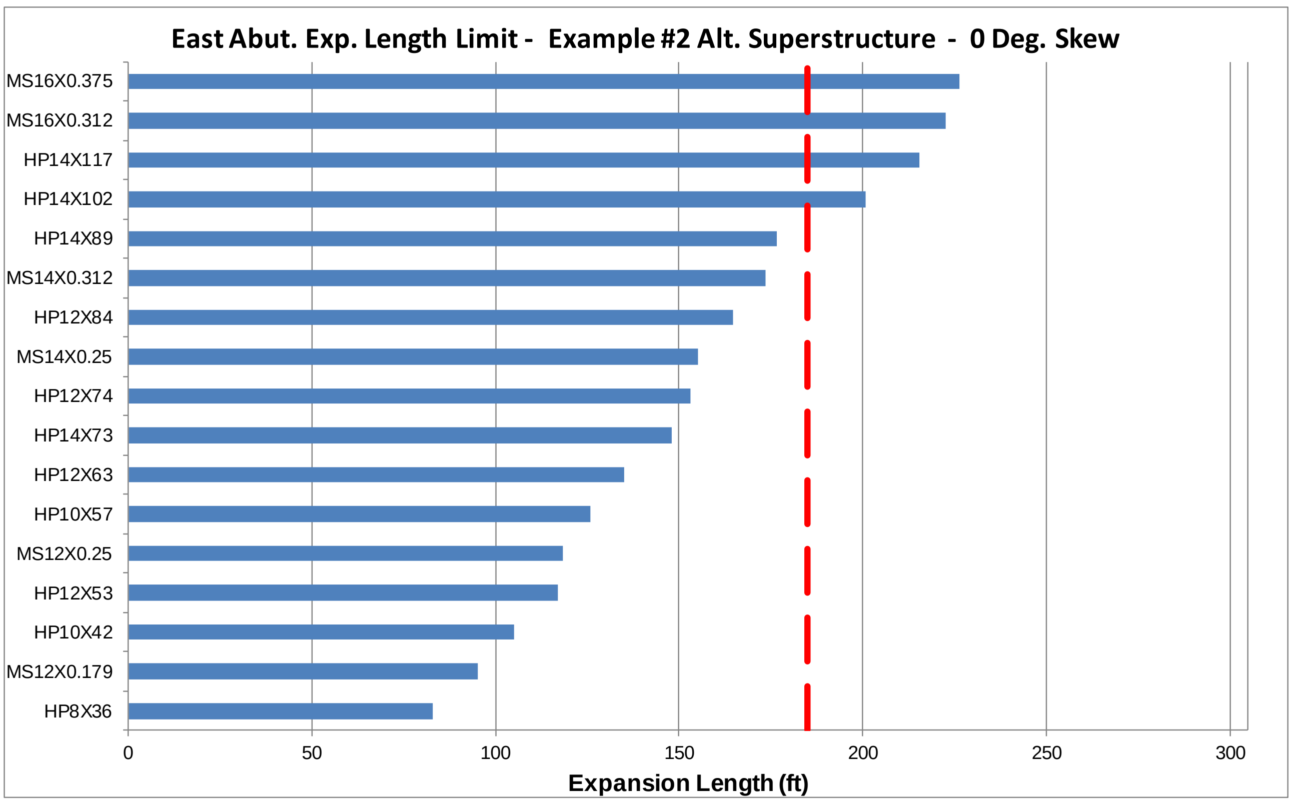
450 - 265 = 185 ft

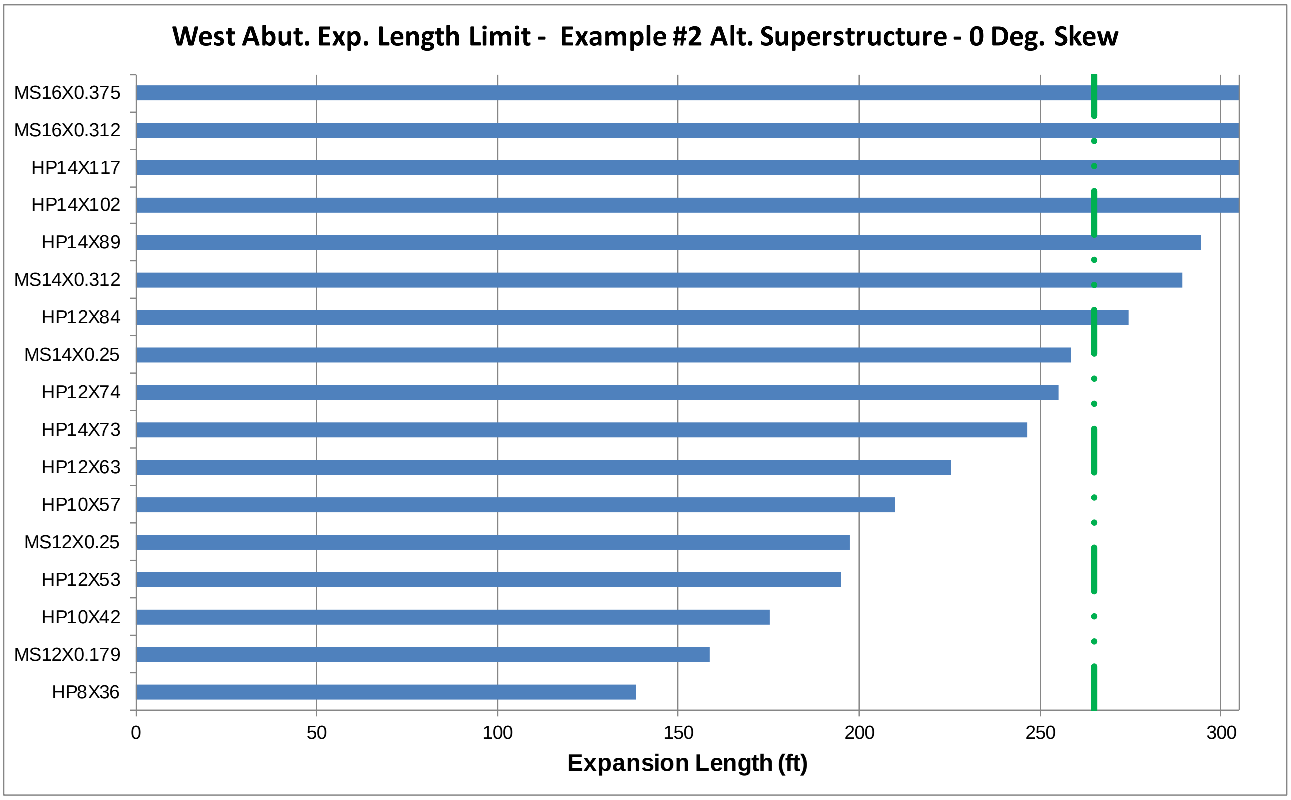
The following soil strength correction factor must be applied at the east abutment for the displacement capacity and permissible expansion length of the pile since the average Q_u for the abutment is greater than 1.5 tsf.

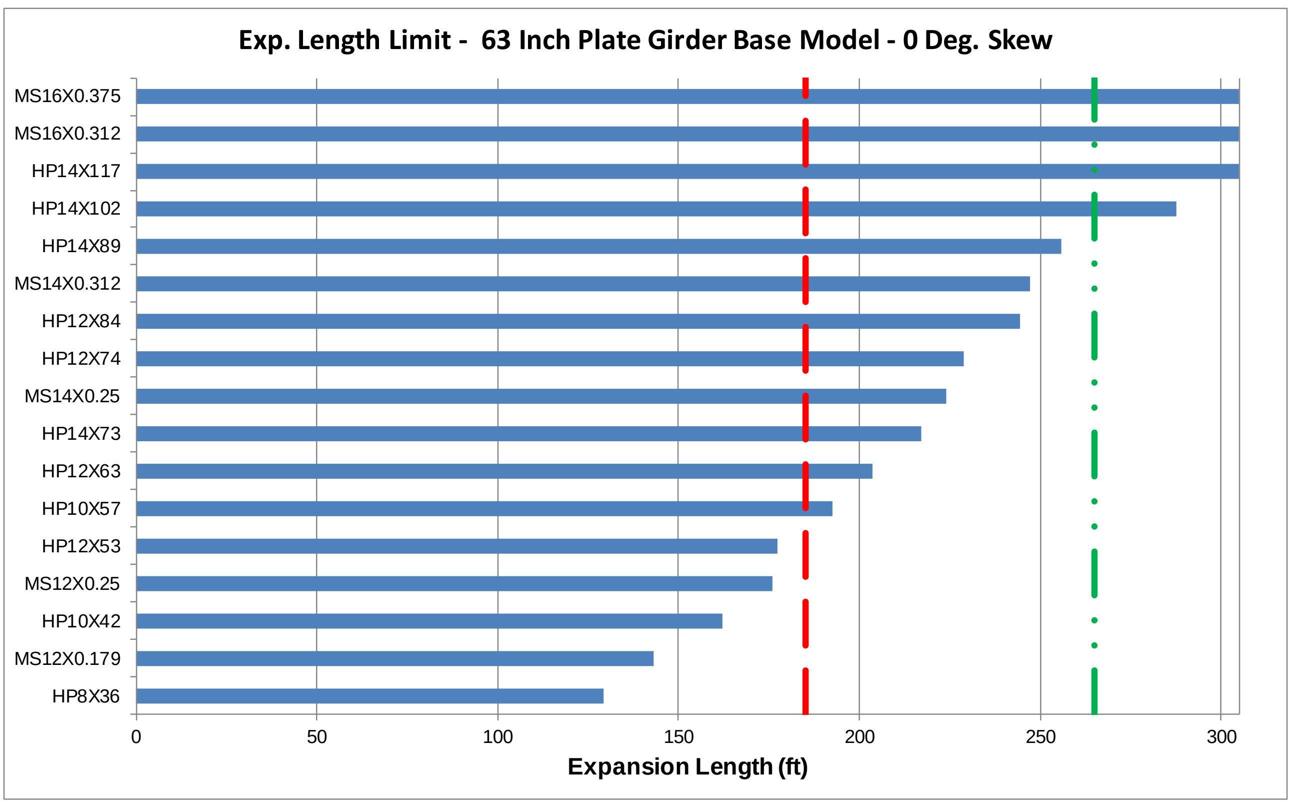
$$\frac{1.5}{2.5} = 0.6$$

Similar to Example 1, the following tables and graph show the various correction factors and corresponding expansion lengths for each pile.

	BASE MODEL EXP. LENGTH (FT)	SUPERSTRUCTURE STIFFNESS CORRECTION FACTOR	EAST ABUTMENT		WEST ABUTMENT	
			SOIL STRENGTH CORRECTION FACTOR	CORRECTION FACTOR PRODUCT	SOIL STRENGTH CORRECTION FACTOR	CORRECTION FACTOR PRODUCT
HP14X117	305	1.18	0.6	0.71	1	1.18
HP14X102	288	1.16	0.6	0.70	1	1.16
HP14X89	256	1.15	0.6	0.69	1	1.15
HP14X73	217	1.14	0.6	0.68	1	1.14
HP12X84	244	1.12	0.6	0.67	1	1.12
HP12X74	229	1.12	0.6	0.67	1	1.12
HP12X63	204	1.11	0.6	0.66	1	1.11
HP12X53	177	1.10	0.6	0.66	1	1.10
HP10X57	193	1.09	0.6	0.65	1	1.09
HP10X42	162	1.08	0.6	0.65	1	1.08
HP8X36	129	1.07	0.6	0.64	1	1.07
MS12X0.179	143	1.11	0.6	0.67	1	1.11
MS12X0.25	176	1.12	0.6	0.67	1	1.12
MS14X0.25	224	1.16	0.6	0.69	1	1.16
MS14X0.312	247	1.17	0.6	0.70	1	1.17
MS16X0.312	305	1.22	0.6	0.73	1	1.22
MS16X0.375	305	1.24	0.6	0.74	1	1.24

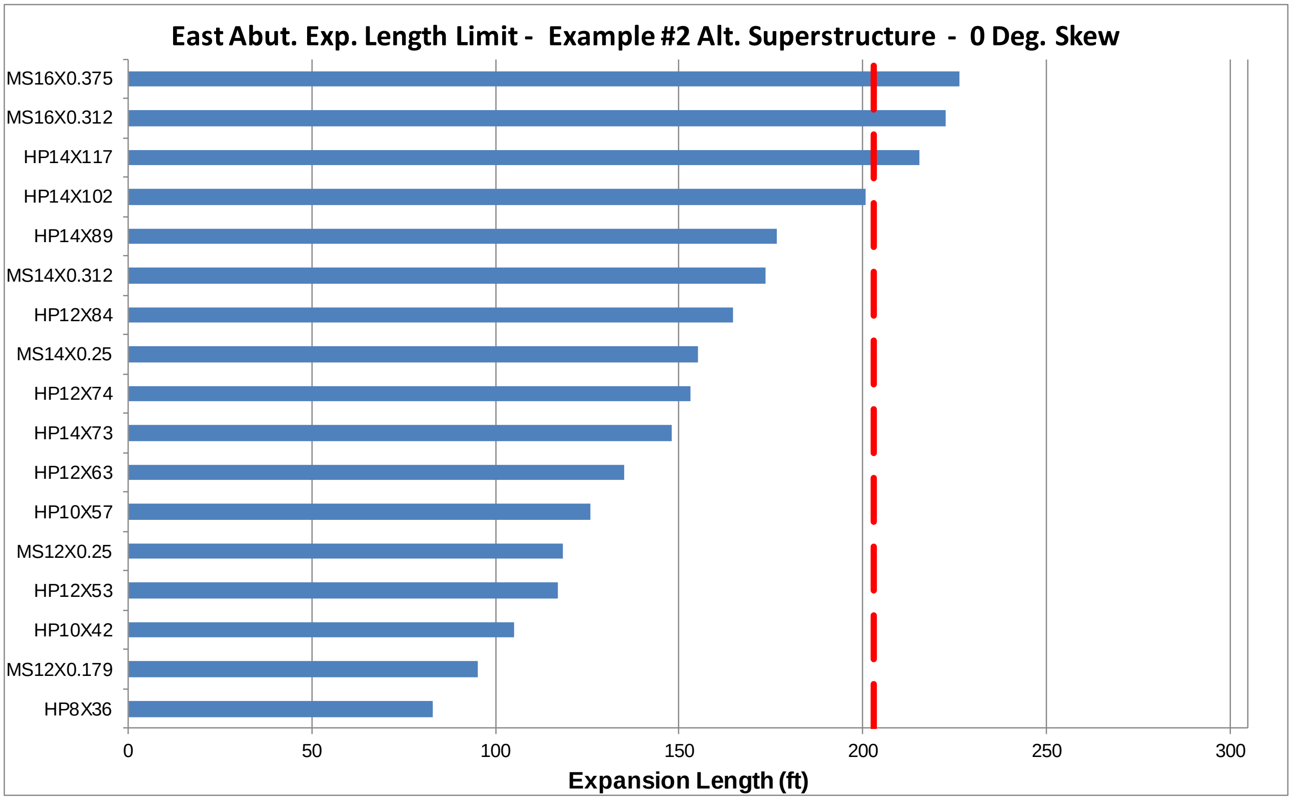


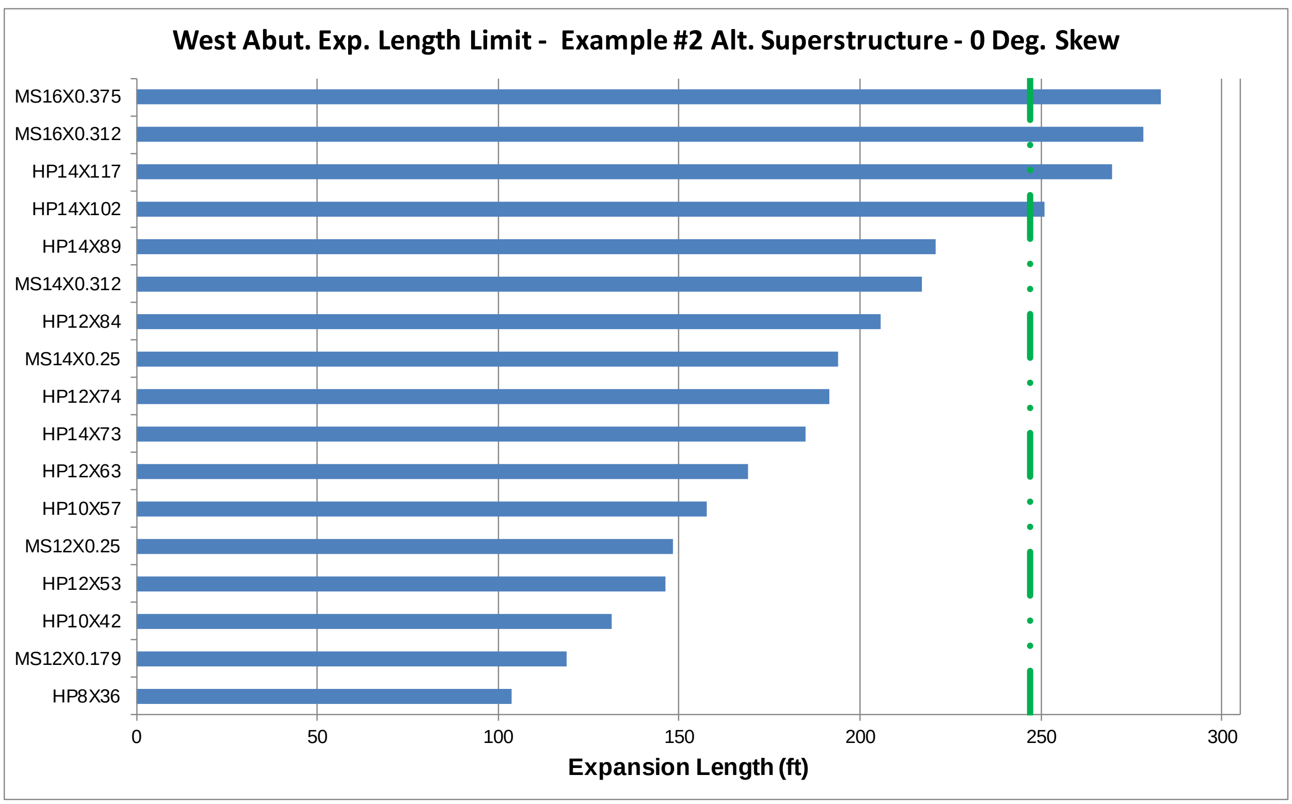

West Abut. Exp. Length Limit - Example #2 Alt. Superstructure - 0 Deg. Skew
MS16X0.375
MS16X0.312
HP14X117
HP14X102
HP14X89
MS14X0.312
HP12X84
MS14X0.25
HP12X74
HP14X73
HP12X63
HP10X57
MS12X0.25
HP12X53
HP10X42
MS12X0.179
HP8X36
0
50
100
150
200
250
300
Expansion Length (ft)


Exp. Length Limit - 63 Inch Plate Girder Base Model - 0 Deg. Skew
MS16X0.375
MS16X0.312
HP14X117
HP14X102
HP14X89
MS14X0.312
HP12X84
HP12X74
MS14X0.25
HP14X73
HP12X63
HP10X57
HP12X53
MS12X0.25
HP10X42
MS12X0.179
HP8X36
0
50
100
150
200
250
300
Expansion Length (ft)

<u>Note</u>: If Example 2 had an average Q_u within the critical pile depth of 2.0 tsf at the west abutment and 2.5 tsf at the east abutment, the west abutment would also require a pile stiffness modifier which results in the distance from the centroid of stiffness to the west abutment and controlling expansion length increasing to approximately 247 ft. However, the permissible expansion length for the piles would also need to be adjusted for the Q_u correction factor at the west abutment by multiplying by the ratio of 1.5/2.0 (= 0.75). The following tables and graph show the various correction factors and corresponding expansion lengths for each pile. Conversely, if the controlling expansion length is divided by the Q_u correction factor for comparison with the pile limits for the base case and Q_u of 1.5 tsf (i.e., 247/0.75), this would result in an effective expansion length of approximately 329 ft which exceeds the maximum length of 305 ft and suggests the structure is unacceptable for integral abutments. However, by considering the benefit of the increased flexibility of the alternative superstructure for the subject example, the structure is able to utilize integral abutments.

			EAST ABUTMENT		WEST ABUTMENT	
	BASE MODEL EXP. LENGTH (FT)	SUPERSTRUCTURE STIFFNESS CORRECTION FACTOR	SOIL STRENGTH CORRECTION FACTOR	CORRECTION FACTOR PRODUCT	SOIL STRENGTH CORRECTION FACTOR	CORRECTION FACTOR PRODUCT
HP14X117	305	1.18	0.6	0.71	0.75	0.88
HP14X102	288	1.16	0.6	0.70	0.75	0.87
HP14X89	256	1.15	0.6	0.69	0.75	0.86
HP14X73	217	1.14	0.6	0.68	0.75	0.85
HP12X84	244	1.12	0.6	0.67	0.75	0.84
HP12X74	229	1.12	0.6	0.67	0.75	0.84
HP12X63	204	1.11	0.6	0.66	0.75	0.83
HP12X53	177	1.10	0.6	0.66	0.75	0.82
HP10X57	193	1.09	0.6	0.65	0.75	0.82
HP10X42	162	1.08	0.6	0.65	0.75	0.81
HP8X36	129	1.07	0.6	0.64	0.75	0.80
MS12X0.179	143	1.11	0.6	0.67	0.75	0.83
MS12X0.25	176	1.12	0.6	0.67	0.75	0.84
MS14X0.25	224	1.16	0.6	0.69	0.75	0.87
MS14X0.312	247	1.17	0.6	0.70	0.75	0.88
MS16X0.312	305	1.22	0.6	0.73	0.75	0.91
MS16X0.375	305	1.24	0.6	0.74	0.75	0.93


East Abut. Exp. Length Limit - Example #2 Alt. Superstructure - 0 Deg. Skew
MS16X0.375
MS16X0.312
HP14X117
HP14X102
HP14X89
MS14X0.312
HP12X84
MS14X0.25
HP12X74
HP14X73
HP12X63
HP10X57
MS12X0.25
HP12X53
HP10X42
MS12X0.179
HP8X36
0
50
100
150
200
250
300
Expansion Length (ft)


West Abut. Exp. Length Limit - Example #2 Alt. Superstructure - 0 Deg. Skew
MS16X0.375
MS16X0.312
HP14X117
HP14X102
HP14X89
MS14X0.312
HP12X84
MS14X0.25
HP12X74
HP14X73
HP12X63
HP10X57
MS12X0.25
HP12X53
HP10X42
MS12X0.179
HP8X36
0
50
100
150
200
250
300
Expansion Length (ft)

Example 3

This example is similar to Example 2 (a continuous 450 ft. long structure consisting of 3 – 150 ft. spans; average Qu at west abutment = 1.5 tsf and average Qu at east abutment = 2.0 tsf), except the structure is flared. The west abutment is wider than the east abutment and has 10 piles compared to 6 piles at the east abutment.

Determine the centroid of stiffness from the west abutment.

$$\Sigma_{\text{Stiff.W. Abut.}} = \frac{(10\text{ piles})(0\text{ ft.})+(6\text{ piles})(1.18)(450\text{ ft.})}{(10\text{ piles})+(6\text{ piles})(1.18)} = 186.5\text{ ft.}$$

The distance from the centroid of stiffness to the centerline of the east abutment is 263.5 ft. and is the controlling expansion length. However, because the Qu at the east abutment is 2.0 tsf, the Qu correction factor would cause the EEL to be:

$$(263.5\text{ ft.})\,\frac{(2.0\text{ tsf.})}{(1.5)} = 351.3\text{ ft.}$$

The Integral Abutment Pile Selection Chart indicates that this structure cannot be integral.