Rehabilitation of I-95 from I-495 to North of Brandywine River Bridge
Ramp D: W 2nd Street to Southbound Interstate I-95

Delaware Department of Transportation
Wilmington, DE

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Geotechnical Engineering Report, Revision 2

TABLE OF CONTENTS

LIST OF TABLES

LIST OF APPENDICES

EXECUTIVE SUMMARY

The new Ramp D will require replacing Bridge 1-750 over Lancaster Ave. and widening Bridge 1-748S between Pier 33S and Pier 40S. This Geotechnical Engineering Report includes a summary of site explorations, subsurface conditions and recommendations for new foundations and mechanically stabilized earth (MSE) retaining walls to support the new ramp. Key items discussed in this report include:

1. **Subsurface Conditions:** Subsurface conditions are based on material encountered in twelve soil test borings with rock coring. Subsurface conditions consist of fine and coarse-grained sediment deposited over a residual soil mantle. The residual soil mantle has weathered in place from the parent metamorphic gneiss bedrock encountered at depths varying from 20 to 40 feet below the ground surface.

2. **Steel Piles:** Foundations for the new Bridge 1-750 abutments and Bridge 1-748S widening abutment are recommended as HP12x53 steel piles bearing in weathered rock or rock. Piles should be installed with steel pile points/shoes to protect the pile tip from damage during driving. Pile tip elevations are estimated at 3 to -10 feet. The estimated tip elevations correspond to pile lengths between 35 to 45 feet. The upper surface of weathered rock and rock is undulating across the site, actual pile lengths will vary.

3. **Pile Driving:** Piles should be driven to a nominal resistance of 385 kips. The nominal resistance equates to a 250-kip factored resistance using a resistance factor of 0.65. Dynamic pile testing is recommended on the first pile driven at each substructure unit to check for excessive pile stresses during driving and to determine pile driving criteria for subsequent production piles. Hammer energy to install piles to nominal resistance is estimated at 40 to 70 kip-feet.

4. **Micropiles:** Pier 34D through 39D are recommended on micropiles with a factored design resistance of 200 kips. The micropiles are 9 5/8-inch diameter with a bond length of 20.5 feet and casing plunge length of 1-foot. The bond length is designed as a pressure grouted installation in either residual soil or rock.

5. **Settlement:** Wall and embankment settlements will vary along the length of the planned MSE embankment but are expected to be in the range of 0.5-inches. An additional 1-inch of consolidation settlement is possible at Abutment 40D. Due to the variability of settlements, wall facings should be designed to withstand a differential settlement of 1/300 in/in

6. **MSE Embankment:** The MSE embankment is constructed as back-to-back MSE walls. MSE walls shall be designed by an MSE wall designer/builder. Maximum exposed height of the MSE wall is 26 feet. Minimum MSE wall strap lengths vary according to wall height and are listed in this report.

7. **Downdrag:** Due to settlement of the MSE wall embankment, approximately 55 kips of factored downdrag is possible at Abutment 40D. The factored downdrag resistance is less than live load on the structure. According to the DelDOT Bridge Design Manual this structure should not be designed for downdrag.

1.0 INTRODUCTION

This Revised Geotechnical Report describes geotechnical elements of planned Ramp D: W 2nd Street to southbound interstate I-95 in Wilmington, Delaware. The revised report addresses comments to the Semi-Final Design submission.

Ramp D will replace existing Ramp B and Ramp C and require demolition of Bridge 1-758, widening of Bridge 1-748S and replacement of Bridge 1-750. The inclined portion of the planned Ramp D is steeper than the existing ramps resulting in a merge to I-95 Wilmington Viaduct approximately 400 feet north of the current merge. The gain in elevation will allow for a longer acceleration lane and increased visibility of vehicles entering the interstate. Ramp D is part of the larger project to rehabilitate Interstate I-95 from I-495 to North of Brandywine River Bridge. The planned ramp construction is the alternative design to widening Bridge Nos. 1-748 and 1-748S south of existing Pier 33S, over the northeast corridor Amtrak rail lines

1.1 Site Description

The new Ramp D is planned in an area currently occupied by two onramps (Ramp B and Ramp C). A site location map is included in Appendix A. The existing Ramp B slopes up from an elevation of roughly 33 feet at S. Jackson St. to elevation 45 feet at the Bridge 1-758 abutment. Bridge 1-758 elevates the ramp to the elevation of southbound I-95 (Bridge 1-748S) near elevation 63 feet and terminates at Bridge 1-748S Pier 33S, north of the intersection of Maryland Ave. and Linden St. The existing Ramp C slopes up on an earth embankment from elevation 26 feet at W 2nd St. to elevation 38 feet at the north abutment of Bridge 1-750 over Lancaster Ave. The south abutment of Bridge 1-750 is near elevation 46 feet. The ramp dips down slightly moving south over the second portion of the earth embankment to elevation 45 feet at the 1-758 bridge abutment and merges with Ramp B. Figure 1-1 is a map of the site area with existing structures labeled.

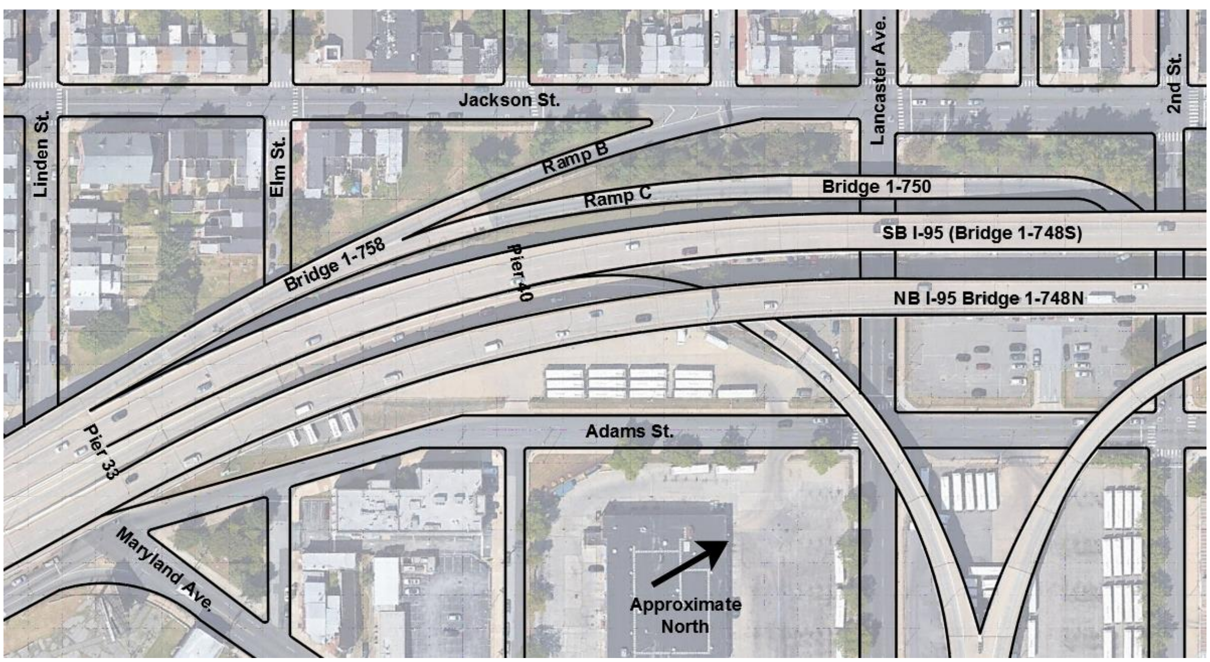

Figure 1-1: Existing Structures and Area Map.

Portions of Ramp C between W 2nd St. and Bridge 1-750 and between Bridge 1-750 and Bridge 1-758 are supported on earth embankments. The east embankment slopes are mowed grass sloping down to a parking lot underneath the viaduct. North of Lancaster Ave., the west embankment slopes are mowed grass and trees that slope down to N. Jackson St. South of Lancaster Ave., the west embankment slopes down from Ramp C to Ramp B; this area is mostly dense shrub and weeds with some small trees. The area west of Ramp B is grass and trees that slope gently down to S. Jackson St. Existing embankment slopes vary across the site but are generally graded at a 2.5(H):1(V) slope or less.

Multiple roadway drains are located throughout the site. The drains collect surface runoff water from both the at-grade and bridge ramp roadways. Drain locations may intercept excavations for new foundations; drain locations should be verified before construction.

Two existing sewer pipes cross under the raised Interstate I-95 and on-ramps near the project site. The first pipe is a 12-inch terracotta pipe flowing to the east and buried inline and near the center of Chestnut St. The pipe continues south from Chestnut St. under S. Jackson St. and existing Ramp B and Ramp C about 30 feet north of the existing onramp Bridge 1-758 abutment. The existing pipe appears to have been re-routed around southbound I-95 Bridge 1-748S Pier 40 with an added stretch diverting south and a second stretch back to the original alignment just east of Pier 40. The alignment continues to the east, under S. Adams St. to Chestnut St.

The second pipe is a 10-inch terracotta pipe flowing to the southeast and buried in line with Read St. The pipe crosses under the Ramp B entrance from S. Jackson St and under Ramp C approximately 90 feet south of the existing Bridge 1-750 south abutment. The pipe continues in a straight light past S. Adams St. The pipe has an invert elevation of 24.44 feet at the manhole northwest of the ramps.

1.2 Existing Bridge Structures

The original Bridge 1-748, 1-748N and 1-748S and adjacent on/off-ramps were built in 1964. Bridge 1-748 was widened in 1978. The bridges were rehabilitated in 2000 and 2004 to address bearing and joint deficiencies, install seismic retrofits and perform miscellaneous repairs to extend the service life of each bridge.

Bridge 1-748S (I-95 Wilmington Viaduct) Pier 33S through Pier 40S are located adjacent to the planned Ramp D area and are constructed of steel girders supported on concrete rigid frame piers. Typical span lengths between bridge piers range from 68 to 75 feet. Span lengths at W 2nd St., Lancaster Ave. and Maryland Ave. crossings are 94, 80, and 137.5 feet respectively.

Bridge 1-750 (Ramp C over Lancaster Ave.) is 3 spans with approximate span lengths of 40, 78 and 28 feet from north to south for a total abutment-to-abutment length of about 146 feet. The bridge is constructed of steel girders on concrete abutments and concrete hammerhead type piers. Existing bridge drawings show each abutment supported by 19 total piles divided into two rows.

Bridge 1-758 (Ramp B to Southbound I-95) is 7 spans with approximate span lengths of 50, 60, 60, 85, 73, 70 and 64.5 feet from north to south for a total length of about 462.5 feet. The bridge is supported at the north end by an independent abutment located at the south end of the Ramp B earth embankment. Existing bridge drawings show the bridge abutment as 53' – 9 ½" wide at the front face. The abutment is supported by 28 total piles arranged in two rows with the front row battered with pile tips pointed to the south. South of the abutment the bridge is supported by six independent piers. The northern most bridge piers (Pier 1B and Pier 2B) are concrete wall piers supported by 16 piles per pier. The remaining piers (Pier 3B through 6B) are concrete hammerhead style piers supported by 15 piles per pier. The piers vary in height and foundation size. Bottom of pile cap elevations vary from 30 to 35 feet. The south end of the southernmost span is supported by southbound I-95 Bridge 1-748S

Pier 33S. Pier 33S is a three-column concrete rigid frame pier. Drawings show the pier supported by 33 total plumb and battered piles.

Existing pier and abutment bridge foundations near the new Ramp D are supported on plumb and battered piles except for 1-78S Pier 45S and 1-750 Pier 1C and Pier 2C which are supported by spread foundations. The existing piles are concrete-filled 12-inch diameter close-ended steel monotube piles with a listed design capacity of 40 tons. Typical pile details show the uppermost 10-feet of the piles reinforced with six #7 longitudinal reinforcing bars and ¼-inch spiral steel reinforcement. Existing plan drawings list estimated pile lengths for each pile foundation. Typical pile batter is 1(H):4(V). Spread footing foundations have a listed design bearing capacity of 2 tons per square foot.

Existing pier and abutment elevations around the site are listed in Table 1-1.

Table 1-1: Summary of Existing Deep Foundations at Southbound (Westernmost) Columns.

Substructure Unit	Elevation (Feet)			
	Top of Pier Cap or Abut. Seat	Approximate Ground Surface	Bottom of Pile Cap	Estimated 12" Dia. Monotube Pile Tip
1-750 North Abut.	35.62 – 36.43	37	29.0	4.0
1-750 Pier 1C	36.62	25	14.5	--
1-750 Pier 2C	40.57	25	17.0	--
1-750 South Abut.	42.53 – 43.14	45	35.5	10.5
1-758 Abut.	41.94 – 43.39	45	35.0	10.0
1-758 Pier 1B	44.00 – 45.20	40	34.0	9.0
1-758 Pier 2B	47.75	40	35.0	10.0
1-758 Pier 3B	51.00	39	35.0	5.0
1-758 Pier 4B	55.58	38	34.0	4.0
1-758 Pier 5B	58.00	37	33.0	3.0
1-758 Pier 6B	58.64	35	30.0	0.0
1-748S, Pier 33S	53.00	32	26.5	-3.5

1.3 Proposed Construction

The entrance to Ramp D will be from the same Ramp C entrance location at W 2nd St. The grade of Ramp D will be steeper than the existing ramp (and will require a design exception). The planned Ramp D requires replacement of Bridge 1-750, demolition of Bridge 1-758, widening of Bridge 1-748S from Pier 40S to Pier 33S and construction of an MSE embankment on the south side of Bridge 1-750. The length of the Ramp D MSE embankment is 344'-7 3/4" measured along the baseline Ramp D.

The new Bridge 1-750 is planned as a single-span with an approximate length of 108 feet. The new bridge will be designed without joints, supported on fully integral pile-supported abutments located between the existing abutments and piers 1C and 2C.

The new back-to-back MSE wall at Bridge 1-750 north abutment (Abutment A) extends north of the abutment for a length of 50 feet. The back-to-back MSE wall south of Bridge 1-750 extends for a length of 356'-3 ½" feet measured from the front face wall at Bridge 1-750 Abutment B (Wall B2) to the front face of the wall at Bridge 1-748S Abutment 40D (Wall B4). The MSE walls will bear on the existing earth embankment fill. The exposed height of the MSE wall increases from approximately 9 feet at the north end to about 26 feet at the southeast corner. General plan and elevation drawings for Ramp D and roadway sections along the MSE wall portion of the ramp are included in Appendix A.

Widening of Bridge 1-748S will include one abutment constructed at the south end on the MSE embankment and six new piers. Abutment 40D is planned west of existing 1-748S Pier 40. Piers 39D through 34D are planned west of existing Bridge 1-748S Pier 39 through Bridge 1-748 Pier 34S, north to south.

2.0 GEOLOGY

Geologic information provided in this report is a composite of information obtained from a bedrock geologic map of Delaware (Schenck, Plank and Srogi, 2000) and a geologic map of the Wilmington area (Woodruff and Thompson 1975). A geologic site map is included in Appendix A. At the area of the project site, geologic maps show relatively thin layers of recent sediment overlying metamorphic rock of the Piedmont Province. Sediments around the project site consist of relatively recent, Holocene age fine sand, silt and clay and Pleistocene age sand and gravel with interbedded silt layers. Holocene deposits are derived from nearby weathered Piedmont crystalline rocks. Pleistocene deposits are fluvial deposits derived from glaciated areas to the north. Quaternary isopachs, or lines of equal depth of Quaternary sediments, show sediment thickness at about 20 feet near Maryland Ave., decreasing to about 0 feet near Lancaster Ave.

Geologic mapping indicates that the sediments are underlain by Cambrian-Ordovician period Brandywine Blue Gneiss of the Wilmington Complex. The rock is a blue to dark gray, medium to coarse-grained, massive, felsic banded gneiss and granulite containing plagioclase, quartz and pyroxene with thin, fine grained mafic layers. Typically rock in the Piedmont is covered by rock that has undergone chemical and physical weathering to form a mantle of residual soil and decomposed rock that retains the structure of the parent rock. The groundwater surface in the Piedmont varies seasonally but often resembles a subdued replica of the ground surface.

3.0 SUBSURFACE INVESTIGATION

3.1 Historic Boring Logs

Twenty soil test borings were performed between Pier 33 and Pier 50 in the summer of 1960 during the original design of the viaduct. The location and log records of the historic borings are shown on historic bridge drawings of Interstate I-95 South Wilmington Viaduct, Contract 64-03-004. Locations of the historic test borings are listed in Table 3-1.

Table 3-1: Location of Historic Test Borings: Station, Offset and Coordinates.

Boring	Ground Surface Elevation (Feet)	I-95 Baseline Station	Offset	Coordinates	
				Northing (Feet)	Easting (Feet)
135	33.7	1300+84.32	7.02 RT	633595	615144

Boring	Ground Surface Elevation (Feet)	I-95 Baseline Station	Offset	Coordinates	
				Northing (Feet)	Easting (Feet)
136	34.6	1302+11.09	99.09 RT	633716	615241
137	38.7	1302+56.27	0.85 LT	633767	615145
138	35.8	1304+53.29	81.43 RT	633947	615259
139	30.8	1307+6.64	34.23 LT	634216	615227
140	20.2	1308+62.02	50.08 LT	634365	615280
141	26.7	1310+36.11	50.04 LT	634520	615361
142	28.4	1311+80.13	47.97 RT	634602	615515
207	43.5	1304+47.33	70.59 LT	633972	615109
208	37.9	1305+63.99	47.43 LT	634083	615164
209	40.4	1305+44.55	130.46 LT	634088	615078
211	30.9	1307+77.90	48.15 LT	634287	615244
212	27.9	1307+77.65	45.85 RT	634248	615329
213	25.5	1308+48.40	88.09 LT	634370	615240
214	23.5	1308+48.14	18.08 LT	634338	615302
215	23.2	1309+27.09	22.05 LT	634411	615335
216	21.4	1309+28.11	0.05 LT	634401	615355
217	24.5	1310+02.11	46.96 RT	634445	615431
218	23.6	1310+02.11	46.96 RT	634445	615431
226	35.3	1311+51.13	50.54 LT	634622	615414

3.2 Additional Subsurface Exploration

Twelve soil test borings (17B-R1 through 17B-R12) were performed along the planned Ramp D alignment in June and July 2017. Locations of the current test borings are listed in Table 3-2. Boring locations were approximated by triangulating distances from on site features. Previously, twenty-one borings (16B-P13 through 16B-P33) were performed in the fall of 2016 to investigate conditions for a previously planned viaduct widening. Two of the 2016 borings, 16B-P30 and 16B-P33, are mentioned in this report. Boring WB-16 was performed to investigate conditions for light and sign structures southwest of Bridge I-750 and is included in this report.

The current borings were performed by Walton Corporation using multiple drill rigs. The borings were advanced into the ground by either mechanically rotating hollow stem augers or mud rotary drilling. Standard penetration testing (SPT) was performed continuously in the upper 20 feet of the boring and then at 5-foot intervals to auger refusal or drilling termination. Borings at bridge foundation locations were drilled to four consecutive samples in residual soil with blow counts exceeding 50 blows per foot or cored 10 continuous feet with RQD greater than 50%. Borings along the new MSE wall alignment were drilled to a termination depth of 50 feet or auger refusal. Borings B17-R11 and B17-R12 were drilled to boring termination depths of 20 and 30 feet, respectively.

Sampling and testing were performed in general accordance with ASTM D 1586: a split-spoon sampler was driven into the ground with blows from a 140-pound hammer falling 30 inches. The hammer was first seated 6 inches into the ground then driven three additional 6-inch intervals. The blows required to drive the hammer the second and third intervals were summed and recorded as the standard penetration resistance (N-value). N-values are used to estimate the consistency or relative density of subsurface soil strata. The hammer type for the drill rig was a rope safety hammer with cathead assist. The energy transfer efficiency of the rope safety hammer is generally assumed as 60%, the industry standard for N-value correlations and comparison. Rock coring was performed in multiple borings by rotating a diamond-tip core barrel into the rock mass.

Representative soil profiles and individual boring log records are included in Appendix B. Soil and rock descriptions listed on the DelDOT boring logs are based on laboratory inspection of the soil and rock samples and results of soil classification testing. Soil types listed on the soil profiles are field descriptions made by a WRA engineer at the time of drilling.

Table 3-2: Elevation and Location of Current Test Borings: Station, Offset and Coordinates

Boring	Approximate Ground Surface Elevation (Feet)	From 1-95 Centerline		Coordinates		Boring Depth (Feet)
		Station	Offset (Feet)	Northing (Feet)	Easting (Feet)	
16B-P30	29.0	1297+52.25	74.41 LT	633264	615063	53.5
16B-P33	32.0	1300+38.01	82.65 LT	633550	615054	36.1
17B-R1	35.0	1300+93.55	57.60 LT	633606	615080	47.5
17B-R2	37.8	1301+73.06	60.89 LT	633688	615079	55.5
17B-R3	38.5	1302+17.46	64.82 LT	633734	615078	49.5
17B-R4	38.5	1302+49.02	65.10 LT	633767	615081	48.2
17B-R5	38.8	1303+43.16	65.53 LT	633864	615093	52
17B-R6	44.0	1305+15.00	108.87 LT	634051	615090	47.1
17B-R7	40.5	1305+96.97	118.05 LT	634138	615107	35.25
17B-R8	32.5	1307+32.57	32.42 LT	634239	615239	27.0
WB-16	31.0	1307+92.37	152.10 LT	634344	615155	25.5
17B-R9	28.2	1308+33.85	33.78 LT	634333	615282	52.0
17B-R10	26.0	1309+56.69	44.39 LT	634447	615329	32.5
17B-R11	26.9	1311+30.70	34.10 LT	634596	615419	31.0
17B-R12	21.0	1311+25.89	102.79 RT	634528	615538	28.3

3.3 Laboratory Testing

Soil classification testing was performed by DelDOT during preparation of test boring logs. Soil descriptions included in the boring logs reflect laboratory soil classification and results of Atterberg moisture limit and gradation testing. Engineering & Testing Services, Inc. (ETS) performed soil pH and resistivity testing on select soil samples from multiple boring locations and one direct shear strength test on an undisturbed sample of fine-grained soil deposits encountered in boring 17B-R7. ETS also performed multiple uniaxial compressive strength testing of select rock core samples.

Results from completed pH and resistivity testing are listed below in Table 3-3; results from completed uniaxial compressive strength tests are listed in Table 3-4. Complete laboratory test results from ETS are included in Appendix C.

Table 3-3: Summary of pH and Resistivity Laboratory Results.

Boring	Sample	Sample Depth (Feet)	Measured pH	Measured Resistivity (ohms/cm)
16B-P30	S-4	6 – 8	4.7	2500
17B-R2	S-6	12 – 14	6.8	2045
17B-R4	S-4	8 – 10	5.7	2421
17B-R6	S-2	4 – 6	5.8	1650
17B-R9	S-5	8 – 10	7.8	1490
17B-R10	S-3	4 – 6	6.8	2300

Table 3-4: Summary of Compressive Strength Test Results.

Boring	Sample Depth (Feet)	Compressive Strength (PSI)
16B-R2	45 – 50	35,536
16B-R3	39 – 44	19,015
16B-R5	47 – 52	25,346
16B-R6	37 – 42	15,848
16B-R9	42 – 47	22,396
16B-R10	22 – 27	14,467

4.0 SUBSURFACE CONDITIONS

4.1 Stratigraphy (General Subsurface Conditions)

Subsurface material encountered on site is comprised of fill material, recent coarse and fine-grained sediments, residual soil, decomposed rock and rock.

4.1.1 Fill (F)

Fill is any material placed by humans. Fill is typically sourced from local sites and may resemble undisturbed site soils, but often includes deleterious material such as crushed stone, wood fragments, construction debris and organic matter. Fill material was encountered in each of the current borings and most of the historic borings.

Current borings encountered between 5 to 15 feet of fill with an average depth around 10 feet. Composition of fill encountered in the borings varied but generally consisted of medium dense silty or clayey sand and firm to stiff clay and silt. A layer of very loose sand was encountered in boring 17B-R10. Layers of very hard clay were recorded in borings 17B-R7 and 17B-R8, the high penetration values are likely due to debris or gravel in the fill material.

4.1.2 Quaternary Sediments (Q)

Quaternary silt and clay layers were encountered beneath fill soil from boring 16B-P33 to boring 17B-R8. The sediments consist of fine-grained Holocene silt and clay and coarse-grained Pleistocene sand and gravel with silt layers. Thicknesses of quaternary deposits encountered in the borings roughly match isopachs described in the geology section of this report.

The thickness and depth of sediments decreased in borings to the north. The fine-grained sediments were roughly 30 feet thick at Pier 33 and gradually decreased to less than 5 feet in boring 17B-R7, near I-95 Pier 41. Penetration resistances in the fine-grained sediments ranged from extremes of 2 to over 50 bpf but were typically in the range of 10 to 20 bpf.

A layer of coarse-grained sediments was encountered in borings 17B-R2 through 17B-R5. The material was mostly classified as loose to dense silty or clayey sand with penetration resistances ranging from 5 to 34 bpf.

4.1.3 Residual Soil and Decomposed Rock (RS and DR)

Residual soil was encountered in each current boring performed and each historic boring. Composition of the residual material was described as gray, green or brown silt, silty sand or sandy clay with visible layered structure. Penetration resistances recorded during current borings in the residual material generally ranged from 20 to 40 blows per foot with lenses of decomposed rock.

Decomposed rock is residual material with a penetration resistance greater than 60 bpf. Highly weathered, decomposed rock was encountered in multiple borings immediately above rock or auger refusal. The decomposed rock was sampled as brown clayey sand or sandy clay with rock fragments.

4.1.4 Rock (R)

Rock coring was performed in six total borings, or four borings at new pier locations and two borings near the planned bridge 1-750 abutment locations. Rock cores were described as gray gneiss with varying degrees of weathering. One 5-foot length of rock core in boring 17B-R2 encountered decomposed rock sampled as sandy clay with relict gneiss structure beneath a competent rock core sample. Fracture orientation and the condition of fracture surfaces varied between and within test holes. Rock core recovery and rock quality designation (RQD) generally increased to the north of the site. RQD is a measure of rock core continuity quantified by a modified core recovery percentage that includes all rock core pieces greater than 4 inches in length. Boring location, rock core sample elevations, rock core recovery and RQD are listed in Table 4-1. Descriptions of rock cores are included on the boring logs and photos of rock core samples are included in the Appendix.

Table 4-1: Summary of Rock Core Samples.

Boring	Core Elevation Range (Feet)	Rock Core Recovery (%)	Rock Quality Designation, RQD (%)
17B-R2	-7.7 to -12.7	95	49
	-12.7 to -17.7	32	0
17B-R3	-1.0 to -6.0	72	31
	-6.0 to -11.0	72	35
17B-R5	-3.2 to -8.2	77	53
	-8.2 to -13.2	72	38
17B-R6	6.9 to 1.9	93	93
	1.9 to -3.1	100	100
17B-R9	-13.8 to -18.8	100	96
	-18.8 to -23.8	96	96
17B-R10	3.5 to -1.5	100	91
	-1.5 to -6.5	77	77

4.2 Groundwater Conditions

Historic boring logs show the groundwater table was encountered typically between elevation 15 feet and 30 feet near Ramp D. Current borings encountered water at lower elevations than historic borings. This may be due to groundwater drawdown but is more likely slow water influx through fine-grained silt and clay during drilling. Groundwater elevations encountered at the time of drilling are listed in Table 4-2.

Table 4-2: Groundwater Elevations at Time of Drilling.

Boring	Approximate Elevations (Feet)		
	Ground Surface	Groundwater During Drilling	Cave After Completion
16B-P33	32.0	6.4	*
17B-R1	35.0	28.7	20.4
17B-R2	37.8	24.8	*
17B-R3	38.5	25.8	4.5
17B-R4	38.5	**	*
17B-R5	38.8	22.8	*
17B-R6	44.0	29.0	40.3

Boring	Approximate Elevations (Feet)		
	Ground Surface	Groundwater During Drilling	Cave After Completion
17B-R7	40.5	16.5	20.9
17B-R8	32.5	**	29.6
WB-16	31.0	-4.0	9.0
17B-R9	28.2	**	*
17B-R10	26.0	16.0	*
17B-R11	26.9	8.7	22.2
17B-R12	21.0	**	13.6

*Not measured
**No groundwater encountered at time of drilling

5.0 ANALYSES AND FOUNDATION RECOMMENDATIONS

Geotechnical calculations are summarized in the following sections. Calculations are included in Appendix D.

5.1 Geotechnical Design Parameters

Geotechnical design parameters were developed based on empirical relationships for penetration resistances recorded in the borings and soil unit weight and strength (Bowles, 1977). Typical parameters are listed in Table 5-1.

Table 5-1: Summary of Drained Design Parameters

Soil Layer	Total Unit Weight, γ (pcf)	Drained Strength		Undrained Strength	
		Friction Angle, Φ' (Degrees)	Soil Cohesion, c' (psf)	Friction Angle, Φ (Degrees)	Soil Cohesion, c (psf)
Fill (F)	115	30 - 32	0	30 - 32	0
Sediments (Q)	110 – 115	28 – 32	0	0	1000
Residual Soil & Decomposed Rock (RS/DR)	130	33 - 36	0	33 - 36	0
Rock (R)	135	45	0	45	0

5.2 Seismic Site Class

Based on the subsurface conditions encountered and N-values recorded in the current test borings a Seismic Site Class D (Stiff Soil Profile) is recommended for seismic design. Seismic design parameters were generated using the United States Geological Survey (USGS) Design Maps program. Values were modified using site factors for the applicable seismic site class. Seismic design parameters are listed in Table 5-2. Per DelDOT Bridge Design Manual and AASHTO Specifications, the site is classified as a Seismic Performance Zone 1 (S_{D1} < 0.15). Acceleration due to gravity (g) is 32.2 ft/s^2.

Table 5-2: Seismic Design Parameters.

Seismic Site Class	D
Peak Ground Acceleration, PGA	0.066g
Corrected PGA, A_S	0.105g
Short-Period (0.2 s) Spectral Acceleration, S_{DS}	0.210g
Short-Period (1.0 s) Spectral Acceleration, S_{D1}	0.078g

5.3 Corrosion Loss

Laboratory testing of four soil samples indicate soil pH ranging from 4.7 to 7.8 and resistivity ranging from 1490 to 2500 ohm-cm. Results of pH and resistivity testing indicate multiple areas of the site have mild corrosion potential. Generally, soil samples with resistivity less than 2,000 ohm-cm and pH less than 5.5 indicate a corrosive environment (AASHTO, 2014 and FHWA, 2016). The presence of stray currents and contaminated soil may promote deterioration of metallic components. For steel piles driven in non-aggressive fill, a steel section loss of 0.085 inches of steel loss is assumed over the 100-year design life (ArcelorMittal 2008). A preliminary check of pile resistance in axial compression was performed for the corrosion-reduced pile section. The reduced section is adequate to support anticipated compression loads.

5.4 Ramp D MSE Wall

The total MSE embankment is approximately 400 feet long and 35 feet wide. The embankment will be constructed as back-to-back MSE walls with a total MSE wall length of around 900 feet. The exposed height of the MSE walls range from less than 7 feet north of Bridge 1-750 to about 26 feet at the southeast corner of the Bridge 1-754S Abutment 40D.

5.4.1 External Stability (Strength Limit State)

External stability analyses were performed using the MSEW (version 3.0) program by ADAMA Engineering, Inc. MSEW is a comprehensive calculation software that checks MSE wall internal and external stability according to AASHTO design guidelines. The program expresses stability of each facet of the MSE wall by a capacity/demand ratio (CDR). CDRs greater than 1.0 indicate the analyzed aspect of the wall is stable. External stability checked as part of this report includes sliding at the base of the wall, overturning (eccentricity) and bearing resistance.

MSE wall reinforcement length and toe embedment were adjusted during analyses to provide adequate resistance (CDR > 1.0) for external stability. Soil strength parameters for reinforced, retained and foundation soil are listed in Table 5-3. The wall sections analyzed, resulting toe embedment and reinforcing length for each wall section are listed in Table 5-4. Calculated bearing resistances for the Strength and Extreme Event (Seismic) limit states are also listed in Table 5-4. Toe embedment and strap lengths in the table should be shown on drawings as minimum values, bearing resistances should be shown on drawings as limiting resistances. If actual bearing pressures exceed the listed resistances the reinforcing strap lengths should be lengthened and bearing resistance recalculated.

MSEW performs internal stability analyses; however, comprehensive internal wall design was not performed. Typical MSE wall systems include proprietary reinforcement and facing connections. The proprietary wall company must perform a thorough wall design and analyses of internal wall stability before construction.

Table 5-3: Soil Parameters for MSE Wall Analyses.

Soil Material	Total Unit Weight, γ_T (PCF)	Drained Friction Strength, ϕ' (Degrees)	Soil Cohesion, c (PSF)	Nominal Active Pressure Coefficient, k_A
Reinforced Fill (No. 57 Stone)	105	34	0	0.28
Retained Fill	120	30	0	0.33
Foundation Soil	120	32	0	0.31

Table 5-4: MSE Wall Embedment, Strap Length and Bearing Resistance.

MSE Wall Design Section Location	Exposed Wall Height (Feet)	Minimum Wall Toe Embedment (Feet)	Minimum Reinforcing Strap Length (Feet)	Strength Factored Bearing Resistance, φ_b = 0.65 (PSF)	Extreme Event Factored Bearing Resistance, φ_b = 1.0 (PSF)
304+00 RT	7	2	8	3700	5200
306+50 LT	10	3	12	5700	8000
306+50 RT	14	3	15	7100	9800
307+50 LT	17	3	16	7200	9700
308+50 LT	23	3	19	8200	10500
308+60 LT	26	3	23	10500	14000

5.4.2 External Stability: Lateral Squeeze

Lateral squeeze occurs when excessive vertical loads push soft fine-grained materials laterally away from the structure. N-values in fine-grained deposits at the planned Abutment 40D indicated firm to stiff silt and clay. A factor of safety against lateral squeeze of 1.92 was calculated at the Abutment 40D MSE wall near station 308+60. Lateral squeeze calculations are included in Appendix E.

5.4.3 Long-term Global Stability (Drained Condition)

Global stability of MSE wall sections with drained soil parameters was checked using the ReSSA (version 3.0) program by ADAMA Engineering, Inc. ReSSA uses wall parameters developed in the MSEW program and limit equilibrium methods to estimate factors of safety (FOS) against slope failure. Circular rotation and block (wedge) analyses were performed for static and seismic load conditions. A 250 psf strip surcharge load was included in each analysis. A FOS of 1.5 was targeted for the MSE walls bearing in fill material and supporting the ramp roadway and bridge abutments. The 1.5 FOS equates to a 0.65 resistance factor for global stability. Results of the global stability analyses are listed in Table 5-5.

Table 5-5: Results of Preliminary Global Stability Analyses.

Ramp D Station	STATIC FOS		SEISMIC FOS	
	Rotation	Wedge	Rotation	Wedge
304+00 RT	1.83	1.74	1.63	1.59
306+50 LT	1.64	1.77	1.48	1.60
306+50 RT	1.45	1.60	1.31	1.44
307+50 LT	1.52	1.50	1.37	1.37
308+50 LT	1.56	1.52	1.44	1.37
308+60 LT	1.63	1.47	1.50	1.34

5.4.4 Short-term Global Stability (Undrained Condition)

Global stability of the MSE wall at Abutment 40D was analyzed with undrained soil parameters using the ReSSA program described in the previous section. The undrained condition is assumed during short-term conditions when porewater pressure in fine-grained soil layers is elevated due to additional load and slow drainage in the fine-grained layers. Minimum factors of safety of 1.37 and 1.22 were calculated for rotational and wedge failures, respectively. The factors of safety are considered adequate for construction conditions when actual surcharge conditions will be lower than the assumed uniform 250 psf and while site monitoring equipment will be operating.

5.4.5 Embankment Settlement (Service Limit State)

Surface settlements due to the added load of the MSE embankment were estimated using standard elastic methods developed by Schmertmann and Harr. Soil elastic properties were estimated based on correlations with standard penetration (N60) resistances.

The largest amount of elastic settlement is anticipated near Bridge 1-748 Abutment 40D. Maximum settlements at the corner and center of MSE Wall B4 (in front of Abutment 40D) are estimated at 0.6 to 0.35 inches, respectively. A layer of fine-grained silt and clay was encountered in boring 17B-R6 near the location of Abutment 40D. Measured N-values in the soil layer indicated the material is firm to stiff. Consolidation of the soil layer was estimated using moisture limit correlations with consolidation parameters assuming the layer is normally consolidated. Correlations were cross-checked with consolidation data measured during consolidation of direct shear specimens from an undisturbed sample at boring 17B-R7. Additional consolidation settlement may approach 1 inch along the wall face. Consolidation calculations are included in Appendix D. Consolidation data from direct shear testing is included in Appendix E.

Differential settlement along the Abutment 40D wall face is estimated at ¾ inches in 17.5 feet (1/300 in/in). Smaller settlements are anticipated along the MSE wall north of Abutment 40D; however, due to the variability of existing fill material, MSE wall joints should be designed to withstand differential settlement of 1/300 in/in.

Elastic settlement should occur within a short time period after completion of the MSE wall. MSE Wall B4 near Abutment 40D and portions of the abutting MSE Walls B1 and B3 may undergo a period of consolidation settlement. The time for consolidation to occur was estimated at 1 month using t_{90} values measured during consolidation of direct shear test samples extracted from boring

17B-R7. Time of consolidation calculations are included in Appendix E. A settlement waiting period of 1 month is recommended at Abutment 40D. After construction of the MSE wall, the area should be allowed to settle before placing wall copings, abutment concrete and pavement. Termination of the settlement waiting period should be determined after review of the data from the settlement monitoring points installed on the MSE wall around the abutment location.

Settlements may result in downdrag loads on the Abutment 40D piles; downdrag is described in following sections.

An approximate 4-foot zone of loose sand and soft clay was encountered in boring 17B-R10 between elevations 21 and 17 feet, immediately below the planned bottom of MSE wall at Bridge 750 Abutment A. Boring 17B-R10 was performed east of the existing embankment footprint. Material encountered may not be encountered in excavations for the MSE wall. If soft areas discovered during construction, these areas should be removed and replaced with compacted borrow material.

5.4.6 Lateral Pile Loads and Pile Deflection

The abutment wall at Bridge 1-748S Abutment 40D will include lateral soil reinforcement to resist lateral loads from the bridge superstructure as well as earth pressure loading from MSE wall fill. Therefore, no lateral forces are anticipated in the Abutment 40D piles and no lateral pile loads will be resisted by the MSE wall. Lateral loads to be resisted by abutment backwall reinforcement should be clearly listed on bridge drawings.

Bridge 1-750 Abutment A and Abutment B are planned as integral abutments. Based on estimates of expansion and contraction of the bridge superstructure, the maximum anticipated pile-head deflection at Abutments A and B is 0.60 inches. Some additional lateral soil pressure on the MSE wall will result from pile deflection; however, it is not recommended to include lateral loads from piles on the MSE wall in front of integral abutments. Integral abutment piles must be able to deflect laterally to allow for movement of the bridge superstructure, additional MSE wall reinforcement around integral piles stiffens the wall backfill which increases stresses in the piles and on the bridge structure.

Integral abutment piles are sometimes set in casings (“cans”) and backfilled with sand to allow for easier pile deflection. Analyses by others has shown that after a cycle of bridge contraction/expansion, loose sand backfill in casings will densify (Arenas, Filz & Cousins 2013). The densification may result in voids under the abutment or lateral movement of the entire casing during subsequent temperature cycles. Therefore, casings are not recommended for the abutment piles. It is recommended to include a pile-head deflection on the MSE wall drawings but not soil pressure diagrams. MSE wall facings at the integral abutments are recommended with a minimum of 0.5 inch joint spacing and panel areas less than 36 square feet to allow for small panel movement induced by pile displacement.

5.5 Bridge 1-748S Widening: Abutment 40D

The north abutment at the Bridge 1-748S will be constructed as part of the new planned MSE wall. Support for Abutment 40D is planned as multiple steel H-piles driven to decomposed rock or rock. The piles will transfer abutment loads to the underlying rock and prevent settlement of the abutment constructed on the tallest portion of the new MSE embankment. Initial settlement from the added load of the MSE wall will still occur; however, and piles driven to rock may experience some amount of downdrag load.

The amount of downdrag load possible was calculated using the APILE program by Ensoft, Inc. Nominal static skin friction loads in the settling soil layers is estimated at about 52 kips; the factored downdrag load (γ_p = 1.05) is 55 kips. According to specifications listed in the 2017 DelDOT Bridge

Design Manual section 210.7.1.6.2-Downdrag, live loads and downdrag should not be considered simultaneously in any load condition and the bridge design should assume the higher of live load or downdrag. Estimated live loads range from 75 to 110 kips at Abutment 40D; live loads exceed downdrag and downdrag will not be considered on the piles.

5.5.1 Abutment 40D – Pile Resistance (Strength Limit State)

Geotechnical resistance for piles driven to decomposed rock and rock will be primarily from end-bearing. To minimize pile damage, the nominal pile resistance to be measured is limited to the factored structural resistance of the piles. The geotechnical resistance factors for piles driven with dynamic testing (φ_{dyn}) is 0.65 (AASHTO, 2014, Table 10.5.5.2.3-1).

Resistance of piles driven to decomposed rock or rock will be controlled by the pile's structural resistance. Structural resistance of steel piles is controlled by either the lower pile section that is susceptible to damage during driving or the upper section that will experience flexure and axial loads. No lateral loads are anticipated at Abutment 40D so structural resistance will be controlled by possible damage during driving. Structural resistance is calculated as the overall structural strength reduced by a resistance factor for damage during pile driving (φ_{da}) of 0.50.

To verify pile resistance in the field, the first pile driven at each substructure unit should be tested using dynamic pile testing. Factored pile resistances and nominal pile resistance to be measured during installation are listed in Table 5-6. The indicated nominal resistance is based on the use of dynamic pile testing with signal matching on selected test piles. To reduce the likelihood of pile damage, piles should be installed with protective steel pile shoes welded to the tips of each pile. No settlement is anticipated for piles bearing on rock.

Table 5-6: Bridge 1-748S Abutment 40D: Resistance of H12X53 Steel Piles.

Pile Section	Nominal Structural Resistance (Kips)	Reduced Structural Resistance for Pile Damage, φ_{da} = 0.50 (Kips)	Nominal Resistance to be Verified During Driving (Kips)	Factored Pile Resistance, φ_{dr} = 0.65 (Kips)
HP12X53	775	387.5	385	250

5.5.2 Abutment 40D – Pile Installation and Length

Minimum and estimated pile tip elevations for Abutment 40D are listed in Table 5-7. Minimum pile tip elevations are the minimum depth to provide pile fixity in lateral load conditions or to penetrate residual soil material. Estimated pile tip elevations were determined based on the elevations of decomposed rock and rock encountered in the recent soil test borings. Pile tips will likely penetrate the upper surface of decomposed rock. Estimated tip elevations are assumed as five feet into decomposed rock material or the upper surface of intact rock.

Pile drivability analyses were performed using the 2003 GRLWEAP program by GRL Engineers, Inc. The analyses show that a pile hammer energy in the range of 40 to 70 kip-feet can install unplugged H-piles to the estimated pile tip elevations without driving refusal or overstressing the piles. Wave equation analyses are approximate models of pile driving and soil conditions and the hammer energy range is provided as an aid in selecting hammer sizes. The pile driving contractor should select pile driving equipment based on review of soil boring logs, independent analyses and local pile driving experience.

Pile tips will likely penetrate the upper surface of decomposed rock at each substructure unit. To protect pile tips during installation steel pile points or shoes are recommended on all piles. Downdrag loads are lower than anticipated live load at the abutment; therefore, piles may be installed before construction of the abutment MSE walls.

Table 5-7: Bridge 1-748S, Abutment 40D: Estimated and Minimum Pile Tip Elevations.

Substructure Unit	Decomposed Rock Elevation (Feet)	Rock Elevation (Feet)	Minimum Pile Tip Elevation (Feet)	Estimated Pile Tip Elevation (Feet)
Abutment 40D	5.0	3.0	17.0	3.0

5.6 Bridge 1-748S Widening: Pier 34D through Pier 39D

Piers for the Bridge 1-748S widening will be located between the current Ramp B piers and piers for the existing Bridge 1-748S. Residual soil varies from 30 to 35 feet below the existing ground surface at these locations. Relatively weak fine-grained sediments overly the residual material. To limit settlement of piers deep foundation elements are recommended to support the new bridge substructure. To minimize impacts to existing on-ramp traffic, foundations for Pier 36 through Pier 39 will be constructed between the existing ramp bridge and the I-95 viaduct bridge before demolition of the existing on-ramp. Staging for driving steel piles will be difficult and multiple drilled shafts are not required to support the pier loads; therefore, micropiles are recommended for pier foundations. Benefits of micropiles include significant axial load resistance, installation ability in low headroom and small diameter elements that allow for flexibility when working near existing structures.

Micropiles support for new pier foundations are recommended with the dimensions and materials listed in Table 5-8. The micropile casing dimensions and yield stress are recommended because they are common for casing available in the United States. Geotechnical and structural micropile analyses are discussed in the following sections, supporting calculations are included in the appendix.

Table 5-8: Bridge 1-748S, Pier 34D thru 39D: Micropile Materials.

Permanent Casing Size	9 5/8-inch OD
Casing Thickness	0.47 inches
Minimum Yield Strength	75 ksi
Reinforcing Center Bar	#14 (2.25 in^2 area) Grade 75 Threaded Bar
Minimum 28-day Grout Compressive Strength	4000 psi

5.6.1 Axial Geotechnical Resistance (Strength Limit State)

Micropiles at the Wilmington Viaduct Ramp D site are designed assuming a nominal grout-to-ground bond strength of 50 psi (7.2 ksf). Soil test borings performed along the ramp alignment encountered variable surfaces of weathered rock and rock. To eliminate uncertainty of micropile lengths, micropiles are designed as pressure-grouted piles with a mixed soil and rock bond zone.

For the small number of micropiles at the site (total of 42 micropiles) design is recommended using the static resistance factor (φ_{stat}) of 0.55 (AASHTO, 2013, Table 10.5.5.2.5-1) and no preconstruction verification testing. Proof testing throughout micropile installation is recommended. Load testing is described in section 5.6.3. Micropiles have been designed to provide 200 kips factored axial resistance. The factored design resistance and static factor require a nominal axial resistance of 365 kips. The nominal resistance requires a micropile bond length of 20.5 feet. Micropile top, bottom of casing and tip elevations and lengths are listed by pier location in Table 5-9. Bottom of casing elevations in the table include a 1-foot plunge length discussed in the following section.

Table 5-9: Bridge 1-748S, Pier 34D thru 39D: Micropile Elevations.

Substructure Unit	Top of Micropile EL[1] (Feet)	Bottom of Casing EL (Feet)	Micropile Tip EL (Feet)	Cased Length (Feet)	Uncased (Bond) Length[2] (Feet)
1-748S Pier 34D	29	-6	-25.5	35	20.5
1-748S Pier 35D	32	-4	-23.5	36	
1-748S Pier 36D	34	2	-17.5	32	
1-748S Pier 37D	34.25	2	-17.5	32.25	
1-748S Pier 38D	34	4	-15.5	30	
1-748S Pier 39D	32.75	4	-15.5	28.75	

Notes: 1) Elevation assumes 1’ micropile embedment into bottom of footing.
2) Bond length includes 1’ plunge length of casing.

5.6.2 Axial Structural Resistance (Strength Limit State)

Compressive and tensile axial resistance of the 9 5/8-inch OD micropile section is summarized in Table 5-10, numbers in the table include a 0.085 in corrosion loss of the steel casing.

Table 5-10: Structural Resistance of Micropile.

Portion of Micropile	Compression Resistance (kips)		Tension Resistance (kips)	
	Nominal	Factored (φ=75)	Nominal	Factored (φ=80)
Cased Length	1011	758	996	796
Uncased Length	347	260	169	135

The design factored axial resistance of the micropiles is 200 kips. No tension loading is anticipated at the micropiles. The maximum proof test load is recommended at 270 kips, 1.35 times the factored design load. The maximum compressive test load is higher than the factored structural compressive resistance. To allow for the 270-kip test load the micropile casing should include a 1-foot plunge length into the bond zone.

5.6.3 Micropile Load Testing

For the small number of micropiles at the site (total of 42 micropiles) design is recommended using the static resistance factor (φ_{stat}) of 0.55 (AASHTO, 2013, Table 10.5.5.2.5-1) and no preconstruction verification testing. Proof testing throughout micropile installation is recommended. One proof test per substructure unit (6 total) are recommended. Proof tests should be performed to 1.35x the factored design load. A 10-minute or 60-minute creep test should be performed at the maximum test load.

5.7 Bridge 1-750: Abutment Foundations

Spread foundations at the Bridge 1-750 abutments are possible; however, preliminary cost estimates indicate the cost savings for a spread foundation instead of a pile foundation is minimal. In addition, a pile foundation will eliminate settlement at the abutment.

Piles installed at Bridge 1-750 should be end-bearing piles driven to decomposed rock or rock. Pile resistance will be controlled by axial compressive resistance reduced for pile damage during installation. Factored pile resistances and nominal driving resistances are listed in Table 5-11. A relatively small amount of settlement of the MSE embankment surrounding the piles is anticipated under the weight of the new MSE wall; however, only a small portion of the Bridge 1-750 piles will be in fill material. No significant downdrag loads are anticipated at the Bridge 1-750 abutments.

Bridge 1-750 is planned with integral abutments. Integral abutments are constructed without bearings which requires the embedded top of piles to deflect during expansion and contraction of the bridge superstructure. The amount of deflection is a function of anticipated temperature differences and bridge span length. Piles supporting integral abutments at Bridge 1-750 should be oriented with pile webs perpendicular to the centerline of the bridge span to allow for easier deflection (weak axis bending). Shear and moments in piles resulting from forced-deflection of the pile-head were calculated using the LPILE program and are listed in Table 5-12. The LPILE analysis assumes a free-head condition at the top of the pile. The free-head condition is adequate for abutments that include a hinge to allow for pile-head rotation.

Table 5-11: Bridge 1-750 Abutments: Resistance of H12X53 Steel Piles.

Pile Section	Nominal Structural Resistance (Kips)	Reduced Structural Resistance for Pile Damage, $\varphi_{da} = 0.50$ (Kips)	Nominal Resistance to be Verified During Driving (Kips)	Factored Pile Resistance, $\varphi_{dr} = 0.65$ (Kips)
HP12X53	775	387.5	385	250

Table 5-12: Calculated Shear and Moment at Bridge 1-750 Integral Abutments

Substructure Unit	Forced Pile-head Deflection (Inches)	Calculated Shear at Pile-head (Kips/Pile)	Calculated Maximum Moment (Kip-Feet)
Bridge 1-750 Abutment A and B	0.60	10.3	39.2 Kip-Feet @ 5' Below Pile-head

5.7.1 Bridge 1-750 Abutments: Pile Tip Elevations and Pile Installation

Minimum and estimated pile tip elevations for Bridge 1-750 abutments are listed in Table 5-13. Minimum pile tip elevations are the minimum depth to provide pile fixity in lateral load conditions or to penetrate residual soil material. Estimated pile tip elevations were determined based on the elevations of decomposed rock and rock encountered in the recent soil test borings. Pile tips will likely penetrate the upper surface of decomposed rock. Estimated tip elevations are assumed as five feet into decomposed rock material or the upper surface of intact rock.

Pile drivability analyses were performed using the 2003 GRLWEAP program by GRL Engineers, Inc. The analyses show that a pile hammer energy in the range of 40 to 70 kip-feet can install unplugged H-piles to the estimated pile tip elevations without driving refusal or overstressing the piles. Wave equation analyses are approximate models of pile driving and soil conditions and the hammer energy range is provided as an aid in selecting hammer sizes. The pile driving contractor should select pile driving equipment based on review of soil boring logs, independent analyses and local pile driving experience. Pile tips will likely penetrate the upper surface of decomposed rock at each substructure unit. To protect pile tips during installation steel pile points or shoes are recommended on all piles.

Table 5-13: Bridge 1-750, Abutments: Pile Tip Elevations.

Substructure Unit	Decomposed Rock Elevation (Feet)	Rock Elevation (Feet)	Minimum Pile Tip Elevation (Feet)	Estimated Pile Tip Elevation (Feet)
1-750 North (Abut. A)	7.0	2.0	10.0	2.0
1-750 South (Abut. B)	5.0	-8.0	15.0	0.0

6.0 SPECIAL CONSIDERATIONS

6.1 Demolition of Existing Bridge 1-758 Foundations

Foundations for the planned 1-748S Pier 34D will overlap with the existing 1-758 6B foundation and the planned 1-748S Pier 35D foundation will overlap with the existing 1-758 5B foundation. Portions of the 1-758 6B and 5B foundations will require complete removal. The location and verticality of existing piles below the foundation should be measured and verified to not intercept the planned micropile locations.

6.2 Additional Load on Existing Bridge 1-748S Foundations

The eastern edge of the planned MSE embankment wall will be constructed approximately 7 feet from the western edge of the existing pile cap at Bridge 1-748S Pier 40S. Some settlement is anticipated beyond the outer edges of the MSE wall. Settlement estimates included in Appendix C indicate the movement is not large enough to induce significant downdrag forces in the existing piles. It is recommended that the existing westernmost columns of Bridge 1-748S be monitored for movement during construction of the Ramp D MSE embankment. Surveys readings of mounted three-dimensional survey targets should be taken on bi-weekly or daily increments depending on the amount of movement observed. Two survey targets separated vertically by at least 10 feet should be attached to the westernmost column of each pier to determine tilt of the column.

6.3 Dewatering and Drainage

The bottom of pile caps and MSE wall base are not anticipated to extend below the existing groundwater surface. However, shallow perched water may be encountered in excavations. Dewatering methods are at the Contractor’s discretion; however, it is anticipated that sumps and pumps will be adequate to maintain a well-drained excavation. The sump should be located at least 3 feet away from the footing outline to avoid softening of the footing areas. Water should be controlled to maintain a minimum 12 inches dry below the bottom of the proposed bottom of excavation.

6.4 Contaminated Soil

Testing for contaminated soil was not performed during the current subsurface investigation. Extensive contaminated soil was not encountered during drilling for the soil test borings; however, there is a potential for contaminated material given the industrial setting of the project site. Testing for contaminates is recommended before performing excavations for the viaduct widening.

6.5 Construction Monitoring

To monitor settlement of MSE walls, monitoring points should be installed on the front face of MSE wall panels above the finished grade at the bottom of the wall. Monitoring points should be fixable survey targets capable of three-dimensional readings and should be installed at every wall corner and at every 100 feet along the wall alignments. Fixable three-dimensional survey targets should also be installed on existing Bridge 1-748 (southbound I-95) Pier 40S through Pier 46S to verify that MSE wall construction is not affecting the existing structure. At a minimum, settlement monitoring points on MSE walls and existing bridge piers should be read twice weekly for the duration of MSE wall construction and weekly for the remainder of ramp construction (abutment concrete and pavement).

Surveys of MSE wall and existing bridge monitoring points will be used to gauge MSE wall settlement, determine the end of the settlement quarantine period at Abutment 40D and to monitor potential impacts to existing piles. A preconstruction survey of nearby structures and vibration monitoring during pile driving will be used to determine impact of pile driving operations on nearby structures.

6.6 Existing Utilities

Two existing terracotta sewer pipes are buried near the project site. The first pipe has a diameter of 12 inches and is buried in line with Chestnut St. The pipe appears to have been re-routed around southbound I-95 Bridge 1-748S Pier 40 with an added stretch diverting south and a second stretch back to the original alignment just east of Pier 40. As currently located, the pipe will under the Ramp D Bridge 1-748S Abutment 40D. This area will be disturbed by pile driving for the abutment and will undergo over 1 inch of settlement, the pipe will be abandoned, and flow rerouted south of the new abutment. The rerouting will have three segments, the longest segment is about 187 feet long and will cross under the existing Bridge 1-758, the I-95 bridges and at-grade portion of southbound on-ramp

Ramp F. Along the alignment, the pipe invert will drop from about elevation 31.5 feet to about elevation 28 feet.

Rule-of-thumb alignment for typical pipe jacking with augers without guidance is 1 foot per 100 feet of length. Guided borings with pilot tubes and optical steering are capable of significantly increased accuracy. Guided borings use survey equipment in the pipe installation pits to sight illuminated targets at the pilot tube bits. Once the pilot tube advances to the exit pit, product pipe or casing follow the pilot tube to open the hole and install the pipeline. Pilot tube borings typically use small diameter jacking and exit pits on the order of 10-feet in diameter or less. The western end of the long pipe run is in a grass area west of the existing ramps; the eastern end is in the existing paved parking lot. These areas should be adequate as installation staging areas.

Historic borings B-207 and B-128 are located near the ends of the sewer pipe realignment. The borings show the tunnel will be installed in a mix of sand, clay and silt. Both the soil material and length of tunnel appears appropriate for guided boring installation of the tunnel. The accuracy of pilot tube borings is high enough to install the new sewer without casing; however, if the Department requires casing, the oversized casing also can be installed using the guided boring method.

The second pipe will cross Ramp D near station 306+30. It has a diameter of 10 inches and an invert elevation of 24.44 feet at the manhole northwest of the existing ramps. At this elevation, the pipe will be more than 10 feet below the bottom of the ramp MSE wall, below the existing ramp embankment. Settlement at this pipe location was calculated at about 0.10 inches. Although small, the conditions of the existing 10-inch terracotta pipe is not known and it is not apparent if the pipe can handle the 0.10-inch deflection.

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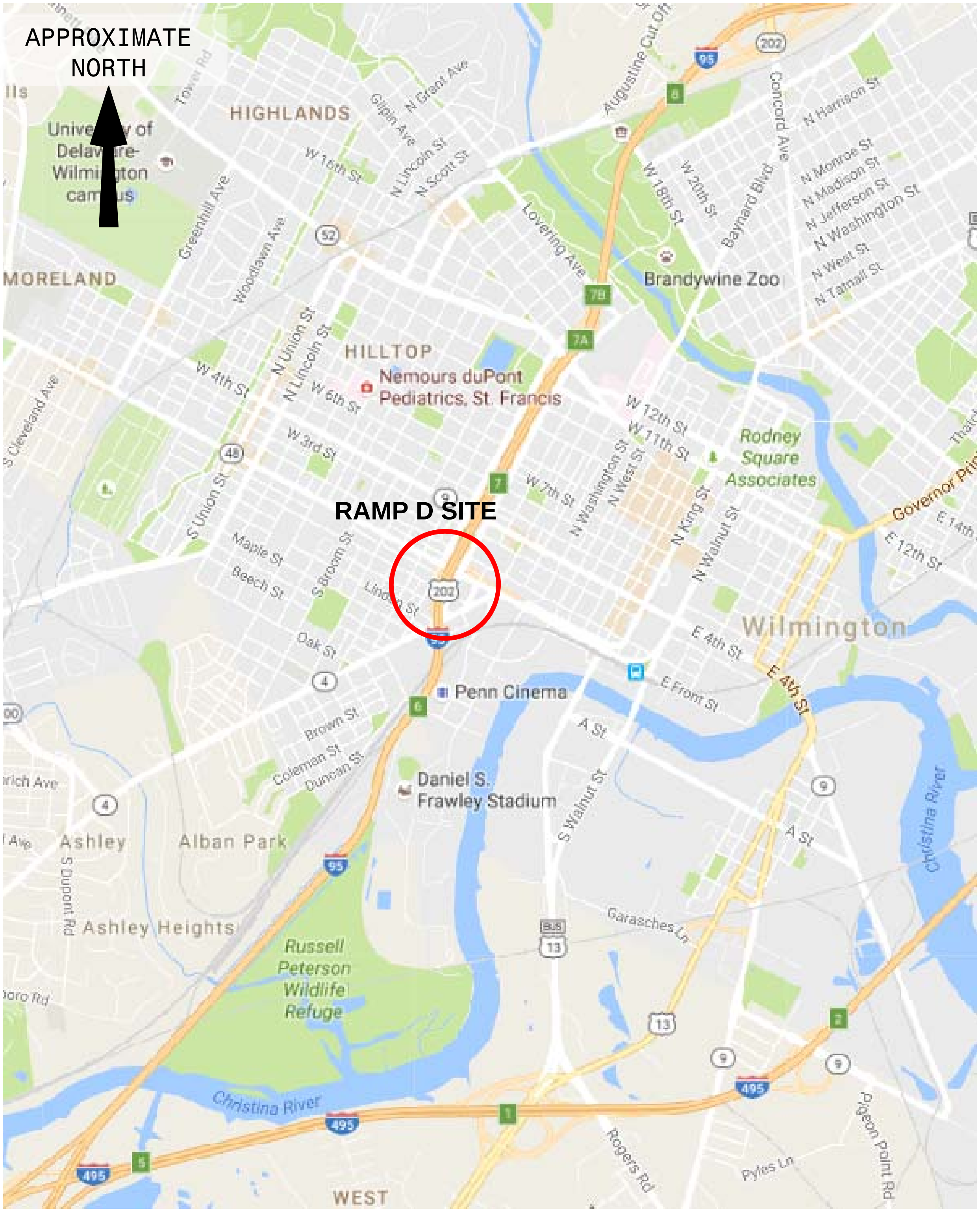
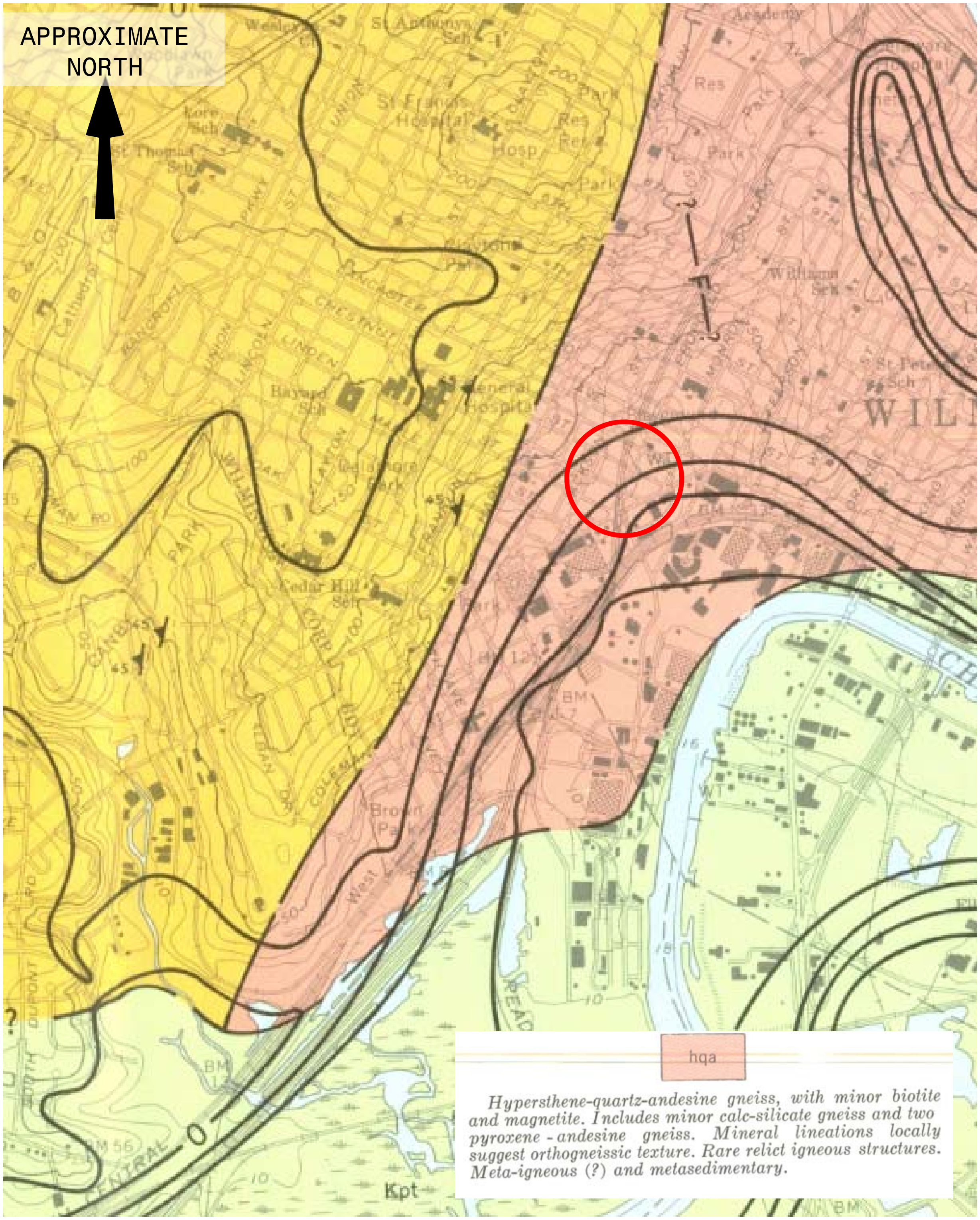
Appendix A



WHITMAN, REQUARDT AND ASSOCIATES, LLP *-Engineers-* 801 S. CAROLINE STREET BALTIMORE, MARYLAND 21231	**SITE LOCATION MAP** **WILMINGTON VIADUCT RAMP D** REFERENCE: ©Google Map Data 2016	DATE November, 2017 SCALE N.T.S.



WHITMAN, REQUARDT AND ASSOCIATES, LLP *-Engineers-* 801 S. CAROLINE STREET BALTIMORE, MARYLAND 21231	**GEOLOGIC SITE MAP** **WILMINGTON VIADUCT RAMP D** REFERENCE: Woodruff, K.D. and Thompson A.M (1975). *Geology of the Wilmington Area, Delaware*	DATE November, 2017 SCALE N.T.S.

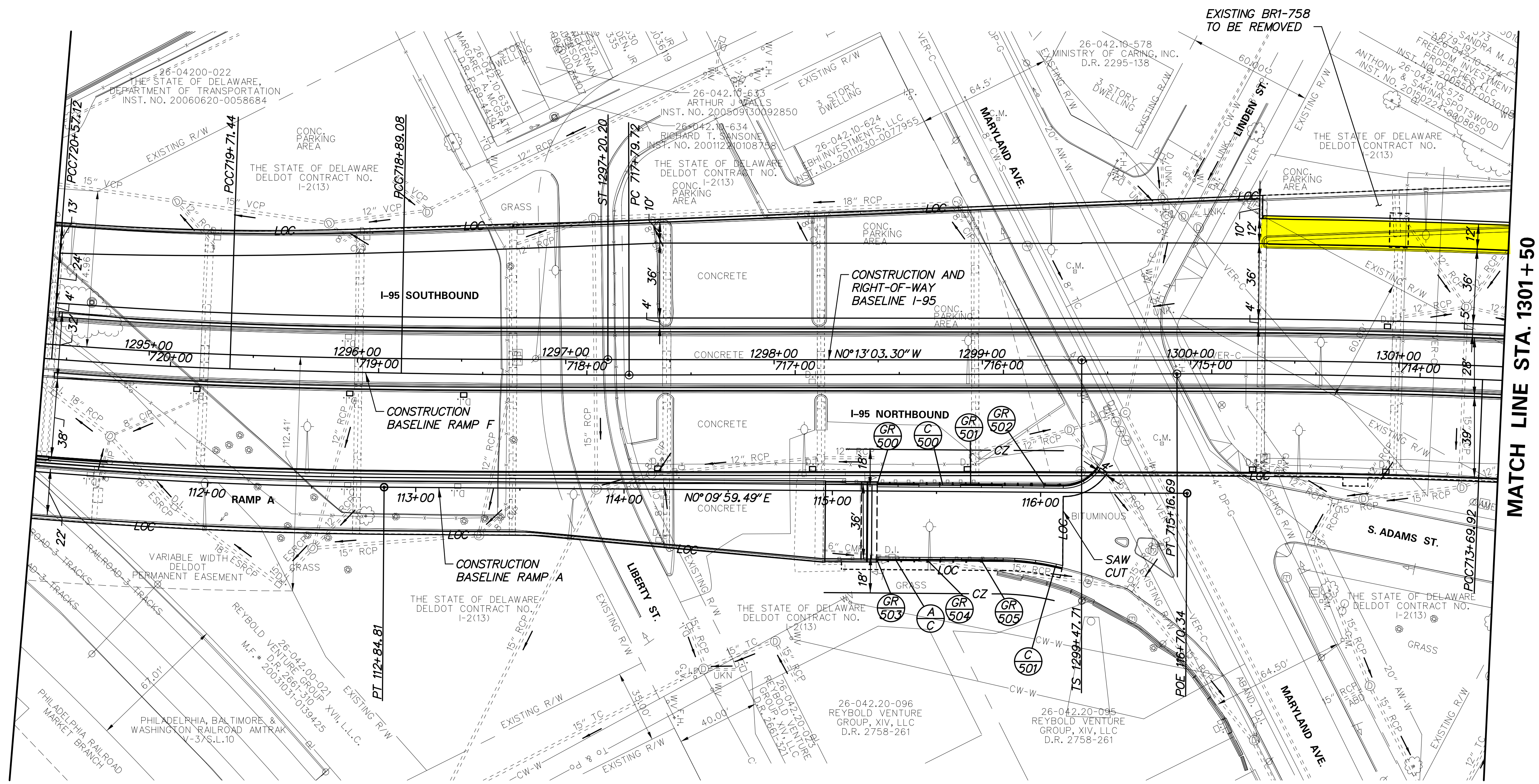
GUARDRAIL SCHEDULE

NO.	ITEM DESCRIPTION / TYPE	BEGIN STA.	OFFSET	LENGTH
500	GUARDRAIL TO BARRIER CONNECTION (EXIT TYPE 31)	XXXX+XX.XX	XXXX.XX	XXXX.XX
501	GALVANIZED STEEL BEAM GUARDRAIL, TYPE 1-31	XXXX+XX.XX	XXXX.XX	XXXX.XX
502	END ANCHORAGE 31	XXXX+XX.XX	XXXX.XX	XXXX.XX
503	GUARDRAIL TO BARRIER CONNECTION (EXIT TYPE 31)	XXXX+XX.XX	XXXX.XX	XXXX.XX
504	GALVANIZED STEEL BEAM GUARDRAIL, TYPE 1-31	XXXX+XX.XX	XXXX.XX	XXXX.XX
505	END ANCHORAGE 31	XXXX+XX.XX	XXXX.XX	XXXX.XX

CURB SCHEDULE

NO.	ITEM DESCRIPTION / TYPE	LENGTH
500	I.P.C.C. CURB AND GUTTER, TYPE 3-4	XXXX.XX
501	I.P.C.C. CURB AND GUTTER, TYPE 3-4	XXXX.XX

Ramp D TS&L Submission Plans 9/1/2017



DELAWARE DEPARTMENT OF TRANSPORTATION	ADDENDUMS / REVISIONS	SCALE 0 30 60 90 FEET	REHABILITATION OF I-95 FROM I-495 TO NORTH OF BRANDYWINE RIVER BRIDGE	CONSTRUCTION PLAN

CONTRACT	T201407404
COUNTY	NEW CASTLE
BRIDGE NO.	1-748, 1-749, 1-758, 1-758G
DESIGNED BY:	JAD
CHECKED BY:	BRT
	CP-18
SHEET NO.	12
TOTAL SHTS.	51

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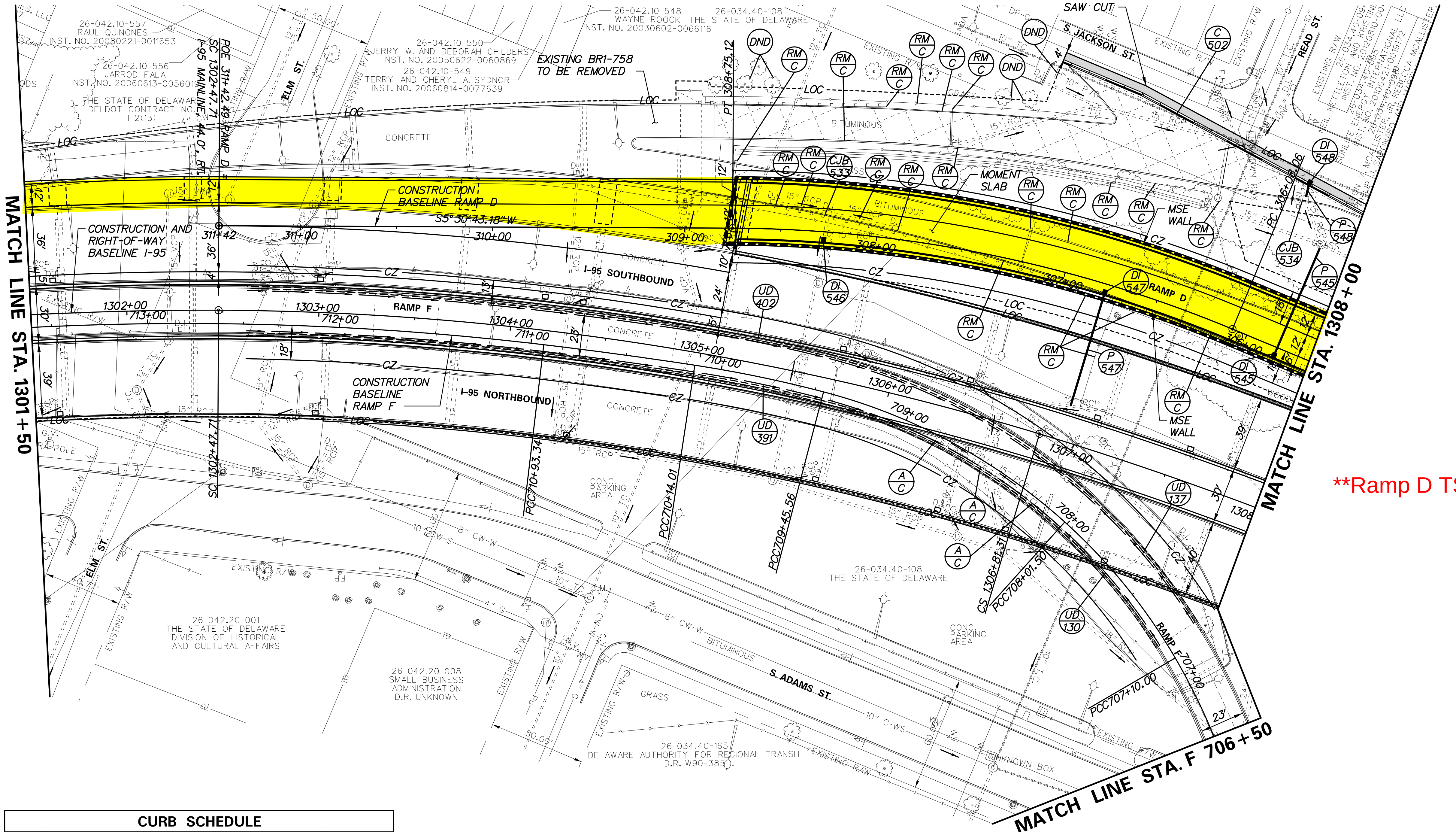

Ramp D TS&L Submission Plans 9/1/2017

CURB SCHEDULE

NO.	ITEM DESCRIPTION / TYPE	LENGTH
502	I.P.C.C. CURB AND GUTTER, TYPE 3-4	XXXX.XX

DRAINAGE INLET SCHEDULE

NO.	STATION	OFFSET	BOX SIZE	GRATE	T.G. EL.	INV. EL.
XXXX	XXXX+XX.XX	XXXX.XX	XX" x XX"	X	XXXX.XX	XXXX.XX

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DELAWARE DEPARTMENT OF TRANSPORTATION

ADDENDUMS / REVISIONS

SCALE 0 30 60 90 FEET

REHABILITATION OF I-95 FROM I-495 TO NORTH OF BRANDYWINE RIVER BRIDGE

CONTRACT	T201407404
COUNTY	NEW CASTLE
BRIDGE NO.	1-748, 1-750, 1-758, 1-758F, 1-758G
DESIGNED BY:	JAD
CHECKED BY:	BRT

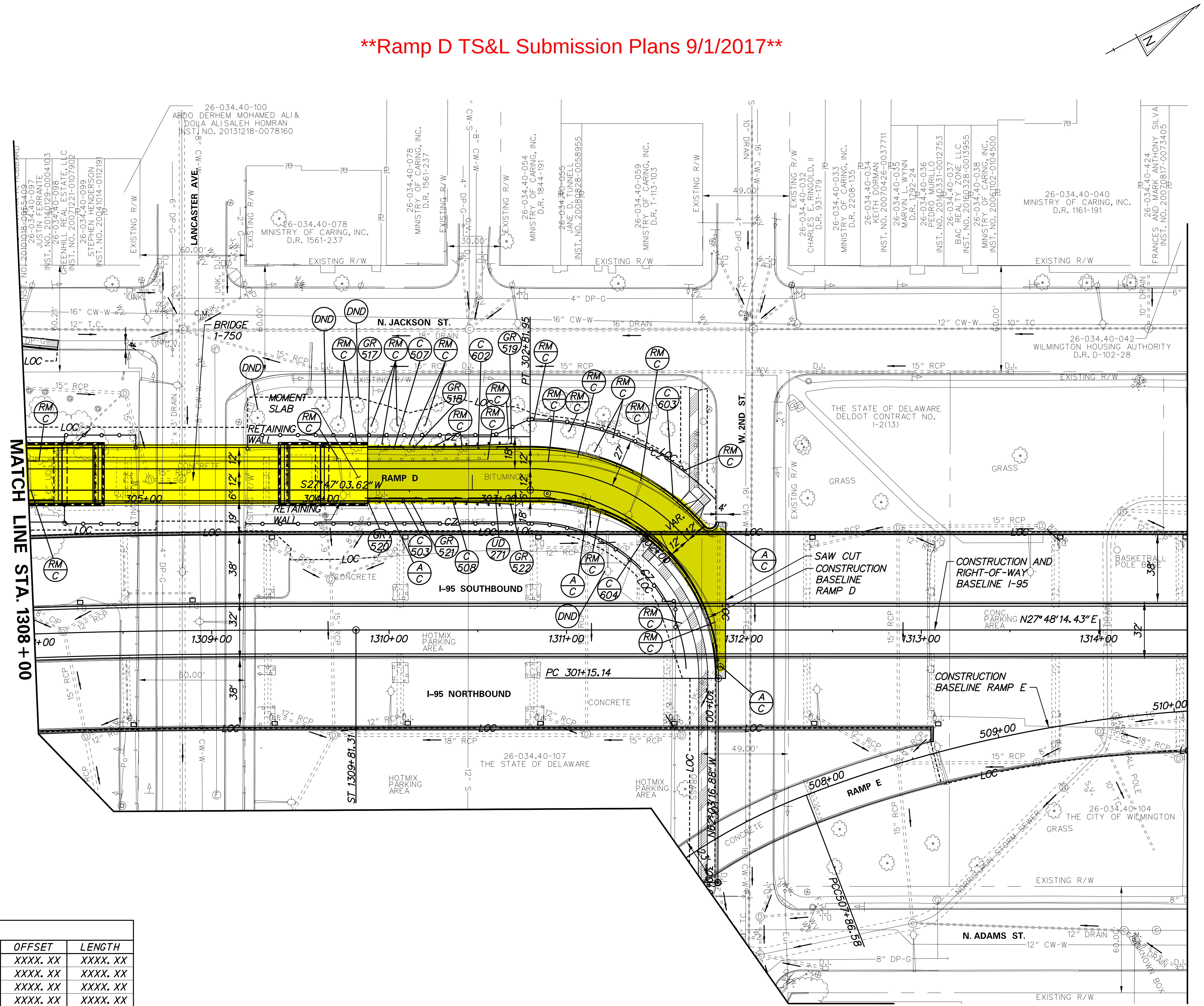
CONSTRUCTION PLAN

CP-19

SHEET NO. 13

TOTAL SHTS. 51

Ramp D TS&L Submission Plans 9/1/2017



CURB SCHEDULE

NO.	ITEM DESCRIPTION / TYPE	LENGTH
507	I.P.C.C. CURB AND GUTTER, TYPE 3-4	XXXX.XX
602	I.P.C.C. CURB AND GUTTER, TYPE 3-2	XXXX.XX
603	I.P.C.C. CURB AND GUTTER, TYPE 3-4	XXXX.XX
503	I.P.C.C. CURB AND GUTTER, TYPE 3-4	XXXX.XX
508	I.P.C.C. CURB AND GUTTER, TYPE 3-2	XXXX.XX
604	I.P.C.C. CURB AND GUTTER, TYPE 3-4	XXXX.XX

GUARDRAIL SCHEDULE

NO.	ITEM DESCRIPTION / TYPE	BEGIN STA.	OFFSET	LENGTH
517	GUARDRAIL TO BARRIER CONNECTION, APPROACH TYPE 1-31	XXXX+XX.XX	XXXX.XX	XXXX.XX
518	GALVANIZED STEEL BEAM GUARDRAIL, TYPE 1-31	XXXX+XX.XX	XXXX.XX	XXXX.XX
519	GUARDRAIL END TREATMENT, TYPE 1-31, TEST LEVEL 3	XXXX+XX.XX	XXXX.XX	XXXX.XX
520	GUARDRAIL TO BARRIER CONNECTION, APPROACH TYPE 1-31	XXXX+XX.XX	XXXX.XX	XXXX.XX
521	GALVANIZED STEEL BEAM GUARDRAIL, TYPE 1-31	XXXX+XX.XX	XXXX.XX	XXXX.XX
522	GUARDRAIL END TREATMENT, TYPE 1-31, TEST LEVEL 3	XXXX+XX.XX	XXXX.XX	XXXX.XX

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DELAWARE DEPARTMENT OF TRANSPORTATION

ADDENDUMS / REVISIONS

SCALE 0 30 60 90 FEET

REHABILITATION OF I-95 FROM I-495 TO NORTH OF BRANDYWINE RIVER BRIDGE

CONTRACT	T201407404
COUNTY	NEW CASTLE
BRIDGE NO.	1-748, 1-750, 1-758E
DESIGNED BY:	JAD
CHECKED BY:	BRT

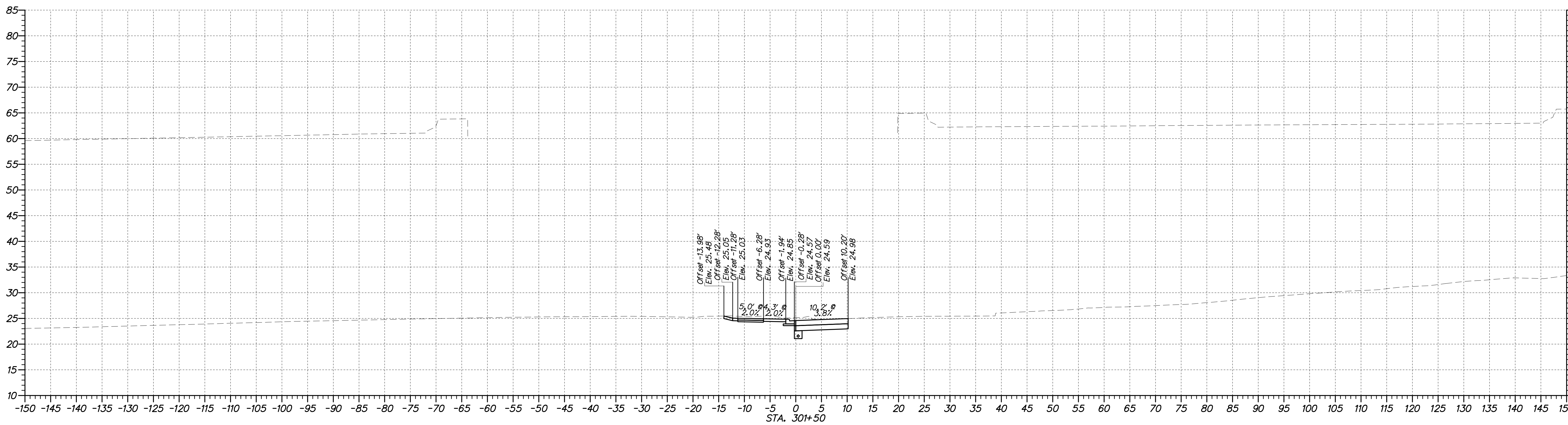
CONSTRUCTION PLAN

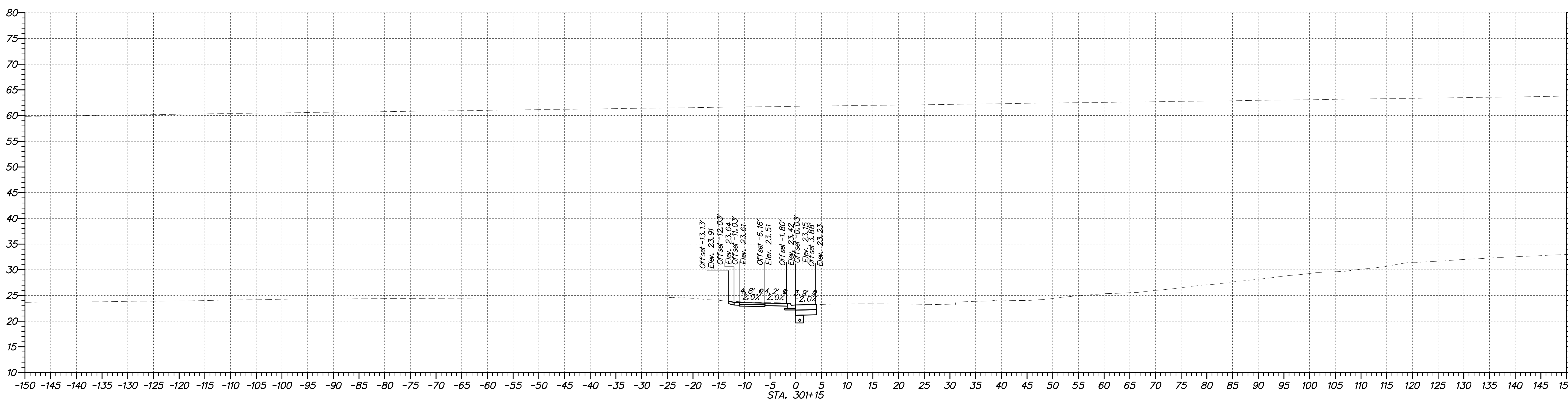
CP-20

SHEET NO. 14

TOTAL SHTS. 51

Ramp D TS&L Submission Sections 9/1/2017





DELAWARE
DEPARTMENT OF TRANSPORTATION

ADDENDUMS / REVISIONS

SCALE 0 10 20 30 FEET

A-6

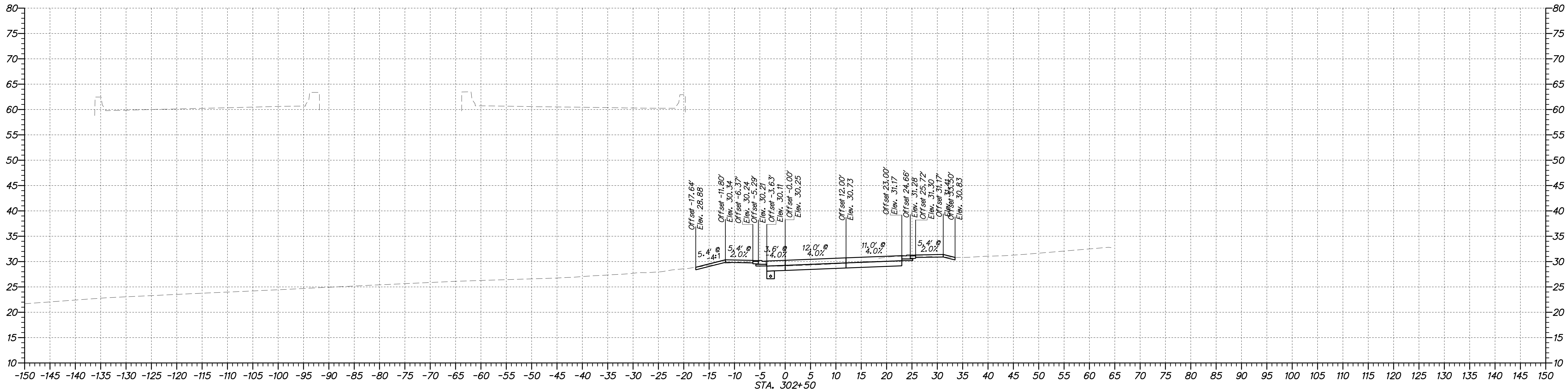
REHABILITATION OF I-95 FROM I-495 TO NORTH OF BRANDYWINE RIVER BRIDGE

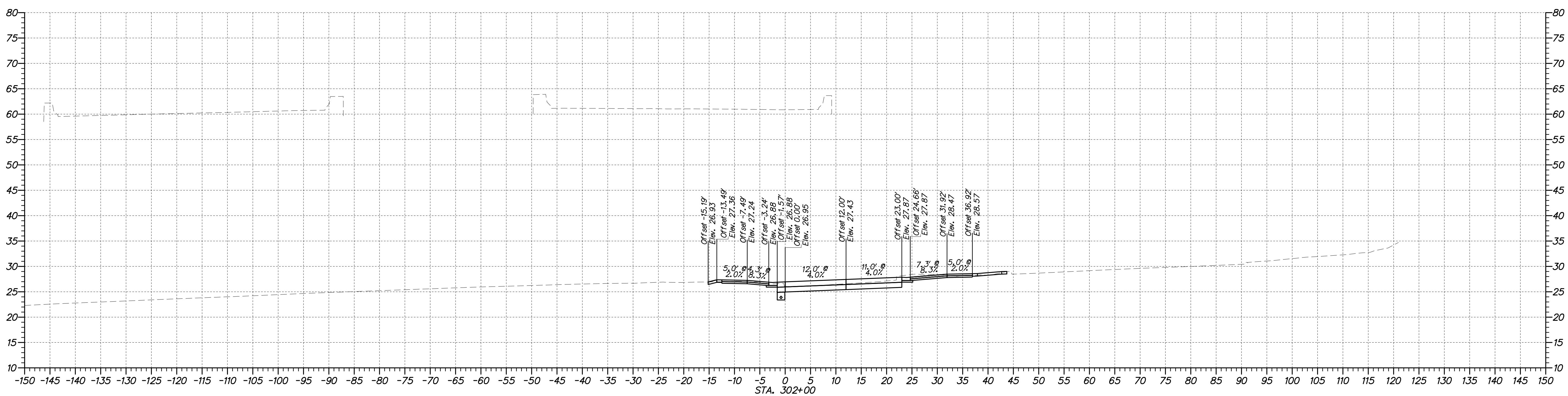
CONTRACT	T201407404
COUNTY	NEW CASTLE
BRIDGE NO.	
DESIGNED BY:	JAD
CHECKED BY:	BRT

CROSS SECTIONS
STA. 301+15 TO STA. 301+50

SHEET NO. 1
TOTAL SHTS. 11

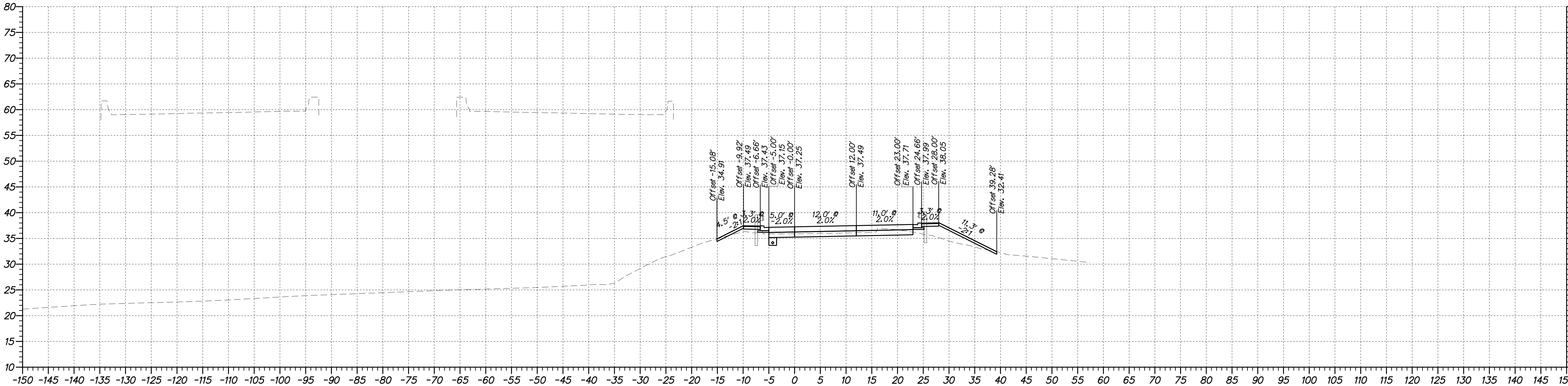
Ramp D TS&L Submission Sections 9/1/2017



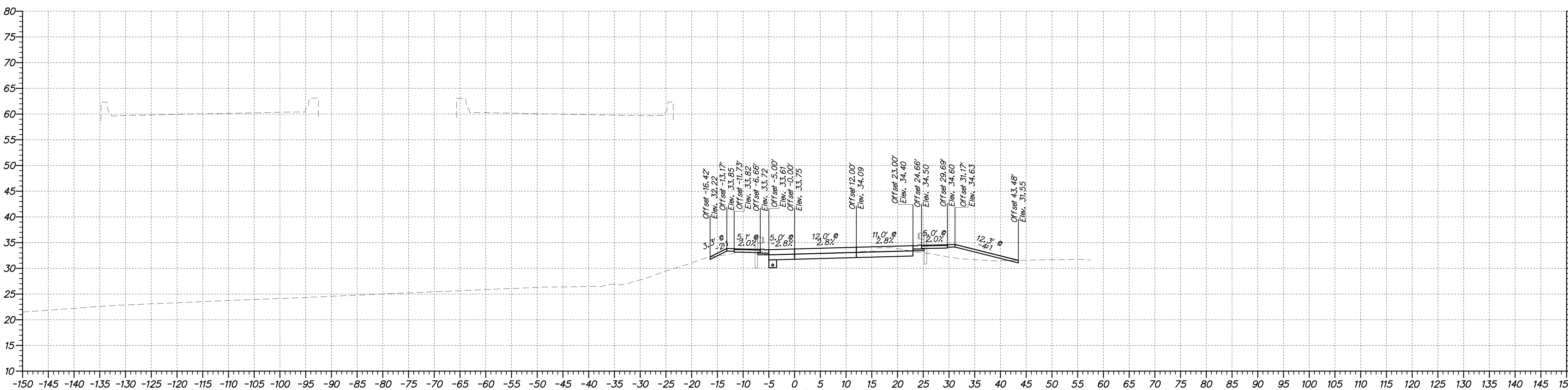


DELAWARE DEPARTMENT OF TRANSPORTATION	ADDENDUMS / REVISIONS	SCALE 0 10 20 30 FEET	REHABILITATION OF I-95 FROM I-495 TO NORTH OF BRANDYWINE RIVER BRIDGE	CONTRACT T201407404 COUNTY NEW CASTLE	BRIDGE NO. DESIGNED BY: JAD CHECKED BY: BRT	CROSS SECTIONS STA. 302+00 TO STA. 302+50	SHEET NO. 2 TOTAL SHTS. 11

Ramp D TS&L Submission Sections 9/1/2017



STA. 303+50

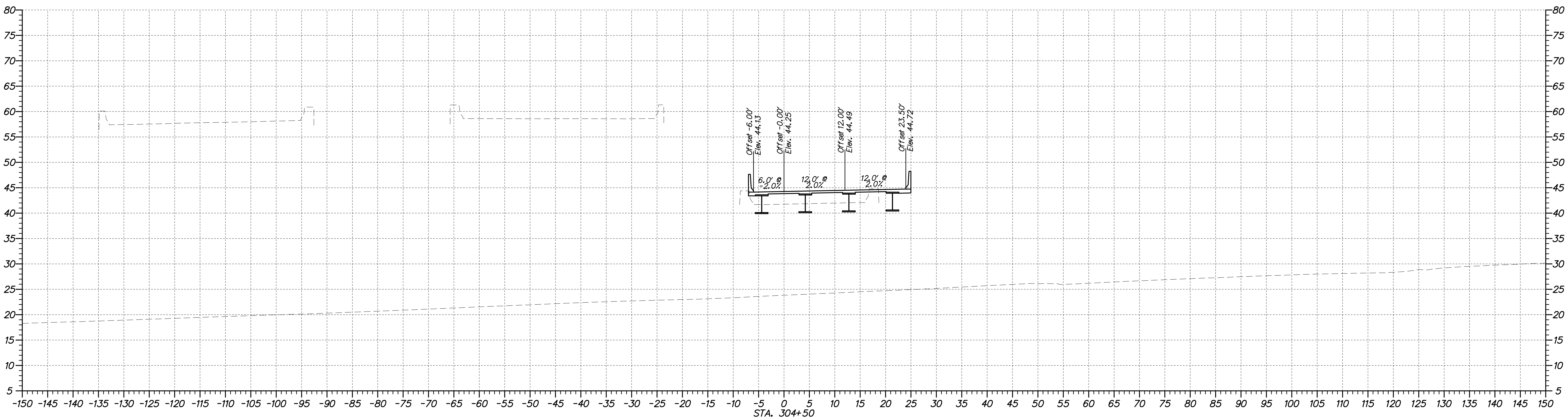


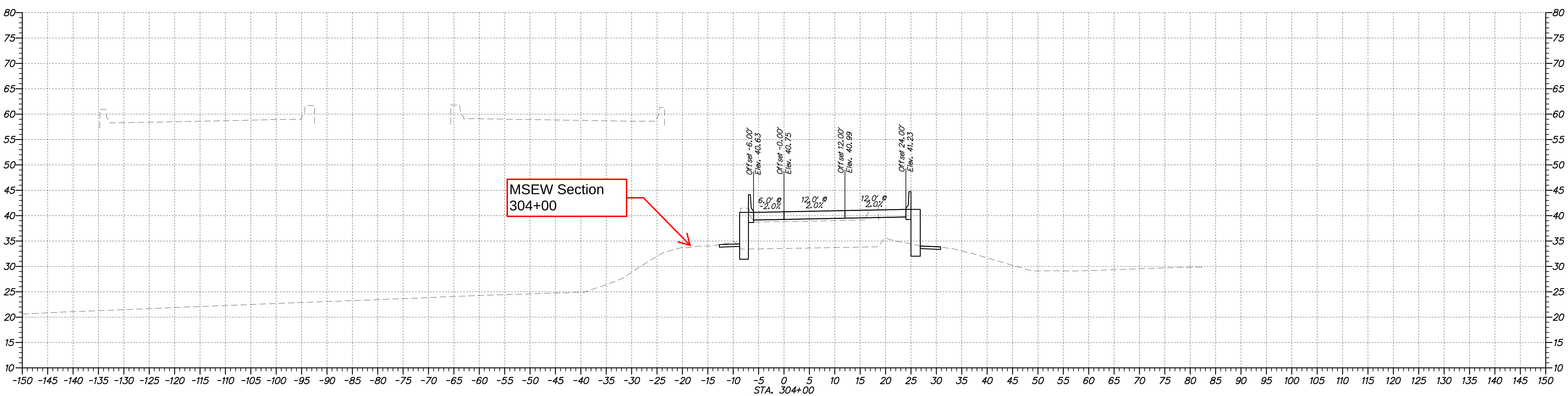
STA. 303+00

DELAWARE DEPARTMENT OF TRANSPORTATION	ADDENDUMS / REVISIONS	SCALE 0 10 20 30 FEET	REHABILITATION OF I-95 FROM I-495 TO NORTH OF BRANDYWINE RIVER BRIDGE	CONTRACT T201407404 COUNTY NEW CASTLE	BRIDGE NO. DESIGNED BY: JAD CHECKED BY: BRT	CROSS SECTIONS STA. 303+00 TO STA. 303+50	SHEET NO. 3 TOTAL SHTS. 11

A-8

Ramp D TS&L Submission Sections 9/1/2017

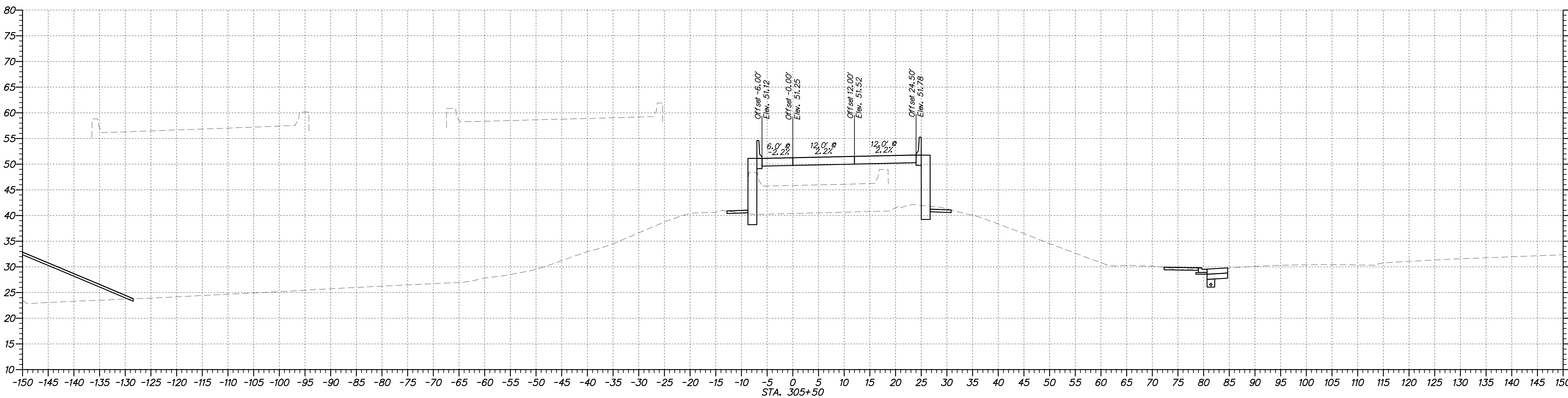


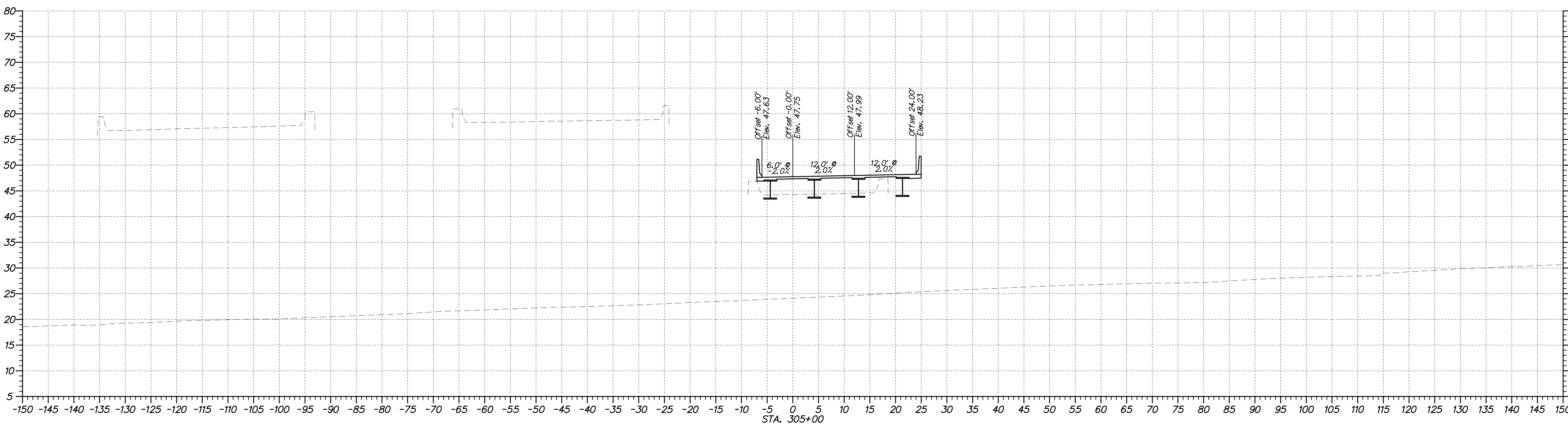


DELAWARE DEPARTMENT OF TRANSPORTATION	ADDENDUMS / REVISIONS	SCALE 0 10 20 30 FEET	REHABILITATION OF I-95 FROM I-495 TO NORTH OF BRANDYWINE RIVER BRIDGE	CONTRACT T201407404 / COUNTY NEW CASTLE	BRIDGE NO. / DESIGNED BY: JAD / CHECKED BY: BRT	CROSS SECTIONS STA. 304+00 TO STA. 304+50	SHEET NO. 4 / TOTAL SHTS. 11

A-9

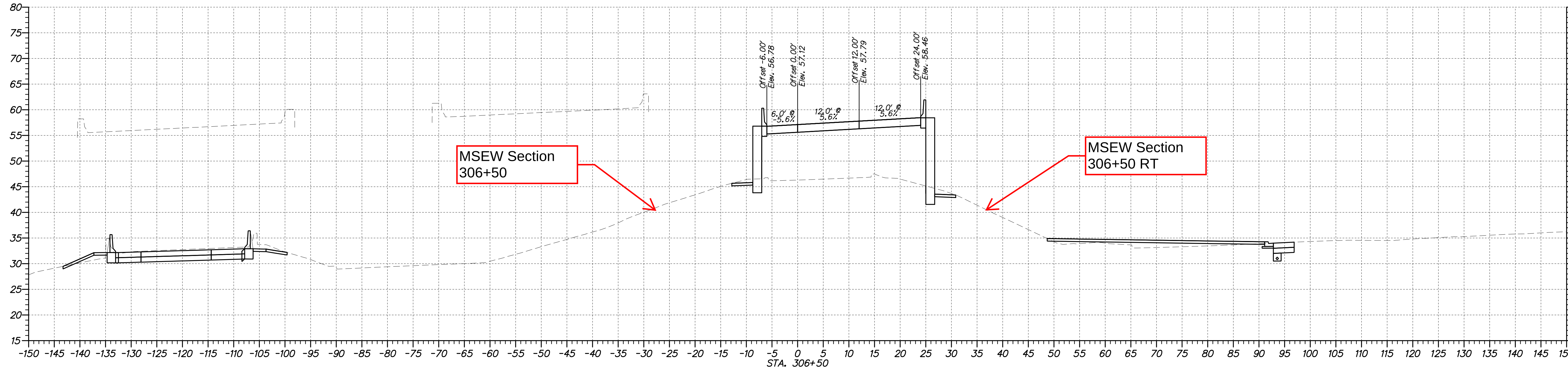
Ramp D TS&L Submission Sections 9/1/2017

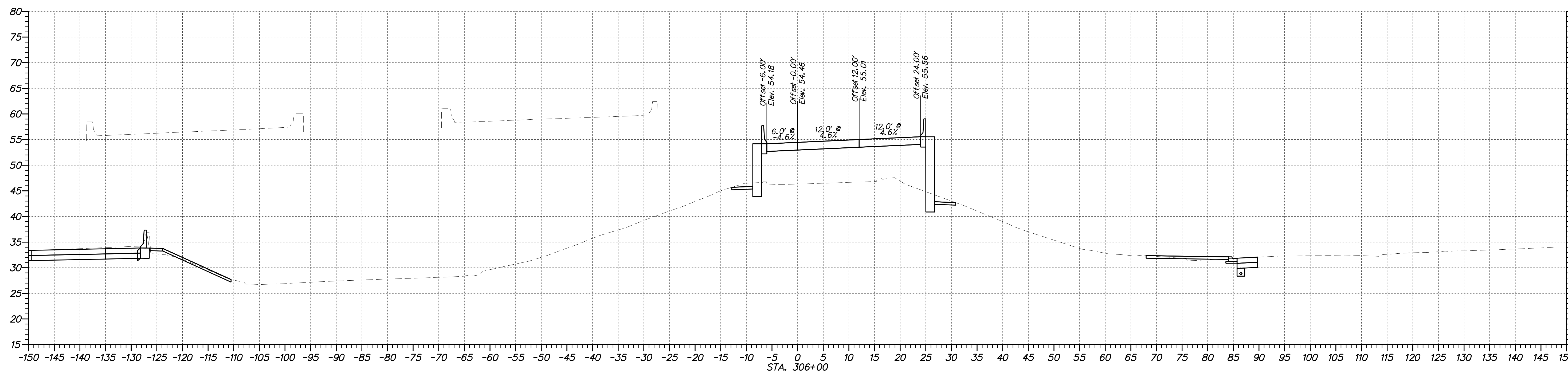




DELAWARE DEPARTMENT OF TRANSPORTATION	ADDENDUMS / REVISIONS	SCALE 0 10 20 30 FEET A-10	REHABILITATION OF I-95 FROM I-495 TO NORTH OF BRANDYWINE RIVER BRIDGE	CONTRACT T201407404 COUNTY NEW CASTLE	BRIDGE NO. DESIGNED BY: JAD CHECKED BY: BRT	CROSS SECTIONS STA. 305+00 TO STA. 305+50	SHEET NO. 5 TOTAL SHTS. 11

Ramp D TS&L Submission Sections 9/1/2017





DELAWARE DEPARTMENT OF TRANSPORTATION

ADDENDUMS / REVISIONS

SCALE 0 10 20 30 FEET

A-11

REHABILITATION OF I-95 FROM I-495 TO NORTH OF BRANDYWINE RIVER BRIDGE

CONTRACT T201407404

COUNTY NEW CASTLE

BRIDGE NO.

DESIGNED BY: JAD

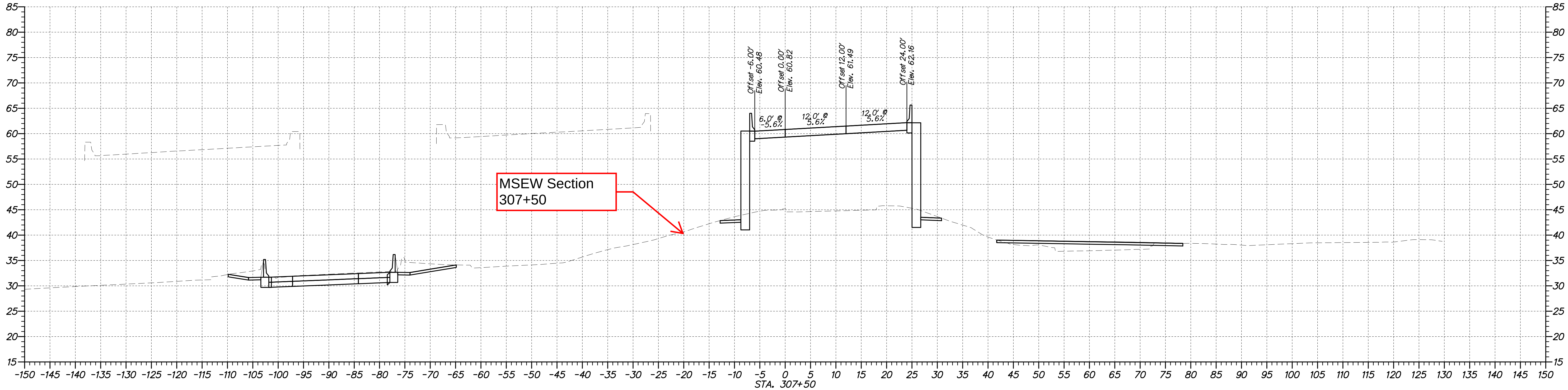
CHECKED BY: BRT

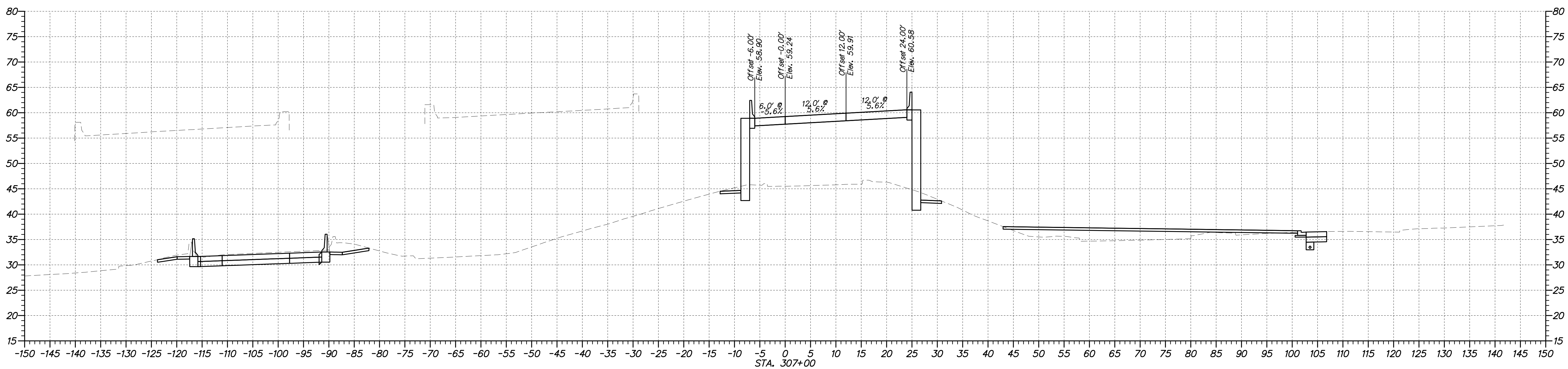
CROSS SECTIONS
STA. 306+00 TO STA. 306+50

SHEET NO. 6

TOTAL SHTS. 11

Ramp D TS&L Submission Sections 9/1/2017





DELAWARE DEPARTMENT OF TRANSPORTATION

ADDENDUMS / REVISIONS

SCALE 0 10 20 30 FEET

A-12

REHABILITATION OF I-95 FROM I-495 TO NORTH OF BRANDYWINE RIVER BRIDGE

CONTRACT T201407404

COUNTY NEW CASTLE

BRIDGE NO.

DESIGNED BY: JAD

CHECKED BY: BRT

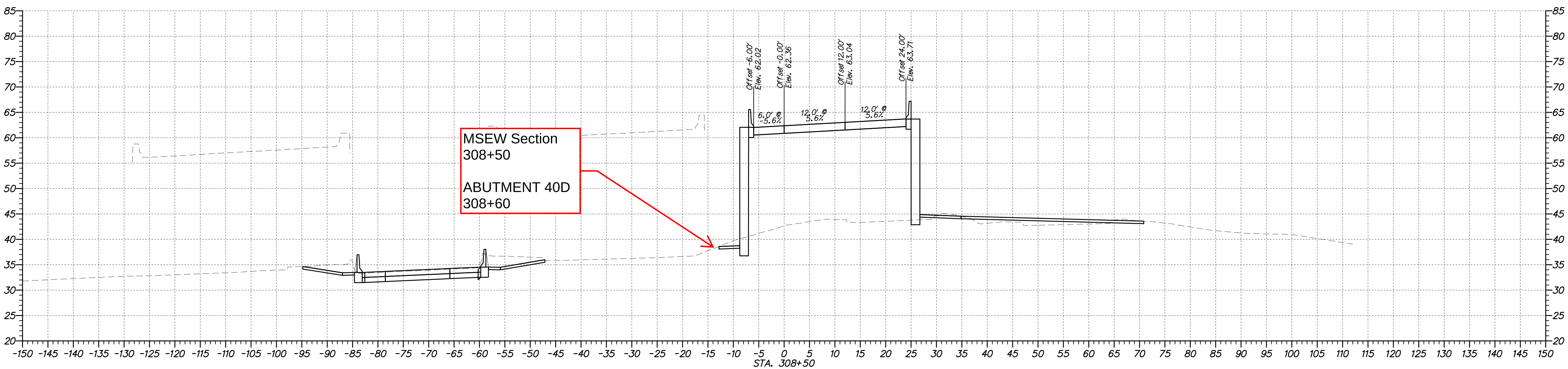
CROSS SECTIONS

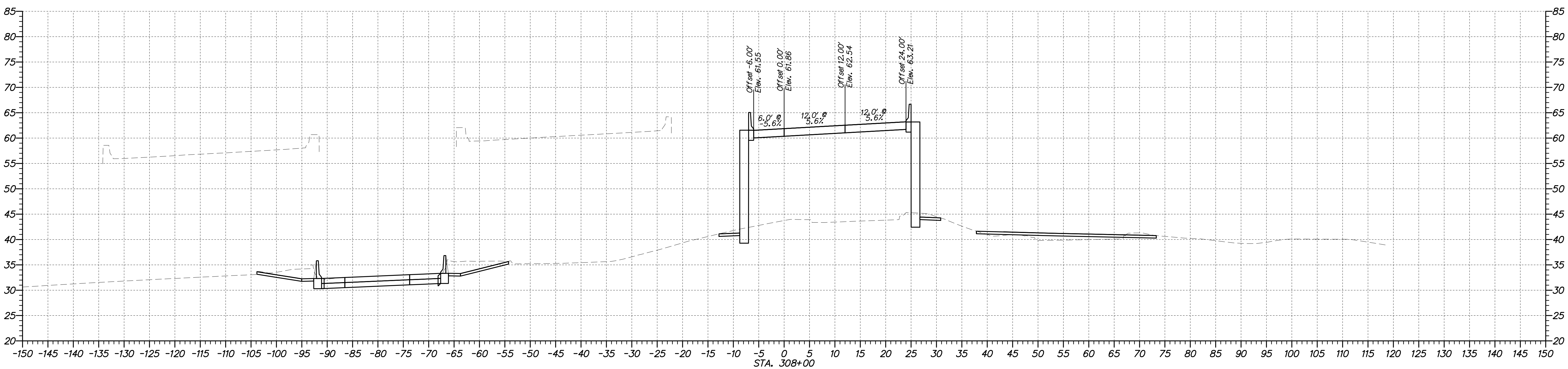
STA. 307+00 TO STA. 307+50

SHEET NO. 7

TOTAL SHTS. 11

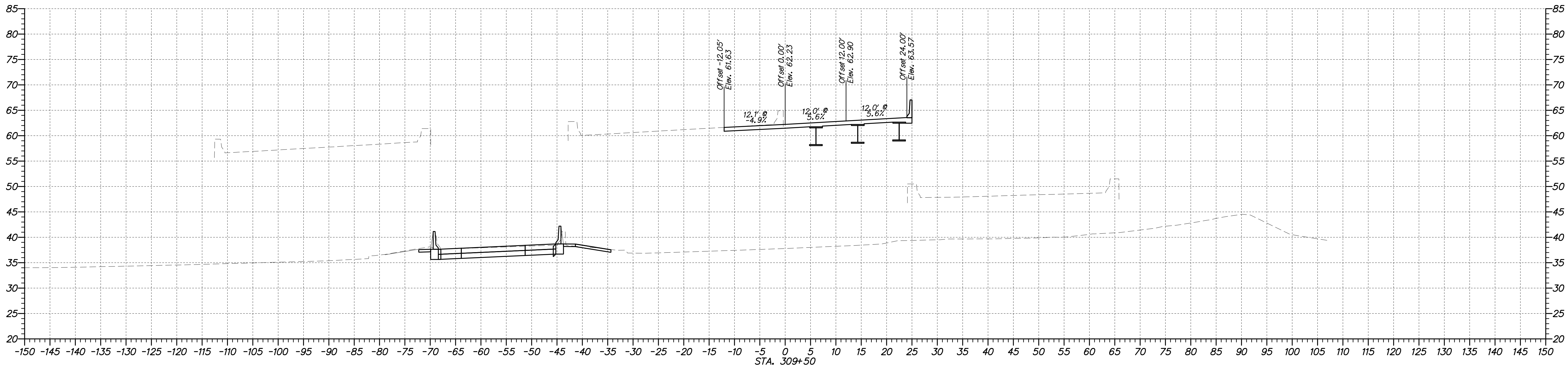
Ramp D TS&L Submission Sections 9/1/2017

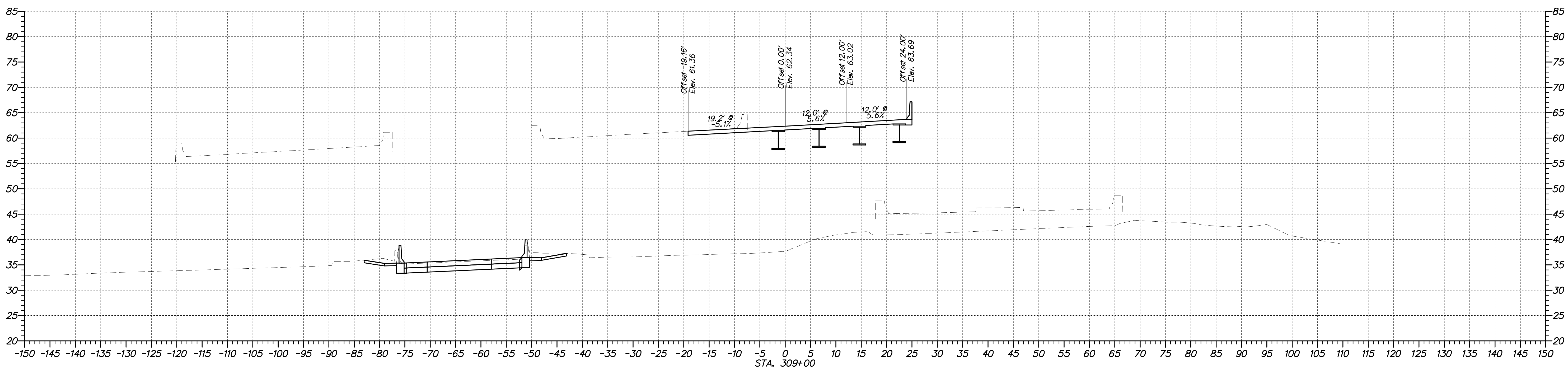




DELAWARE DEPARTMENT OF TRANSPORTATION	ADDENDUMS / REVISIONS	SCALE 0 10 20 30 FEET	REHABILITATION OF I-95 FROM I-495 TO NORTH OF BRANDYWINE RIVER BRIDGE	CONTRACT T201407404	BRIDGE NO.	CROSS SECTIONS	SHEET NO. 8
				COUNTY NEW CASTLE	DESIGNED BY: JAD	STA. 308+00 TO STA. 308+50	TOTAL SHTS. 11
					CHECKED BY: BRT		

Ramp D TS&L Submission Sections 9/1/2017





DELAWARE DEPARTMENT OF TRANSPORTATION

ADDENDUMS / REVISIONS

SCALE 0 10 20 30 FEET

A-14

REHABILITATION OF I-95 FROM I-495 TO NORTH OF BRANDYWINE RIVER BRIDGE

CONTRACT T201407404

COUNTY NEW CASTLE

BRIDGE NO.

DESIGNED BY: JAD

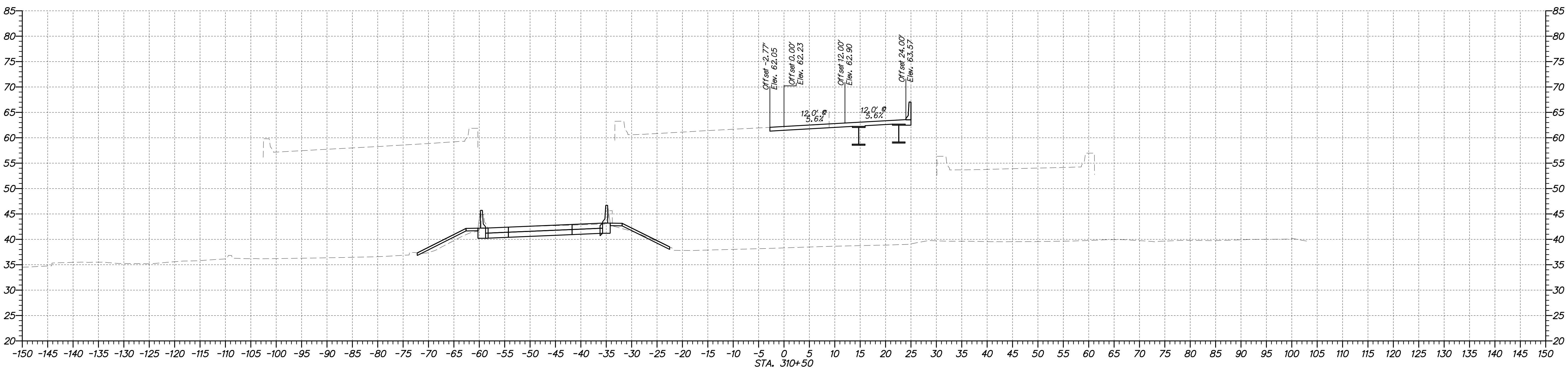
CHECKED BY: BRT

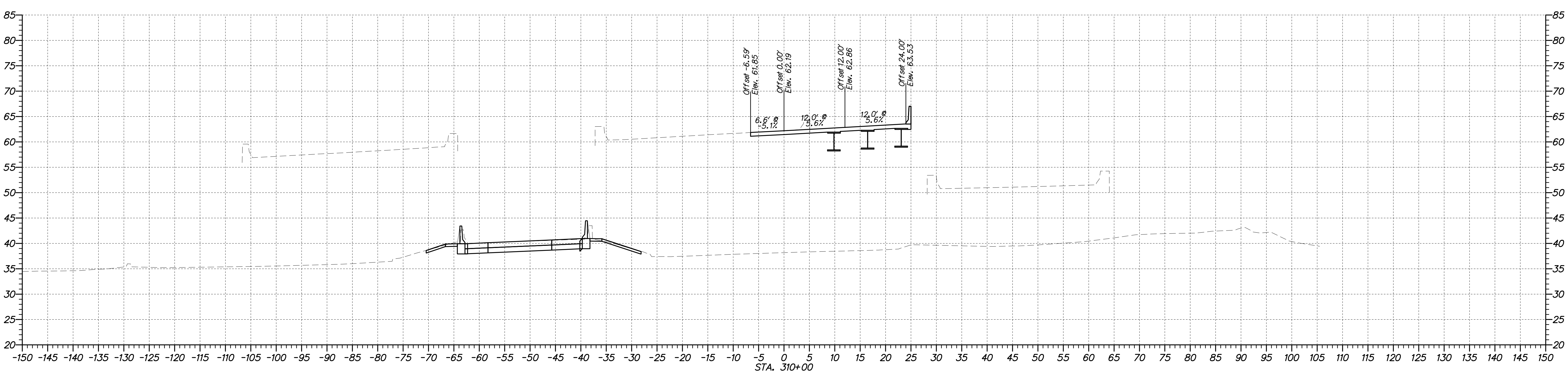
CROSS SECTIONS
STA. 309+00 TO STA. 309+50

SHEET NO. 9

TOTAL SHTS. 11

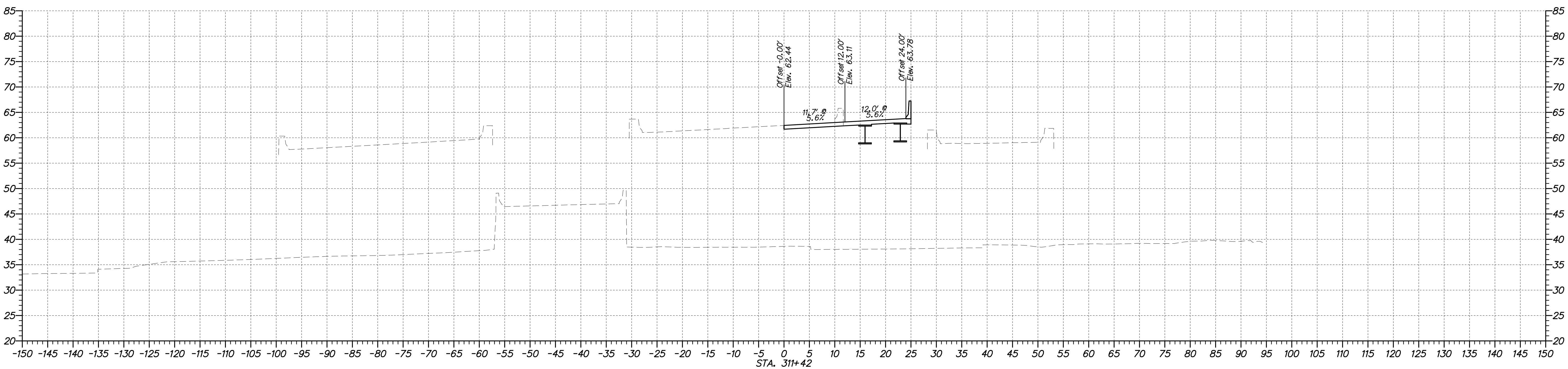
Ramp D TS&L Submission Sections 9/1/2017

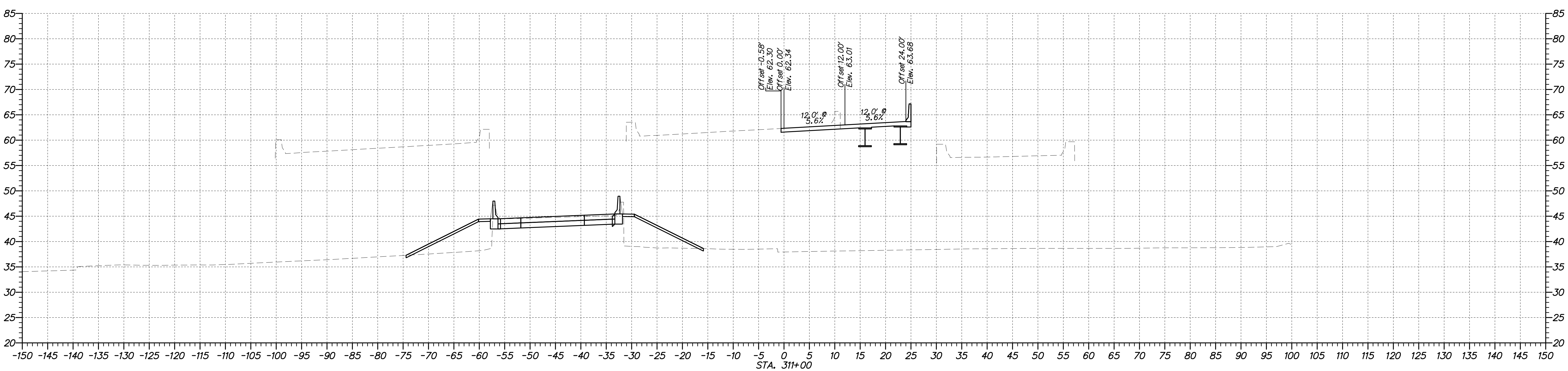




DELAWARE DEPARTMENT OF TRANSPORTATION	ADDENDUMS / REVISIONS	SCALE 0 10 20 30 FEET	REHABILITATION OF I-95 FROM I-495 TO NORTH OF BRANDYWINE RIVER BRIDGE	CONTRACT T201407404	BRIDGE NO.	CROSS SECTIONS	SHEET NO. 10
		A-15		COUNTY NEW CASTLE	DESIGNED BY: JAD	STA. 310+00 TO STA. 310+50	TOTAL SHTS. 11
					CHECKED BY: BRT		

Ramp D TS&L Submission Sections 9/1/2017





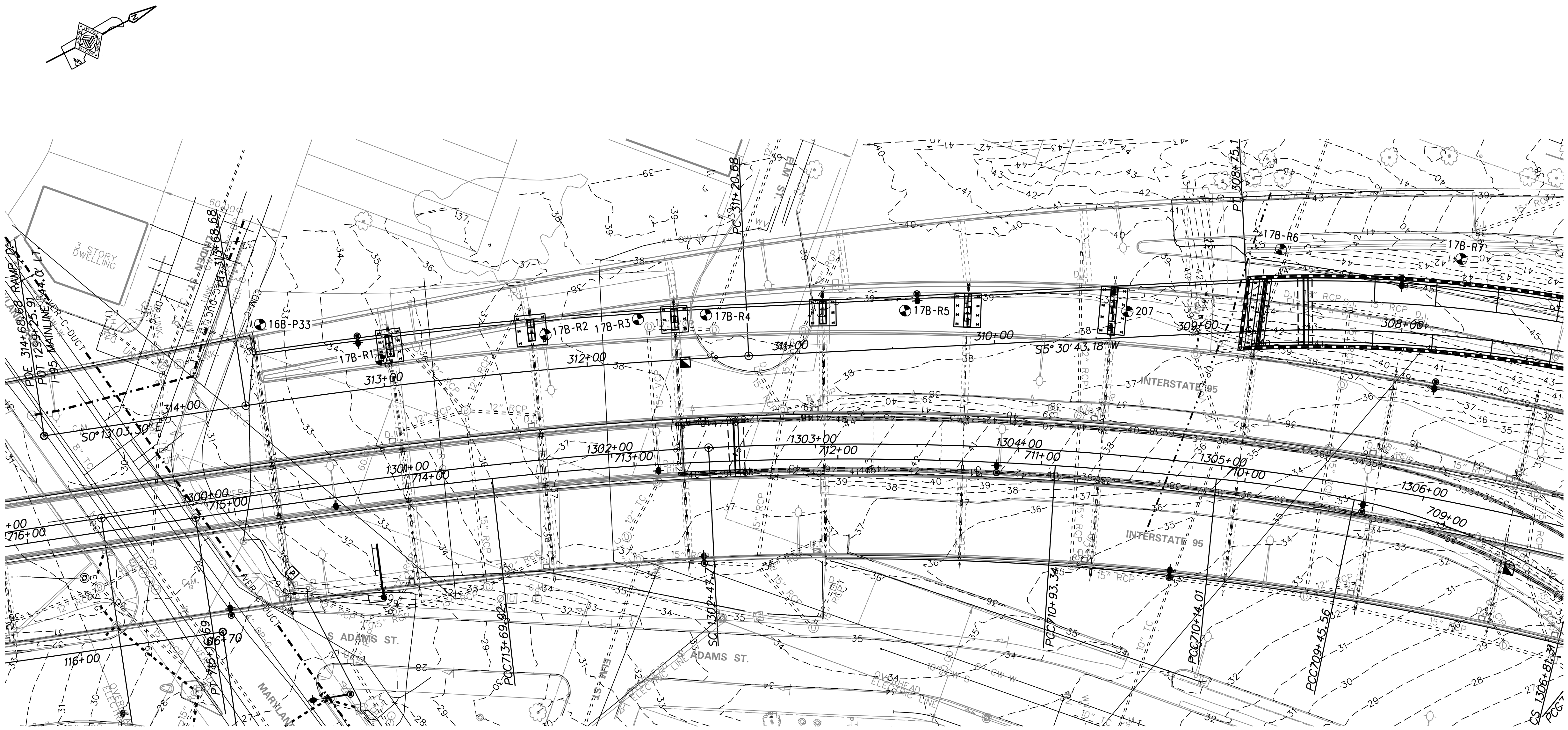
DELAWARE DEPARTMENT OF TRANSPORTATION	ADDENDUMS / REVISIONS	SCALE 0 10 20 30 FEET	REHABILITATION OF I-95 FROM I-495 TO NORTH OF BRANDYWINE RIVER BRIDGE	CONTRACT T201407404 COUNTY NEW CASTLE	BRIDGE NO. DESIGNED BY: JAD CHECKED BY: BRT

A-16

CROSS SECTIONS
STA. 311+00 TO STA. 311+42

SHEET NO. 11
TOTAL SHTS. 11

Appendix B



SOIL BORING REQUEST SUMMARY

NO.	STATION	OFFSET	EXIST. GROUND ELEVATION	DEPTH	COORDINATE	
					NORTH	EAST
B16-P33	1300+38.01	82.65 LT.	EL. 29 +/-	53.5'	633550	615054
B17-R1	1300+93.55	57.60 LT.	EL. 35 +/-	47.5'	633606	615080
B17-R2	1301+73.06	60.89 LT.	EL. 38 +/-	55.5'	633688	615079
B17-R3	1302+17.46	64.82 LT.	EL. 38 +/-	49.5'	633734	615078
B17-R4	1302+49.02	65.10 LT.	EL. 39 +/-	48.2'	633767	615081
B17-R5	1303+43.16	65.53 LT.	EL. 39 +/-	52'	633864	615093
B17-R6	1305+15.00	108.87 LT.	EL. 44 +/-	47.1'	634051	615090
B17-R7	1305+96.97	118.05 LT.	EL. 41 +/-	35.3'	634138	615107

LEGEND

⊕	BORING LOCATIONS

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1-748
B0-02

DELAWARE DEPARTMENT OF TRANSPORTATION

ADDENDUMS / REVISIONS

SCALE 0 30 60 90 FEET

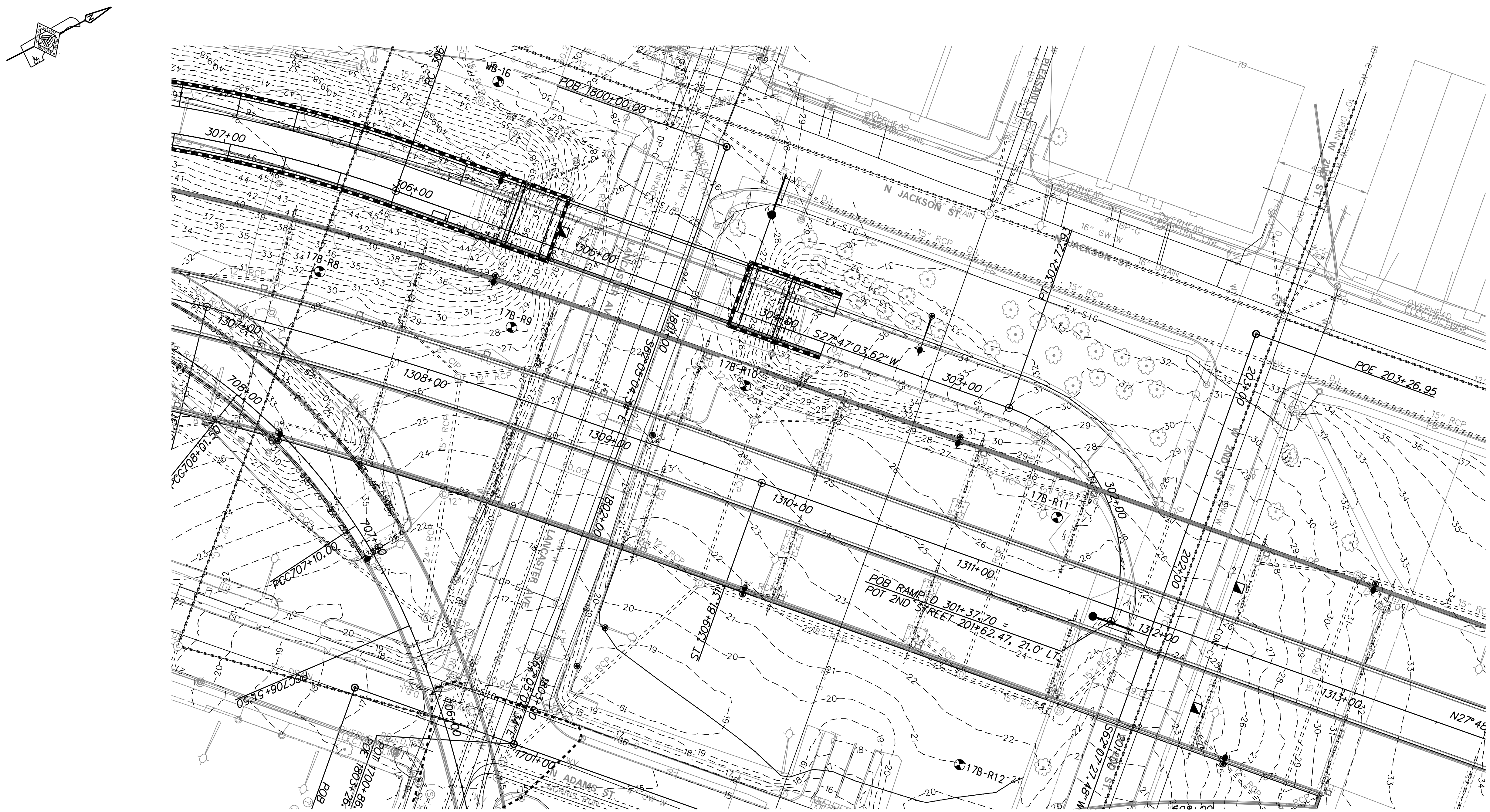
REHABILITATION OF I-95 FROM I-495 TO NORTH OF BRANDYWINE RIVER BRIDGE

CONTRACT	T201407404
COUNTY	NEW CASTLE
BRIDGE NO.	1 748 059
DESIGNED BY:	CFT
CHECKED BY:	

BORING LOCATION PLAN - 1

SHEET NO. 1

TOTAL SHTS. 2



LEGEND
BORING LOCATIONS

SOIL BORING REQUEST SUMMARY

NO.	STATION	OFFSET	EXIST. GROUND ELEVATION	DEPTH	COORDINATE	
					NORTH	EAST
B17-R8	1307+32.57	32.42 LT.	EL. 33 +/-	27'	634239	615239
B17-R9	1308+33.85	33.78 LT.	EL. 28 +/-	52.0'	634333	615282
B17-R10	1309+56.69	44.39 LT.	EL. 26 +/-	32.5'	634447	615329
B17-R11	1311+30.70	34.10 LT.	EL. 27 +/-	31.0'	634596	615419
B17-R12	1311+25.89	102.79 RT.	EL. 21 +/-	28.3'	634528	615538
WB-16	1307+92.37	152.10 RT.	EL. 31 +/-	25.5'	634344	615155

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DELAWARE
DEPARTMENT OF TRANSPORTATION

ADDENDUMS / REVISIONS

SCALE
0 30 60 90
FEET

B-2

REHABILITATION OF I-95 FROM I-495 TO NORTH OF BRANDYWINE RIVER BRIDGE

CONTRACT: T201407404
COUNTY: NEW CASTLE

BRIDGE NO. 1 748 059
DESIGNED BY: CFT
CHECKED BY:

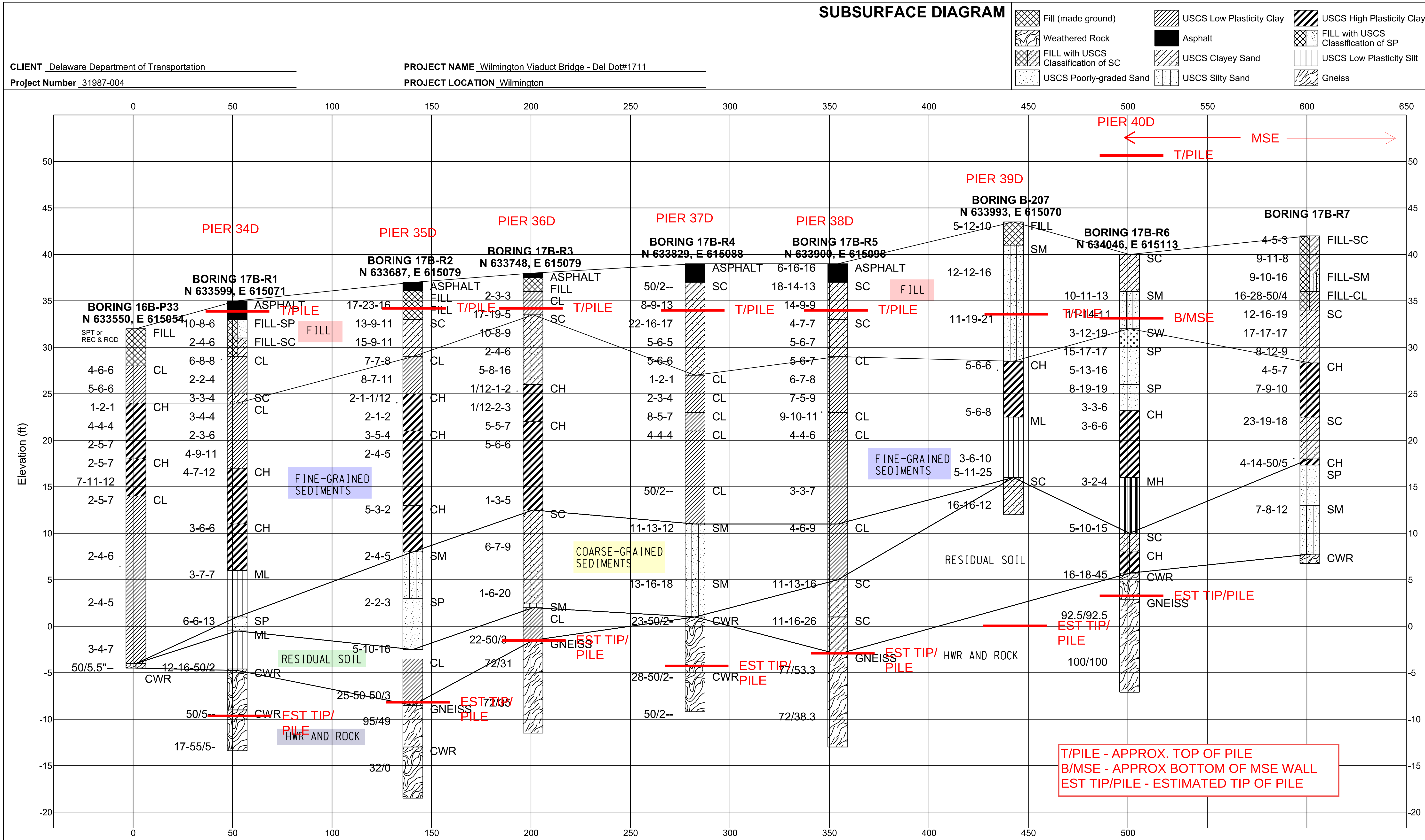
BORING LOCATION PLAN – 2

1-748
B0-02

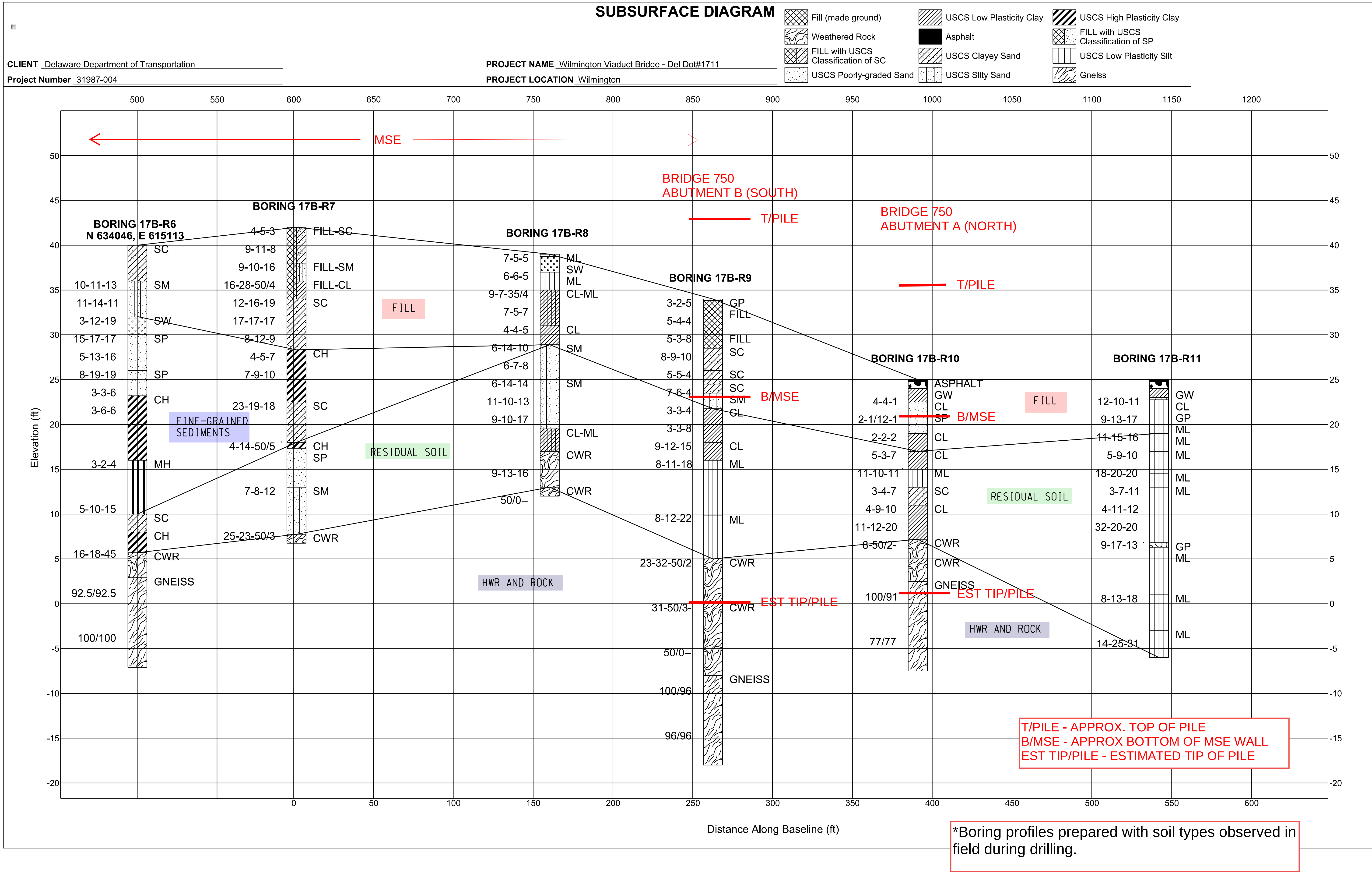
SHEET NO. 2
TOTAL SHTS. 2

SUBSURFACE DIAGRAM

CLIENT Delaware Department of Transportation
Project Number 31987-004
PROJECT NAME Wilmington Viaduct Bridge - Del Dot#1711
PROJECT LOCATION Wilmington



*Boring profiles prepared with soil types observed in field during drilling.


SUBSURFACE DIAGRAM
Fill (made ground)
USCS Low Plasticity Clay
USCS High Plasticity Clay
Weathered Rock
Asphalt
FILL with USCS Classification of SP
FILL with USCS Classification of SC
USCS Clayey Sand
USCS Low Plasticity Silt
USCS Poorly-graded Sand
USCS Silty Sand
Gneiss
CLIENT Delaware Department of Transportation
Project Number 31987-004
PROJECT NAME Wilmington Viaduct Bridge - Del Dot#1711
PROJECT LOCATION Wilmington
MSE
BRIDGE 750
ABUTMENT B (SOUTH)
T/PILE
BRIDGE 750
ABUTMENT A (NORTH)
T/PILE
BORING 17B-R6
N 634046, E 615113
BORING 17B-R7
BORING 17B-R8
BORING 17B-R9
BORING 17B-R10
BORING 17B-R11
FILL
FINE-GRAINED SEDIMENTS
RESIDUAL SOIL
HWR AND ROCK
B/MSE
EST TIP/PILE
FILL
RESIDUAL SOIL
HWR AND ROCK
T/PILE - APPROX. TOP OF PILE
B/MSE - APPROX BOTTOM OF MSE WALL
EST TIP/PILE - ESTIMATED TIP OF PILE
Elevation (ft)
Distance Along Baseline (ft)
*Boring profiles prepared with soil types observed in field during drilling.

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

BORING R-1

Project Name: Wilmington Viaduct
Location: Wilmington, DE
State Contract #: T201407404
Federal Contract #:
Station/Offset:
Northing: 633595.825
Easting: 615077.610
Boring Surface Elev.:
Reference:
Date Started: 7/17/17
Date Completed: 7/17/17

Wt. of Sample Hammer:	140	**Lbs.**	**Average Fall:**	30	**IN.**
Type of: D-Sampler:	Split Barrel	**O.D.**	**O.D. of Sampler:**	2	**IN.**
S-Sampler:		**O.D.**	**O.D. of Samp. Tube:**		**IN.**
U-Sampler:		**O.D.**	**O.D. of Samp. Tube:**		**IN.**
Core Bit:		**O.D.**	**O.D. of Rock Core:**		**IN.**

Hollow Stem Auger Diameter: 3 1/4" **Inches** **From Depth of:** 0.0 **To:** 49.5
Mud Rotary: **From Depth of:** **To:**

Water Level Readings

Date	Depth to Water (ft)	Caved Depth (ft)
7/17/17	27.9	
7/17/17	6.6	14.9

Boring Contractor: Walton Corporation
Equipment/Rig Type: CME 55 Truck
Driller: Dave Burt
Logged By: Randy Ferguson

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
		1	1.0' 2.0'		Moist brown fine gravelly coarse sand w/ some fine sand and silt. 12" RECOVERY	A-1-b	Hot-mix 2" Stone 10"
2.53		2	2.0' 4.0'	10 8 6 6	Moist stiff brown coarse sandy fine gravelly silt w/some fine sand. 12" RECOVERY	A-4(0)	
5.06		3	4.0' 6.0'	2 4 6 8	Wet stiff brown coarse sandy clay w/some fine sand, fine gravel and silt. 7" RECOVERY	A-6(3)	
7.59	▼	4	6.0' 8.0'	6 8 8 9	Wet stiff brown silty clay w/trace fine to coarse sand and fine gravel. 24" RECOVERY	A-6(9)	
10.12		5	8.0' 10.0'	2 2 4 5	Wet firm brown silt w/some clay, trace of coarse to fine sand. 24" RECOVERY	A-4(2)	
		6	10.0' 12.0'	3 3 4 4	Saturated firm brown silty clay w/trace coarse to fine sand. 24" RECOVERY	A-6(14)	
12.65		7	12.0' 14.0'	3 4 4 4	Saturated firm brown clayey silt w/trace coarse to fine sand. 23" RECOVERY	A-4(11)	
		8	14.0'	3 3	Saturated stiff brown silt w/trace fine to	A-4(0)	

Remarks:

Reviewed By: Hany Fekry **Soils Supervisor:** Aaron Wieczorek

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

Project Name: Wilmington Viaduct
Boring No.: R-1
State Contract: T201407404

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
15.18			16.0'	6 8	coarse sand. 24" RECOVERY		
17.71		9	16.0' 18.0'	4 9 11 16	Saturated very stiff gray clayey silt w/trace fine to coarse sand. 20" RECOVERY	A-4(10)	
20.24 22.77		10	18.0' 20.0'	4 7 12 22	Saturated very stiff gray clayey silt w/some coarse to fine sand and fine gravel. 18" RECOVERY	A-4(2)	
25.3 27.83	■	11	24.0' 26.0'	3 6 6 6	Saturated stiff gray silty clay w/some fine sand, trace of coarse sand and fine gravel. 24" RECOVERY	A-6(7)	
30.36 32.89		12	29.0' 31.0'	3 7 7 9	Saturated stiff gray coarse sandy clay w/some fine sand and silt, trace of fine gravel. 24" RECOVERY	A-6(3)	
35.42		13	34.0' 36.0'	6 6 13 42	Saturated medium dense gray silty fine to coarse sand and fine gravel. 9" RECOVERY	A-2-4(0)	

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

Project Name: Wilmington Viaduct
Boring No.: R-1
State Contract: T201407404

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
37.95							
40.48		14	39.0' - 41.0'	12 16 50/2	Saturated very dense gray silty fine sand and fine gravel w/some coarse sand. 6" RECOVERY	A-2-4(0)	
43.01							
45.54		15	44.0' - 46.0'	50/5	No sieve analysis - Indication of Saturated very dense gray silty fine to coarse sand w/ trace fine gravel. 3" RECOVERY		
48.07		16	47.5' - 49.5'	10 50/5	Saturated very stiff gray clayey fine to coarse sand w/some silt, trace of fine gravel. 10" RECOVERY	A-2-7(0)	
50.6					End Boring		
53.13							
55.66							
58.19							

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

BORING R-2

Project Name: Wilmington Viaduct
Location: Wilmington, DE
State Contract #: T201407404
Federal Contract #:
Station/Offset:
Northing: 633687
Easting: 615079
Boring Surface Elev.:
Reference:
Date Started: 7/6/17
Date Completed: 7/6/17

Wt. of Sample Hammer: 140 **Lbs.** **Average Fall:** 30 **IN.**
Type of: **D-Sampler:** Split Barrel **O.D.** **O.D. of Sampler:** 2 **IN.**
S-Sampler: **O.D.** **O.D. of Samp. Tube:** **IN.**
U-Sampler: **O.D.** **O.D. of Samp. Tube:** **IN.**
Core Bit: **O.D.** **O.D. of Rock Core:** **IN.**

Hollow Stem Auger Diameter: 3 1/4" **Inches** **From Depth of:** 0.0 **To:** 45.5
Mud Rotary: **From Depth of:** **To:**

Water Level Readings

Date	Depth to Water (ft)	Caved Depth (ft)
7/6/17	30.0	
7/6/17	16.0	23.0

Boring Contractor: Walton Corporation
Equipment/Rig Type: CME 55 Truck
Driller: Billy Holden
Logged By: Randy Ferguson

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
		1	0.0' 2.0'		No Recovery NR" RECOVERY		Hot-mix 5" Stone 6" Fill 12"
2.53		2	2.0' 4.0'	17 22 16 11	Moist hard brown fine gravelly silt w/some coarse sand, trace of fine sand and clay. 13" RECOVERY	A-4(0)	
5.06		3	4.0' 6.0'	8 13 11 13	Moist very stiff gray clayey silt w/some coarse to fine sand and fine gravel. 15" RECOVERY	A-4(4)	
7.59		4	6.0' 8.0'	15 9 11 19	Moist very stiff brown clayey coarse sandy silt w/some fine sand, trace of fine gravel. 23" RECOVERY	A-4(3)	
10.12		5	8.0' 10.0'	7 7 7 6	Moist stiff gray silt w/some coarse to fine sand and clay, trace of fine gravel. 20" RECOVERY	A-4(1)	
		6	10.0' 12.0'	6 7 11 15	Moist very stiff brown fine sandy silt w/some coarse sand and fine gravel. 12" RECOVERY	A-4(0)	
12.65		7	12.0'	1 1 1 1	pH Sample		

Remarks:

Reviewed By: Hany Fekry
Soils Supervisor: Aaron Wieczorek

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

Project Name: Wilmington Viaduct **Boring No.:** R-2
State Contract: T201407404

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
			14.0'		16" RECOVERY		
15.18		8	14.0' – 16.0'	3 3 4 5	Saturated firm gray silty clay w/trace fine to coarse sand. 14" RECOVERY	A-6(13)	
17.71		9	16.0' – 18.0'	4 5 6 5	Saturated stiff gray clayey silt w/trace fine to coarse sand and fine gravel. 18" RECOVERY	A-4(7)	
20.24, 22.77		10	18.0' – 20.0'	3 3 4 5	Saturated firm gray clayey silt w/trace coarse to fine sand. 18" RECOVERY	A-4(5)	
25.3, 27.83		11	24.0' – 26.0'	5 2 3 3	Saturated firm gray clayey silt w/some coarse to fine sand and fine gravel. 8" RECOVERY	A-4(2)	
30.36, 32.89		12	29.0' – 31.0'	3 13 6 6	Saturated very stiff gray clayey silt w/some coarse to fine sand, trace of fine gravel. 24" RECOVERY	A-4(5)	
35.42		13	34.0' – 36.0'	1 2 3 4	Saturated loose gray coarse sand w/some fine sand and fine gravel, trace of silt. 18" RECOVERY	A-1-b	

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

Project Name: Wilmington Viaduct
State Contract: T201407404
Boring No.: R-2

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
37.95							
		14	39.0'	3 8 14 18	Saturated medium dense gray coarse sand w/ some fine gravel and silt, trace of fine sand.	A-1-b	
40.48			41.0'		17" RECOVERY		
43.01							
		15	44.0'	24 48 50/3	Saturated very dense gray coarse to fine sand w/some silt, trace of fine gravel.	A-1-b	
45.54		R-1	45.5' 46.0'		Granite 20" RECOVERY		Recovery 95% RQD 50%
48.07							
50.6			50.5'		57" RECOVERY		
		R-2	50.5'		Brown clayey decomposed rock.		Recovery 33.3% RQD 0%
53.13							
55.66			55.5'		20" RECOVERY		
					End Boring		
58.19							

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

BORING R-3

Project Name: Wilmington Viaduct
Location: Wilmington, DE
State Contract #: T201407404
Federal Contract #:
Station/Offset:
Northing: 633748
Easting: 615076
Boring Surface Elev.:
Reference:
Date Started: 6/27/17
Date Completed: 6/27/17

Wt. of Sample Hammer:	140	**Lbs.**	**Average Fall:**	30	**IN.**
Type of: D-Sampler:	Split Barrel	**O.D.**	**O.D. of Sampler:**	2	**IN.**
S-Sampler:		**O.D.**	**O.D. of Samp. Tube:**		**IN.**
U-Sampler:		**O.D.**	**O.D. of Samp. Tube:**		**IN.**
Core Bit:		**O.D.**	**O.D. of Rock Core:**		**IN.**

Hollow Stem Auger Diameter: 3 1/4" **Inches** **From Depth of:** 0.0 **To:** 39.5
Mud Rotary: **From Depth of:** **To:**

Water Level Readings

Date	Depth to Water (ft)	Caved Depth (ft)
6/27/17	10.4	
6/27/17	12.9	34.0

Boring Contractor: Walton Corporation
Equipment/Rig Type: CME 55 Trailer Rig
Driller: Jason Truver
Logged By: Randy Ferguson

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
		1	0.5' – 2.0'		Moist brown fine gravelly coarse sand w/ some fine sand and silt. 18" RECOVERY	A-1-b	Hot mix 2" Stone 4"
2.53		24	2.0' – 4.0'	2 2 3 2	Moist loose brown coarse sandy fine gravel w/some fine sand and silt. 6" RECOVERY	A-1-b	
5.06		3	4.0' – 6.0'	17 19 5 5	Moist very stiff brown clayey coarse sandy silt w/some fine sand, trace of fine gravel. 16" RECOVERY	A-4(2)	
7.59		4	6.0' – 8.0'	11 8 8 9	Moist stiff brown fine to coarse sandy silt w/ some clay, trace of fine gravel. 24" RECOVERY	A-4(0)	
10.12		5	8.0' – 10.0'	1 5 6 6	Moist medium dense brown silty coarse to fine sand w/some fine gravel. 17" RECOVERY	A-1-b	
	■	6	10.0' – 12.0'	5 8 16 6	Saturated very stiff brown silty clay w/trace fine to coarse sand. 19" RECOVERY	A-6(13)	
12.65	▽	7	12.0' – 14.0'	WR WR 2 1	Saturated soft gray clayey coarse sandy silt w/some fine sand, trace of fine gravel. 20" RECOVERY	A-4(1)	

Remarks:

Reviewed By: Hany Fekry
Soils Supervisor: Aaron Wieczorek

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

Project Name: Wilmington Viaduct **Boring No.:** R-3
State Contract: T201407404

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
15.18		8	14.0' - 16.0'	1 2 3 4	Saturated firm gray clayey silt w/trace fine to coarse sand. 23" RECOVERY	A-4(12)	
17.71		9	16.0' - 18.0'	5 5 7 9	Saturated stiff gray clayey silt w/trace fine to coarse sand. 18" RECOVERY	A-4(8)	
20.24, 22.77		10	18.0' - 20.0'	5 6 6 7	No Recovery NR" RECOVERY		
25.3, 27.83		11	24.0' - 26.0'	1 3 5 7	Saturated firm gray silty clay w/some coarse to fine sand and fine gravel. 18" RECOVERY	A-6(7)	
30.36, 32.89		12	29.0' - 31.0'	6 7 9 13	Saturated medium dense gray clayey coarse to fine sand w/some fine gravel and silt. 19" RECOVERY	A-2-4(0)	
35.42		13	34.0' - 36.0'	1 6 20 8	Saturated medium dense brown fine gravelly coarse sand w/some fine sand and silt. 15" RECOVERY	A-1-b	

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

Project Name: Wilmington Viaduct
Boring No.: R-3
State Contract: T201407404

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
37.95							
		14	39.0' 39.5'	22 55/3	Saturated very dense gray clayey coarse sand and fine gravel w/some fine sand and silt.	A-2-6(0)	
		R-1	39.5'		6" RECOVERY Granite		Recovery 70% RQD 30%
40.48							
43.01			44.5'		42" RECOVERY		
		R-2	44.5'		Granite		Recovery 70% RQD 35%
45.54							
48.07			49.5'		42" RECOVERY		
					End Boring		
50.6							
53.13							
55.66							
58.19							

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

BORING R-4

Project Name: Wilmington Viaduct
Location: Wilmington, DE
State Contract #: T201407404
Federal Contract #:
Station/Offset:
Northing: 633794.169
Easting: 615065.435
Boring Surface Elev.:
Reference:
Date Started: 6/20/17
Date Completed: 6/20/17

Wt. of Sample Hammer: 140 **Lbs.** **Average Fall:** 30 **IN.**
Type of: **D-Sampler:** Split Barrel **O.D.** **O.D. of Sampler:** 2 **IN.**
S-Sampler: **O.D.** **O.D. of Samp. Tube:** **IN.**
U-Sampler: **O.D.** **O.D. of Samp. Tube:** **IN.**
Core Bit: **O.D.** **O.D. of Rock Core:** **IN.**

Hollow Stem Auger Diameter: 3 1/4" **Inches** **From Depth of:** 0.0 **To:** 50.0
Mud Rotary: **From Depth of:** **To:**

Water Level Readings

Date	Depth to Water (ft)	Caved Depth (ft)
6/20/17	25.0	
6/20/17	Dry	16.2

Boring Contractor: Walton Corporation
Equipment/Rig Type: CME 55 Truck Rig
Driller: Billy Holden
Logged By: Randy Ferguson

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
2.53		1	2.0' - 4.0'	50/2	No Recovery NR" RECOVERY		Hot-mix 4" Concrete 13" Stone 7"
5.06		2	4.0' - 6.0'	8 9 13 11	Moist medium dense brown clayey fine gravel w/some coarse to fine sand. 23" RECOVERY	A-2-4(0)	
7.59		3	6.0' - 8.0'	22 16 17 14	Moist dense brown clayey coarse sand w/ some fine sand, fine gravel and silt. 7" RECOVERY	A-2-4(0)	
10.12		4	8.0' - 10.0'	5 6 5 7	Moist stiff brown fine sandy fine gravelly silt w/some coarse sand. 21" RECOVERY	A-4(0)	
		5	10.0' - 12.0'	5 6 6 8	Moist medium dense brown coarse sandy fine gravel w/some fine sand and silt. 16" RECOVERY	A-1-b	
12.65		6	12.0' - 14.0'	1 2 1 2	pH Sample 24" RECOVERY		
15.18		7	14.0'	2 3 4 3	Wet firm brown clayey silt w/trace fine to coarse sand and fine gravel.	A-4(5)	

Remarks:

Reviewed By: Hany Fekry
Soils Supervisor: Aaron Wieczorek

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

Project Name: Wilmington Viaduct **Boring No.:** R-4
State Contract: T201407404

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
			16.0'		16" RECOVERY		
		8	16.0'	8 5 7 6	Wet stiff gray clayey silt w/trace fine to coarse sand and fine gravel.	A-4(7)	
17.71			18.0'		17" RECOVERY		
		9	18.0'	4 4 4 4	Wet firm gray silty clay w/some fine sand, trace of coarse sand.	A-6(12)	
			20.0'		18" RECOVERY		
20.24							
22.77							
		10	24.0'	50/2	Wet hard gray silty clay w/some fine to coarse sand.	A-6(7)	
25.3	■						
			26.0'		2" RECOVERY		
27.83							
		11	28.0'	11 13 12 15	Saturated very stiff brown fine sandy clay w/ some coarse sand and silt.	A-7-5(6)	
			30.0'		20" RECOVERY		
30.36							
32.89							
		12	34.0'	13 16 18 22	Saturated hard brown coarse to fine sandy clay w/some silt.	A-7-5(1)	
35.42							
			36.0'		16" RECOVERY		
37.95							
		13	38.0'	23	Saturated hard brown fine to coarse sandy	A-4(0)	

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

Project Name: Wilmington Viaduct
Boring No.: R-4
State Contract: T201407404

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
				50/2	silt.		
			40.0'		11" RECOVERY		
40.48							
43.01							
		14	44.0'	28 50/2	Saturated very dense brown silty fine to coarse sand and fine gravel.	A-2-4(0)	
45.54			46.0'		6" RECOVERY		
48.07		15	48.0'	50/2	Saturated dense brown silty fine to coarse sand w/trace fine gravel.	A-2-4(0)	
			50.0'		4" RECOVERY		
					End Boring		
50.6							
53.13							
55.66							
58.19							
60.72							

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

BORING R-5

Project Name: Wilmington Viaduct
Location: Wilmington, DE
State Contract #: T201407404
Federal Contract #:
Station/Offset:
Northing: 633900
Easting: 615098
Boring Surface Elev.:
Reference:
Date Started: 6/26/17
Date Completed: 6/26/17

Wt. of Sample Hammer:	140	**Lbs.**	**Average Fall:**	30	**IN.**
Type of: D-Sampler:	Split Barrel	**O.D.**	**O.D. of Sampler:**	2	**IN.**
S-Sampler:		**O.D.**	**O.D. of Samp. Tube:**		**IN.**
U-Sampler:		**O.D.**	**O.D. of Samp. Tube:**		**IN.**
Core Bit:		**O.D.**	**O.D. of Rock Core:**		**IN.**

Hollow Stem Auger Diameter: 3 1/4" **Inches** **From Depth of:** 0.0 **To:** 40.0
Mud Rotary: **From Depth of:** **To:**

Water Level Readings

Date	Depth to Water (ft)	Caved Depth (ft)
6/26/17	28.0	
6/26/17	16.0	26.0

Boring Contractor: Walton Corporation
Equipment/Rig Type: CME 55 Truck
Driller: Billy Holden
Logged By: Randy Ferguson

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
2.53		1	2.0' – 4.0'	6 16 16 23	Moist dense brown silty fine sand and fine gravel w/some coarse sand. 14" RECOVERY pH Sample	A-1-b	Hot mix and Stone Base
5.06		2	4.0' – 6.0'	18 14 13 11	12" RECOVERY		
7.59		3	6.0' – 8.0'	14 9 9 8	Wet medium dense brown silty coarse sand w/some fine sand and fine gravel. 12" RECOVERY	A-2-4(0)	
		4	8.0' – 10.0'	4 7 7 13	Wet medium dense brown silty fine to coarse sand w/trace fine gravel. 17" RECOVERY	A-2-4(0)	
10.12		5	10.0' – 12.0'	5 6 7 6	Wet stiff brown clayey fine clayey silt w/ trace coarse to fine sand. 18" RECOVERY	A-4(1)	
12.65		6	12.0' – 14.0'	6 7 8 10	Wet stiff gray clayey silt w/trace coarse to fine sand. 22" RECOVERY	A-4(5)	
15.18		7	14.0'	7 5 9 11	Wet stiff gray silt w/trace fine to coarse sand and clay.	A-4(2)	

Remarks:

Reviewed By: Hany Fekry
Soils Supervisor: Aaron Wieczorek

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

Project Name: Wilmington Viaduct
State Contract: T201407404
Boring No.: R-5

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
			16.0'		13" RECOVERY		
17.71		8	16.0' - 18.0'	9 10 11 10	Wet very stiff gray silt w/some clay, trace of fine to coarse sand and fine gravel. 24" RECOVERY	A-4(4)	
20.24		9	18.0' - 20.0'	4 4 6 8	Wet stiff brown clayey silt w/trace fine to coarse sand and fine gravel. 24" RECOVERY	A-4(5)	
22.77							
25.3		10	24.0' - 26.0'	3 3 7 7	Wet stiff brown silty clay w/some fine to coarse sand, trace of fine gravel. 24" RECOVERY	A-6(7)	
27.83							
		11	28.0' - 30.0'	4 6 9 11	Wet medium dense gray clayey coarse to fine sand w/some fine gravel and silt. 21" RECOVERY	A-2-6(0)	
30.36							
32.89							
35.42		12	34.0' - 36.0'	11 13 16 26	Saturated medium dense gray silty coarse to fine sand. 21" RECOVERY	A-2-4(0)	
37.95							
		13	38.0'	11	Saturated dense gray silty fine to coarse sand	A-2-4(0)	

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

Project Name: Wilmington Viaduct **Boring No.:** R-5
State Contract: T201407404

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
				16 26 31	w/trace fine gravel.		
			40.0'		13" RECOVERY		
40.48							
		R-1	42.0'		Granite		Recovery 76.7% RQD 53.3%
43.01							
45.54							
			47.0'		46" RECOVERY		
		R-2	47.0'		Granite		Recovery 71.7% RQD 38.3%
48.07							
50.6							
			52.0'		43" RECOVERY		
					End Boring		
53.13							
55.66							
58.19							
60.72							

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

BORING R-6

Project Name: Wilmington Viaduct
Location: Wilmington, DE
State Contract #: T201407404
Federal Contract #:
Station/Offset:
Northing: 634073.402
Easting: 615090.106
Boring Surface Elev.:
Reference:
Date Started: 7/11/17
Date Completed: 7/11/17

Wt. of Sample Hammer: 140 **Lbs.** **Average Fall:** 30 **IN.**
Type of: **D-Sampler:** Split Barrel **O.D.** **O.D. of Sampler:** 2 **IN.**
S-Sampler: **O.D.** **O.D. of Samp. Tube:** **IN.**
U-Sampler: **O.D.** **O.D. of Samp. Tube:** **IN.**
Core Bit: **O.D.** **O.D. of Rock Core:** **IN.**

Hollow Stem Auger Diameter: 3 1/4" **Inches** **From Depth of:** 0.0 **To:** 36.0
Mud Rotary: **From Depth of:** **To:**

Water Level Readings

Date	Depth to Water (ft)	Caved Depth (ft)
7/11/17	20.0	
7/11/17	Dry	3.8

Boring Contractor: Walton Corporation
Equipment/Rig Type: CME 55 ATV
Driller: Jason Truver
Logged By: Randy Ferguson

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
		1	0.0' – 2.0'		Moist brown fine sandy silt w/some coarse sand, clay and fine gravel. 20" RECOVERY	A-4(0)	
2.53		2	2.0' – 4.0'		No Sample 20" RECOVERY		
5.06		3	4.0' – 6.0'	5 10 15 12	Moist very stiff gray clayey silt w/some fine to coarse sand, trace of fine gravel. 24" RECOVERY	A-4(2)	
7.59		4	6.0' – 8.0'	11 14 11 10	Moist very stiff gray clayey fine gravelly silt w/some fine to coarse sand. 24" RECOVERY	A-4(1)	
		5	8.0' – 10.0'	3 15 18 16	Moist dense brown coarse sand and fine gravel w/some fine sand and silt. 20" RECOVERY	A-1-b	
10.12		6	10.0' – 12.0'	15 17 17 19	Moist dense brown fine gravelly coarse sand w/some fine sand and silt. 23" RECOVERY	A-1-b	
12.65		7	12.0'	5 12 15 16	Moist medium dense brown fine gravel and coarse sand w/some fine sand and silt.	A-1-b	

Remarks:

Reviewed By: Hany Fekry
Soils Supervisor: Aaron Wieczorek

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

Project Name: Wilmington Viaduct
State Contract: T201407404
Boring No.: R-6

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
			14.0'		17" RECOVERY		
15.18		8	14.0' 16.0'	8 17 19 7	Moist dense brown coarse to fine sand and fine gravel w/some silt. 16" RECOVERY	A-1-b	
17.71		9	16.0' 18.0'	3 3 6 9	Saturated stiff brown silty clay w/some fine to coarse sand, trace of fine gravel. 16" RECOVERY	A-6(9)	
20.24 22.77	■	10	18.0' 20.0'	3 6 6 7	Saturated stiff brown silt w/some clay, trace of fine to coarse sand and fine gravel. 19" RECOVERY	A-4(2)	
25.3 27.83		11	24.0' 26.0'	2 3 4 11	Saturated firm gray fine sandy clay w/some coarse sand and silt, trace of fine gravel. 24" RECOVERY	A-6(4)	
30.36 32.89		12	29.0' 31.0'	5 10 15 12	Saturated very stiff gray fine to coarse sandy clay w/some silt. 23" RECOVERY	A-7-5(9)	
35.42		13	34.0' 36.0'	6 16 18 45	Saturated hard brown coarse to fine sandy clay w/some silt, trace of fine gravel. 18" RECOVERY	A-7-6(3)	

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

Project Name: Wilmington Viaduct **Boring No.:** R-6
State Contract: T201407404

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
		R-1	37.1'		Granite		Recovery 98.3% RQD 86%
37.95							
40.48							
			42.1'		59" RECOVERY		
		R-2	42.1'		Granite		Recovery 98.3% RQD 95%
43.01							
45.54							
			47.1'		59" RECOVERY		
					End Boring		
48.07							
50.6							
53.13							
55.66							
58.19							

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

BORING R-7

Project Name: Wilmington Viaduct
Location: Wilmington, DE
State Contract #: T201407404
Federal Contract #:
Station/Offset:
Northing: 634143.194
Easting: 615105.843
Boring Surface Elev.:
Reference:
Date Started: 7/25/17
Date Completed: 7/25/17

Wt. of Sample Hammer:	140	**Lbs.**	**Average Fall:**	30	**IN.**
Type of: D-Sampler:	Split Barrel	**O.D.**	**O.D. of Sampler:**	2	**IN.**
S-Sampler:		**O.D.**	**O.D. of Samp. Tube:**		**IN.**
U-Sampler:		**O.D.**	**O.D. of Samp. Tube:**		**IN.**
Core Bit:		**O.D.**	**O.D. of Rock Core:**		**IN.**

Hollow Stem Auger Diameter: 3 1/4" **Inches** **From Depth of:** 0.0 **To:** 36.0
Mud Rotary: **From Depth of:** **To:**

Water Level Readings

Date	Depth to Water (ft)	Caved Depth (ft)
7/25/17	21.0	
7/25/17	Dry	16.0

Boring Contractor: Walton Corporation
Equipment/Rig Type: CME 55 ATV
Driller: Billy Holden
Logged By: Randy Ferguson

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
		1	0.0' - 2.0'	4 5 3 3	Wet firm brown silt w/some fine to coarse sand, fine gravel and clay. 6" RECOVERY	A-4(0)	
2.53		2	2.0' - 4.0'	9 11 8 7	Wet very stiff brown silt w/some fine to coarse sand and clay, trace of fine gravel. 21" RECOVERY	A-4(0)	
5.06		3	4.0' - 6.0'	9 10 16 15	Wet very stiff brown silt w/some fine to coarse sand and clay, trace of fine gravel. 18" RECOVERY	A-4(1)	
7.59		4	6.0' - 8.0'	16 28 50/4	Wet hard brown clayey silt w/some coarse to fine sand and fine gravel. 13" RECOVERY	A-4(1)	
10.12		5	8.0' - 10.0'	11 16 20 15	Wet dense brown silty fine gravel w/some coarse to fine sand, trace of clay. 20" RECOVERY	A-2-4(0)	
		6	10.0' - 12.0'	11 14 15 16	Wet medium dense brown coarse sand and fine gravel w/some fine sand and silt. 19" RECOVERY	A-1-b	
12.65		7	12.0'	5 6 6 7	Wet stiff brown clayey coarse sandy fine gravelly silt w/some fine sand.	A-4(0)	

Remarks:

Reviewed By: Hany Fekry
Soils Supervisor: Aaron Wieczorek

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

Project Name: Wilmington Viaduct **Boring No.:** R-7
State Contract: T201407404

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
			14.0'		16" RECOVERY		
15.18		8	14.0' 16.0'	5 6 7 9	Wet clayey silt w/trace fine to coarse sand. 21" RECOVERY	A-4(7)	
17.71		9	16.0' 18.0'	6 8 11 14	Wet very stiff brown clayey silt w/some fine sand, trace of coarse sand. 21" RECOVERY	A-4(7)	
		U-1	18.0' 20.0'		18" RECOVERY		Shelby Tube - Press Sample
20.24 22.77	■	10	20.0' 22.0'	13 12 14 16	Wet medium dense brown coarse to fine sand and fine gravel w/some silt. 20" RECOVERY	A-1-b	
25.3 27.83		11	24.0' 26.0'	14 16 50/4	Wet very dense brown clayey fine to coarse sand and fine gravel w/some silt. 17" RECOVERY	A-2-6(0)	
30.36 32.89		12	29.0' 31.0'	8 8 11 22	Wet very stiff brown fine to coarse sandy clay w/some fine gravel and silt. 22" RECOVERY	A-6(0)	
35.42		13	34.0' 36.0'	25 25 50/4	Wet very dense brown silty fine to coarse sand w/trace fine gravel. 16" RECOVERY	A-2-4(0)	
					End Boring		

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

BORING R-8

Project Name: Wilmington Viaduct
Location: Wilmington, DE
State Contract #: T201407404
Federal Contract #:
Station/Offset:
Northing: 634231
Easting: 615175
Boring Surface Elev.:
Reference:
Date Started: 7/26/17
Date Completed: 7/26/17

Wt. of Sample Hammer: 140 **Lbs.** **Average Fall:** 30 **IN.**
Type of: **D-Sampler:** Split Barrel **O.D.** **O.D. of Sampler:** 2 **IN.**
S-Sampler: **O.D.** **O.D. of Samp. Tube:** **IN.**
U-Sampler: **O.D.** **O.D. of Samp. Tube:** **IN.**
Core Bit: **O.D.** **O.D. of Rock Core:** **IN.**

Hollow Stem Auger Diameter: 3 1/4" **Inches** **From Depth of:** 0.0 **To:** 30.0
Mud Rotary: **From Depth of:** **To:**

Water Level Readings

Date	Depth to Water (ft)	Caved Depth (ft)
7/26/17	Dry	
7/26/17	Dry	25.0

Boring Contractor: Walton Corporation
Equipment/Rig Type: CME 45 Skid Rig
Driller: Billy Holden
Logged By: Randy Ferguson

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
		1	0.0' 2.0'	8 5 6 4	Moist stiff gray fine sandy silt w/some coarse sand, fine gravel and clay. 16" RECOVERY	A-4(0)	
2.53		2	2.0' 4.0'	6 6 5 5	Moist stiff gray silt w/some fine to coarse sand, fine gravel and clay. 16" RECOVERY	A-4(1)	
5.06		3	4.0' 6.0'	16 50/4	Moist hard gray clayey silt w/some coarse to fine sand and fine gravel. 16" RECOVERY	A-4(3)	
7.59		4	6.0' 8.0'	6 6 7 7	Wet stiff gray silty clay w/some fine to coarse sand and fine gravel. 20" RECOVERY	A-6(6)	
10.12		5	8.0' 10.0'	3 3 4 4	Saturated firm brown clay w/some coarse to fine sand and silt, trace of fine gravel. 22" RECOVERY	A-7-5(15)	
		6	10.0' 12.0'	14 14 8 5	Saturated very stiff brown coarse to fine sandy clay w/some silt. 24" RECOVERY	A-7-5(1)	
12.65		7	12.0'	6 7 8 8	Saturated stiff brown coarse sandy clay w/ some fine sand and silt.	A-7-5(18)	

Remarks:

Reviewed By: Hany Fekry
Soils Supervisor: Aaron Wieczorek

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

Project Name: Wilmington Viaduct
State Contract: T201407404
Boring No.: R-8

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
			14.0'		18" RECOVERY		
15.18		8	14.0' – 16.0'	6 14 14 5	Saturated very stiff brown coarse sandy clay w/some fine sand and silt. 24" RECOVERY	A-7-5(9)	
17.71		9	16.0' – 18.0'	11 16 13 13	Saturated very stiff brown fine to coarse sandy silt. 14" RECOVERY	A-4(0)	
20.24		10	18.0' – 20.0'	8 10 7 14	Saturated very stiff brown coarse sandy clay w/some fine sand and silt. 20" RECOVERY	A-7-5(15)	
22.77							
25.3		11	24.0' – 26.0'	9 13 16 17	Saturated stiff gray fine to coarse sandy silt. 18" RECOVERY	A-5(0)	
27.83		12	28.0' – 30.0'	50/1	No Recovery. NR" RECOVERY		
30.36					End Boring		
32.89							
35.42							

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

BORING R-9

Project Name: Wilmington Viaduct
Location: Wilmington, DE
State Contract #: T201407404
Federal Contract #:
Station/Offset:
Northing: 634313
Easting: 615220
Boring Surface Elev.:
Reference:
Date Started: 7/27/17
Date Completed: 7/27/17

Wt. of Sample Hammer: 140 **Lbs.** **Average Fall:** 30 **IN.**
Type of: **D-Sampler:** Split Barrel **O.D.** **O.D. of Sampler:** 2 **IN.**
S-Sampler: **O.D.** **O.D. of Samp. Tube:** **IN.**
U-Sampler: **O.D.** **O.D. of Samp. Tube:** **IN.**
Core Bit: **O.D.** **O.D. of Rock Core:** **IN.**

Hollow Stem Auger Diameter: 3 1/4" **Inches** **From Depth of:** 0.0 **To:** 41.0
Mud Rotary: **From Depth of:** **To:**

Water Level Readings

Date	Depth to Water (ft)	Caved Depth (ft)
7/27/17	25.0	
7/27/17	22.5	28.0

Boring Contractor: Walton Corporation
Equipment/Rig Type: CME 55 Truck
Driller: Billy Holden
Logged By: Randy Ferguson

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
		1	0.0' - 2.0'	3 3 5 4	No sieve analysis - Indication of moist loose brown silty fine to coarse sand. 4" RECOVERY		
2.53		2	2.0' - 4.0'	4 4 4 4	No sieve analysis - Indication of moist loose brown silty fine to coarse sand. 5" RECOVERY		
5.06		3	4.0' - 6.0'	6 7 7 13	Moist medium dense brown clayey fine gravel w/some coarse to fine sand and silt. 15" RECOVERY	A-2-4(0)	
7.59		4	6.0' - 8.0'	8 9 10 13	Moist very stiff brown clayey fine gravelly silt w/some coarse to fine sand. 20" RECOVERY	A-4(0)	
		5	8.0' - 10.0'	4 4 5 5	pH Sample 14" RECOVERY		
10.12		6	10.0' - 12.0'	7 6 4 3	Wet loose brown clayey coarse sand and fine gravel w/some fine sand and silt. 14" RECOVERY	A-2-7(2)	
12.65		7	12.0'	3 3 4 3	Wet firm brown coarse sandy clay w/some fine sand, fine gravel and silt.	A-7-6(10)	

Remarks:

Reviewed By: Hany Fekry
Soils Supervisor: Aaron Wieczorek

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

Project Name: Wilmington Viaduct
Boring No.: R-9
State Contract: T201407404

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
			14.0'		13" RECOVERY		
15.18		8	14.0' – 16.0'	3 3 8 10	Wet stiff brown coarse sandy clay w/some fine sand, fine gravel and silt. 21" RECOVERY	A-7-6(7)	
17.71		9	16.0' – 18.0'	10 12 12 14	Wet very stiff brown coarse sandy clay w/ some fine sand and silt, trace of fine gravel. 22" RECOVERY	A-7-6(12)	
20.24 / 22.77	▽ (at 22.77)	10	18.0' – 20.0'	9 12 17 18	Wet very stiff gray fine sandy clay w/some coarse sand and silt, trace of fine gravel. 23" RECOVERY	A-7-5(9)	
25.3	■ (at 25.3)	11	24.0' – 26.0'	9 10 16 18	Wet very stiff gray fine sandy clay w/some coarse sand and silt. 23" RECOVERY	A-7-5(8)	
27.83 / 30.36 / 32.89		12	28.0' – 30.0'	17 28 50/3	Wet hard gray fine to coarse sandy clay w/ some silt. 16" RECOVERY	A-6(2)	
35.42		13	34.0' – 36.0'	31 50/3	Wet very dense gray clayey fine to coarse sand w/some fine gravel and silt. 10" RECOVERY	A-2-4(0)	

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

Project Name: Wilmington Viaduct

State Contract: T201407404

Boring No.: R-9

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
37.95							
		14	39.0'	50/1	No Recovery		
40.48			41.0'		NR" RECOVERY		
		R-1	42.0'		Granite		Recovery 100% RQD 66.7%
43.01							
45.54							
			47.0'		60" RECOVERY		
		R-2	47.0'		Granite		Reovery 98.3% RQD 100%
48.07							
50.6							
			52.0'		59" RECOVERY		
					End Boring		
53.13							
55.66							
58.19							

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

BORING R-10

Project Name: Wilmington Viaduct **Location:** Wilmington, DE
State Contract #: T201407404 **Federal Contract #:**
Station/Offset: **Northing:** 634491 **Easting:** 615301
Boring Surface Elev.: **Reference:**
Date Started: 7/31/17 **Date Completed:** 7/31/17

Wt. of Sample Hammer:	140	**Lbs.**	**Average Fall:**	30	**IN.**
Type of: D-Sampler:	Split Barrel	**O.D.**	**O.D. of Sampler:**	2	**IN.**
S-Sampler:		**O.D.**	**O.D. of Samp. Tube:**		**IN.**
U-Sampler:		**O.D.**	**O.D. of Samp. Tube:**		**IN.**
Core Bit:		**O.D.**	**O.D. of Rock Core:**		**IN.**

Hollow Stem Auger Diameter: 3 1/4" **Inches** **From Depth of:** 0.0 **To:** 20.0
Mud Rotary: **From Depth of:** **To:**

Water Level Readings

Date	Depth to Water (ft)	Caved Depth (ft)
7/31/17	11.0	
7/31/17	Dry	2.5

Boring Contractor: Walton Corporation **Equipment/Rig Type:** CME 55 Truck
Driller: Billy Holden **Logged By:** Randy Ferguson

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
		1	0.0' – 2.0'		Moist gray fine gravel w/some coarse sand and silt, trace of fine sand. 20" RECOVERY	A-1-b	
2.53		2	2.0' – 4.0'	4 4 3 4	Moist loose brown clayey coarse to fine sand w/some silt, trace of fine gravel. 18" RECOVERY	A-2-4(0)	
5.06		3	4.0' – 6.0'	2 1 1 2	pH Sample 10" RECOVERY		
7.59		4	6.0' – 8.0'	2 3 3 2	Moist loose brown clayey fine gravel w/ some coarse to fine sand and silt. 12" RECOVERY	A-2-4(0)	
10.12		5	8.0' – 10.0'	4 3 6 5	Wet stiff brown coarse sandy fine gravelly clay w/some silt, trace of fine sand. 22" RECOVERY	A-7-5(10)	
	■	6	10.0' – 12.0'	11 10 11 11	Saturated very stiff brown coarse sandy clay w/some fine sand and silt, trace of fine gravel. 22" RECOVERY	A-7-5(14)	
12.65		7	12.0'	3 4 7 6	Saturated stiff brown coarse sandy fine gravelly clay w/some silt, trace of fine sand.	A-7-5(9)	

Remarks:

Reviewed By: Hany Fekry **Soils Supervisor:** Aaron Wieczorek

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

Project Name: Wilmington Viaduct
Boring No.: R-10
State Contract: T201407404

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
			14.0'		24" RECOVERY		
15.18		8	14.0' - 16.0'	3 8 13 9	Saturated very stiff brown fine gravelly clay w/some coarse sand and silt, trace of fine sand. 22" RECOVERY	A-7-5(9)	
17.71		9	16.0' - 18.0'	11 13 20 18	Saturated dense brown clayey coarse to fine sand w/some fine gravel and silt. 24" RECOVERY	A-2-7(2)	
20.24		10	18.0' - 20.0'	13 50/2	Saturated hard gray coarse to fine sandy clay w/some silt and fine gravel. 5" RECOVERY	A-6(1)	
22.77 / 25.3		R-1	22.5' - 27.5'		Granite 60" RECOVERY		Recovery 100% RQD 73.3%
27.83 / 30.36		R-2	27.5' - 32.5'		Granite 44" RECOVERY		Recovery 73.3% RQD 73.3%
32.89 / 35.42					End Boring		

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

BORING R-11

Project Name: Wilmington Viaduct
Location: Wilmington, DE
State Contract #: T201407404
Federal Contract #:
Station/Offset:
Northing: 634604
Easting: 615364
Boring Surface Elev.:
Reference:
Date Started: 8/1/17
Date Completed: 8/1/17

Wt. of Sample Hammer: 140 **Lbs.** **Average Fall:** 30 **IN.**
Type of: **D-Sampler:** Split Barrel **O.D.** **O.D. of Sampler:** 2 **IN.**
S-Sampler: **O.D.** **O.D. of Samp. Tube:** **IN.**
U-Sampler: **O.D.** **O.D. of Samp. Tube:** **IN.**
Core Bit: **O.D.** **O.D. of Rock Core:** **IN.**

Hollow Stem Auger Diameter: 3 1/4" **Inches** **From Depth of:** 0.0 **To:** 31.0
Mud Rotary: **From Depth of:** **To:**

Water Level Readings

Date	Depth to Water (ft)	Caved Depth (ft)
8/1/17	19.0	
8/1/17	Dry	4.8

Boring Contractor: Walton Corporation
Equipment/Rig Type: CME 55 Truck
Driller: Billy Holden
Logged By: Randy Ferguson

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
		1	0.0' - 2.0'		Wet gray fine gravelly clay w/some coarse sand and silt, trace of fine sand. 20" RECOVERY	A-7-6(8)	
2.53		2	2.0' - 4.0'	11 12 12 14	Wet medium dense brown silty coarse sandy fine gravel w/some fine sand. 6" RECOVERY	A-1-b	
5.06		3	4.0' - 6.0'	9 13 15 9	Wet very stiff brown coarse to fine sandy clay w/some fine gravel and silt. 10" RECOVERY	A-7-5(7)	
7.59		4	6.0' - 8.0'	11 15 16 19	Wet hard brown fine sandy clay w/some coarse sand, fine gravel and silt. 20" RECOVERY	A-7-5(7)	
10.12		5	8.0' - 10.0'	9 13 15 9	Wet very stiff brown clay w/some fine to coarse sand, fine gravel and silt. 22" RECOVERY	A-7-5(10)	
12.65		6	10.0' - 12.0'	18 20 20 17	Wet hard brown fine sandy clay w/some silt, trace of coarse sand and fine gravel. 20" RECOVERY	A-7-6(7)	
		7	12.0'	8 8 9 8	Wet very stiff brown fine sandy clay w/ some coarse sand, fine gravel and silt.	A-7-6(6)	

Remarks:

Reviewed By: Hany Fekry
Soils Supervisor: Aaron Wieczorek

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

Project Name: Wilmington Viaduct
Boring No.: R-11
State Contract: T201407404

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
			14.0'		23" RECOVERY		
15.18		8	14.0' – 16.0'	4 10 13 10	Wet very stiff brown fine sandy clay w/ some coarse sand, fine gravel and silt. 23" RECOVERY	A-7-5(3)	
17.71		9	16.0' – 18.0'	16 18 20 18	Wet hard brown fine to coarse sandy clay w/ some fine gravel and silt. 23" RECOVERY	A-7-5(5)	
20.24	■	10	18.0' – 20.0'	9 13 18 13	Wet dense gray clayey fine gravel w/some coarse to fine sand and silt. 12" RECOVERY	A-2-7(2)	
22.77							
25.3		11	24.0' – 26.0'	9 16 19 20	Saturated hard gray coarse to fine sandy clay w/some silt, trace of fine gravel. 22" RECOVERY	A-7-5(3)	
27.83							
30.36		12	29.0' – 31.0'	13 24 31 31	Saturated very dense brown clayey fine to coarse sand w/some silt, trace of fine gravel. 23" RECOVERY	A-2-7(1)	
32.89					End Boring		
35.42							

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

BORING R-12

Project Name: Wilmington Viaduct
Location: Wilmington, DE
State Contract #: T201407404
Federal Contract #:
Station/Offset:
Northing: 634530
Easting: 615531
Boring Surface Elev.:
Reference:
Date Started: 7/18/17
Date Completed: 7/18/17

Wt. of Sample Hammer: 140 **Lbs.**
Average Fall: 30 **IN.**
Type of: D-Sampler: Split Barrel **O.D.**
O.D. of Sampler: 2 **IN.**
S-Sampler: **O.D.**
O.D. of Samp. Tube: **IN.**
U-Sampler: **O.D.**
O.D. of Samp. Tube: **IN.**
Core Bit: **O.D.**
O.D. of Rock Core: **IN.**

Hollow Stem Auger Diameter: 3 1/4" **Inches**
From Depth of: 0.0 **To:** 30.0
Mud Rotary:
From Depth of: **To:**

Water Level Readings

Date	Depth to Water (ft)	Caved Depth (ft)
7/18/17	Dry	
7/18/17	Dry	7.6

Boring Contractor: Walton Corporation
Equipment/Rig Type: CME 55 Truck
Driller: Dave Burt
Logged By: Randy Ferguson

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
		1	0.0' – 2.0'		Moist brown coarse sandy fine gravel w/ some fine sand and silt. 18" RECOVERY	A-1-b	
2.53		2	2.0' – 4.0'	25 12 12 13	Wet very stiff brown clayey silt w/some fine gravel, trace of coarse to fine sand. 12" RECOVERY	A-4(5)	
5.06		3	4.0' – 6.0'	3 4 5 7	Wet stiff brown fine gravelly clay w/some coarse to fine sand and silt. 11" RECOVERY	A-7-6(6)	
7.59		4	6.0' – 8.0'	6 7 14 14	Wet very stiff brown fine sandy clay w/ some coarse sand, fine gravel and silt. 17" RECOVERY	A-7-5(4)	
10.12		5	8.0' – 10.0'	3 6 7 8	Wet stiff brown fine sandy clay w/some fine gravel and silt, trace of coarse sand. 17" RECOVERY	A-7-5(5)	
		6	10.0' – 12.0'	9 11 16 14	Saturated very stiff brown fine sandy fine gravelly clay w/some silt, trace of coarse sand. 24" RECOVERY	A-7-5(6)	
12.65		7	12.0'	4 6 9 12	Saturated medium dense brown silty fine gravel w/some fine sand, trace of coarse sand.	A-2-7(2)	

Remarks:

Reviewed By: Hany Fekry
Soils Supervisor: Aaron Wieczorek

STATE OF DELAWARE
DEPARTMENT OF TRANSPORTATION
MATERIALS AND RESEARCH

Project Name: Wilmington Viaduct
Boring No.: R-12
State Contract: T201407404

Depth (ft.)	Water Level	No.	Sample Depth	Blows/6"	Sample Description	AASHTO Class.	Remarks
			14.0'		18" RECOVERY		
15.18		8	14.0' – 16.0'	3 6 10 15	Saturated medium dense brown clayey fine sand and fine gravel w/some coarse sand and silt. 24" RECOVERY	A-2-7(3)	
17.71		9	16.0' – 18.0'	12 12 15 20	Saturated very stiff brown fine sandy fine gravelly clay w/some silt, trace of coarse sand. 22" RECOVERY	A-7-5(3)	
20.24		10	18.0' – 20.0'	3 8 13 14	Saturated very stiff brown fine sandy clay w/ some coarse sand and silt. 19" RECOVERY	A-7-5(5)	
22.77							
25.3		11	24.0' – 26.0'	10 17 42 27	Saturated hard brown fine sandy clay w/ some silt, trace of coarse sand and fine gravel. 22" RECOVERY	A-7-5(4)	
27.83		12	28.0' – 30.0'	50/4	Saturated very dense gray fine gravel w/some silt, trace of coarse to fine sand. 3" RECOVERY	A-1-a	
30.36					End Boring		
32.89							
35.42							

KEY TO SYMBOLS

Symbol Description

Strata symbols

Well graded gravels and sands

Silty soils

Plastic clays

Poorly graded, silty or clayey sands and gravel

Expansive plastic clays

Elastic Silts

Misc. Symbols

Description not given for: "WATER2"

Notes:

1. Exploratory borings were drilled using a 3 1/4 - inch diameter hollow stem auger.

2. These logs are subject to the limitations, conclusions, and recommendations in this report.

3. All blow counts are uncorrected.

Appendix C

September 18, 2017

To: Walton Corporation
P.O.Box 1097
Newark, Delaware 19715

Attn: Mr. Randolph Ferguson

Re: **Direct Shear Test Results**
ETS Report No.: ETS-17T219-10
Task 131, T201407404 Wilmington Viaduct Widening

Dear Mr. Ferguson:

Engineering & Testing Services, Inc. (ETS) is pleased to submit the completed Direct Shear Test results performed at our AASHTO accredited soils laboratory in Virginia Beach, Virginia. On August 21, 2017, Walton Corporation personnel delivered a Shelby tube soil sample to our laboratory. The Shelby tube was labeled as R-7, depth 18 to 20 feet, Task 131, T201407404. The following describes the procedures, methodologies, and apparatuses used to complete the test.

1. Test Specimen Extrusion and Preparation

The undisturbed soil sample was carefully extruded from the Shelby tube to ensure minimal disturbance. The undisturbed soil specimens were trimmed from the soil sample per ASTM standards, and immediately transferred into the direct shear machine for testing. A Geotechnical Engineer from our office performed the direct shear test in accordance with ASTM Standard (ASTM D3080 Standard Test Method for Direct Shear Test of Soils under Consolidated Drained Conditions). The test was conducted by using the most advanced direct shear test machine available in the market produced by **HUMBOLDT** as illustrated in Figure 1.

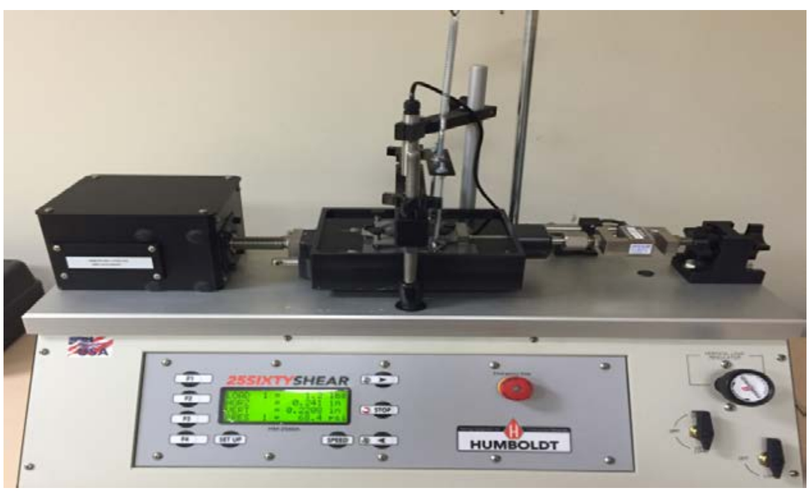


Figure 1: Direct Sheart Test Machine

Three soil specimens were prepared to perform the direct shear test at four different normal loads. The soil specimens were designated as Specimens A through C. The normal loads applied on each specimen during testing are listed in Table 1.

Table 1: Applied normal stress on the soil specimens	
Soil designation	Normal stress (Tsf)
A	0.5
B	1
C	2

2. Consolidation Stage

Before applying the respective normal stresses on the specimens, the shear box container was filled with water to achieve saturation, and kept full for the duration of the test. To initiate the consolidation stage, the specified normal stresses as shown in Table 1 were applied to each soil specimen. The normal stresses were maintained until the primary consolidation was achieved. The completion of primary consolidation was decided based on either of the following: interpretation of time versus normal vertical deformation or a waiting period of at least 24 hours. The total estimated elapsed time to failure was calculated based on the time required to achieve 90 percent consolidation as illustrated in Figure 2. Following, the displacement/strain rate was calculated and used during testing for shearing the specimen under drained condition according with ASTM D-3080-11.

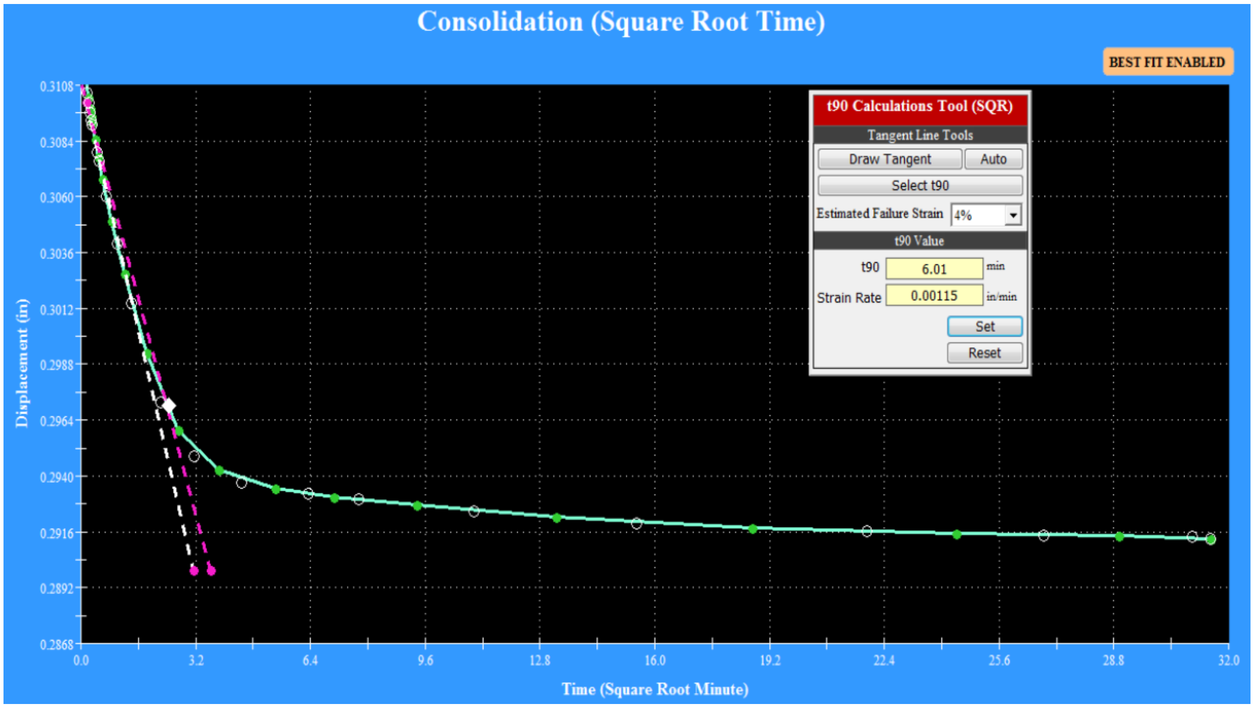


Figure 2: Determination of Time Required to Achieve 90% Consolidation

3. Shear Stage

Prior to applying the shear force on the soil specimens, the ETS geotechnical engineer computed the shearing rate based on the time required to achieve 90% consolidation. Accordingly, the calculated shearing rate was applied during the shearing of the specimens. According to ASTM-D3080, the soil specimen should be sheared within the range from 10 to 15 percent relative lateral displacement. Therefore, it was specified that the threshold value of the shear failure should be at 0.375-inch horizontal displacement and/or the shear force decreases while the horizontal displacement increases. When the sample reached failure due to the applied shear force, the test was terminated. A safety factor of 2.0 should be applied on all soils parameters. The test results should be used conservatively by the designers and are included in Appendix I of this report.

4. Laboratory Tests

At the time of testing, the soils specimens consisted of grey lean clay (CL) classified as A-4(6). The soil sample was subjected to natural moisture content testing (ASTM D2216), - #200 sieve (ASTM D422), Atterberg Limits (ASTM D4318) testing and visual classification (ASTM D2487, D2488) by the ETS engineering staff. All laboratory test procedures were conducted in accordance with applicable ASTM standards, under the supervision of a Professional Engineer licensed in the Commonwealth of Virginia and in strict adherence to the Quality Assurance program established and documented in the ETS Quality Manual and Quality test procedures. The classification laboratory test results are included in Appendix II of this report. The shear strength parameters of the soil should be used conservatively during design.

We appreciate the opportunity to be of service to you on this project. Should you have any questions regarding this report, do not hesitate to call our office at 757-306-1040.

Respectfully Submitted,
Engineering & Testing Services, Inc.

Charlie T. Nabhan, PE
Principal Geotechnical Engineer
VA License No.: 25133

Raju Acharya, PhD, PE
Geotechnical Engineer

APPENDICES

APPENDIX I – DIRECT SHEAR TEST RESULTS

APPENDIX II – CLASSIFICATION LABORATORY TEST RESULTS

APPENDIX I – DIRECT SHEAR TEST RESULTS

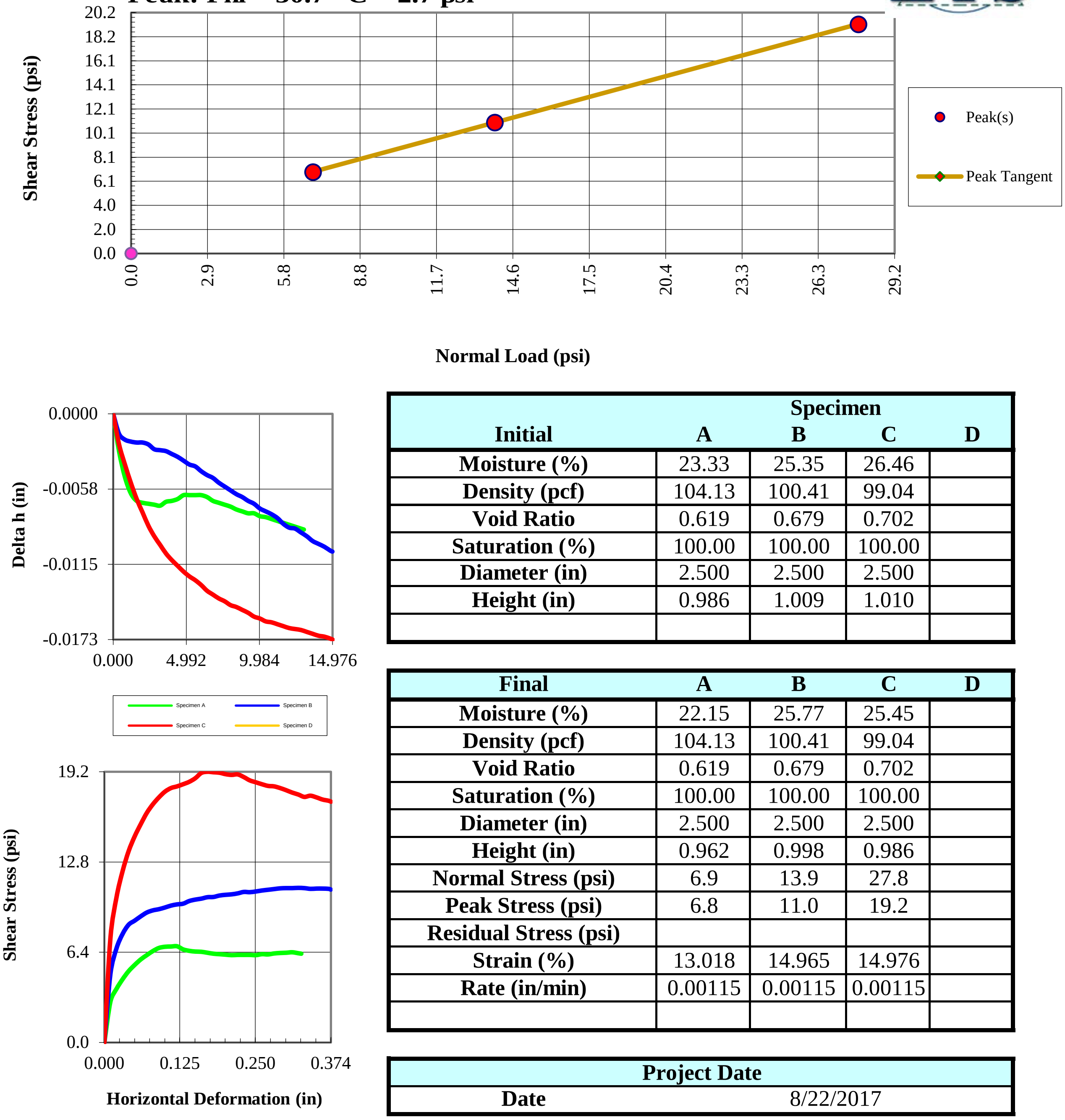
Engineering & Testing Services, Inc

Direct Shear Test (ASTM D3080)

Checked By C.T.N. Date 9/11/2017

Tested By R. Acharya Date 8/22/2017

Peak: Phi = 30.7 C = 2.7 psi



Initial	Specimen A	B	C	D
Moisture (%)	23.33	25.35	26.46	
Density (pcf)	104.13	100.41	99.04	
Void Ratio	0.619	0.679	0.702	
Saturation (%)	100.00	100.00	100.00	
Diameter (in)	2.500	2.500	2.500	
Height (in)	0.986	1.009	1.010	

Final	A	B	C	D
Moisture (%)	22.15	25.77	25.45	
Density (pcf)	104.13	100.41	99.04	
Void Ratio	0.619	0.679	0.702	
Saturation (%)	100.00	100.00	100.00	
Diameter (in)	2.500	2.500	2.500	
Height (in)	0.962	0.998	0.986	
Normal Stress (psi)	6.9	13.9	27.8	
Peak Stress (psi)	6.8	11.0	19.2	
Residual Stress (psi)				
Strain (%)	13.018	14.965	14.976	
Rate (in/min)	0.00115	0.00115	0.00115	

Project Date	
Date	8/22/2017

Project:	Task 131, T201407404 Wilmington Viaduct Widening
Location:	Delaware
Project Number:	ETS-17T219
Boring Number	R-7
Sample Number:	
Depth:	18'-20'
Sample Type:	Undisturbed
Description:	Gray Lean Clay (CL)
Test Type:	Direct Shear
Remarks:	

A B C N/A

Failure Photographs

Task 131 R-7 18-20.HSD

Engineering & Testing Services, Inc

Direct Shear Test (ASTM D3080)

Date 9/11/2017

Checked By C.T.N.

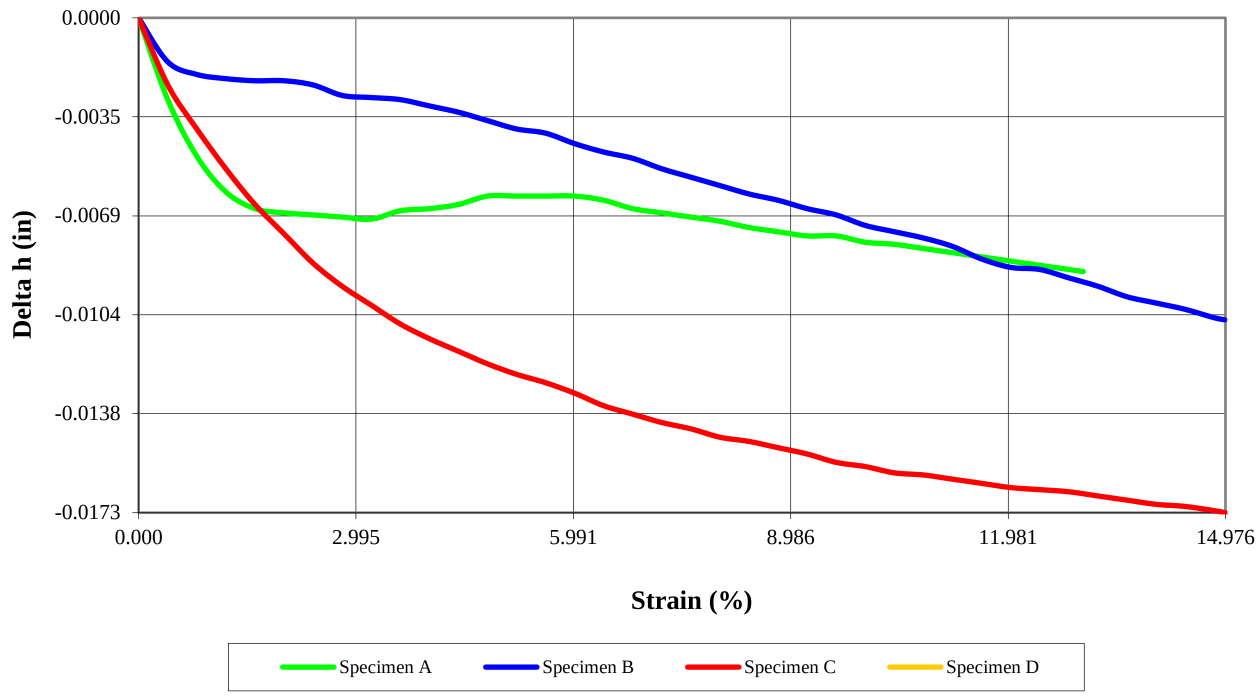
8/22/2017

Date

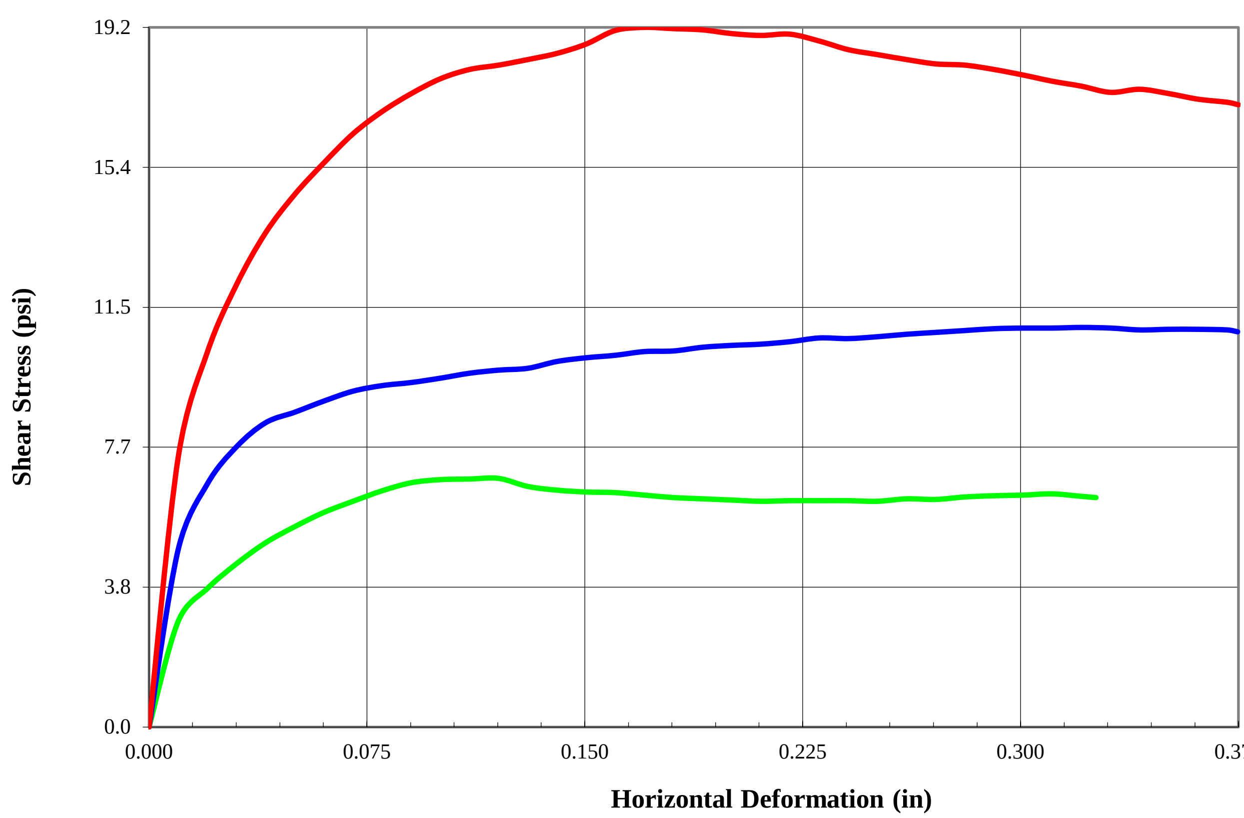
R. Acharya

Tested By

Delta h



Stress-Deformation



Task 131 R-7 18-20.HSD

Engineering & Testing Services, Inc

Direct Shear Test

Date 9/11/2017

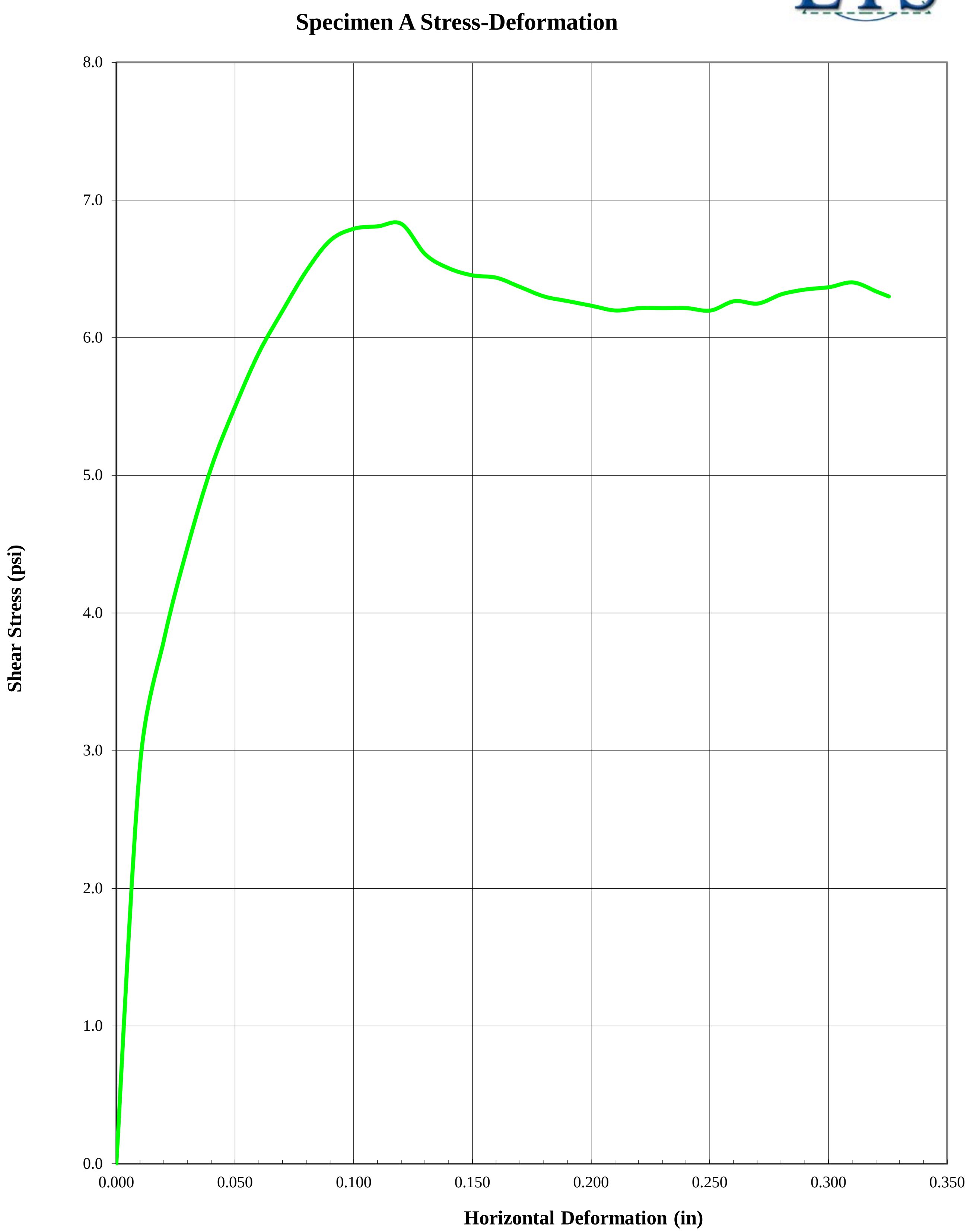
Checked By C.T.N.

8/22/2017

Date

R. Acharya

Tested By



Task 131 R-7 18-20.HSD

Engineering & Testing Services, Inc

Direct Shear Test

Date 9/11/2017

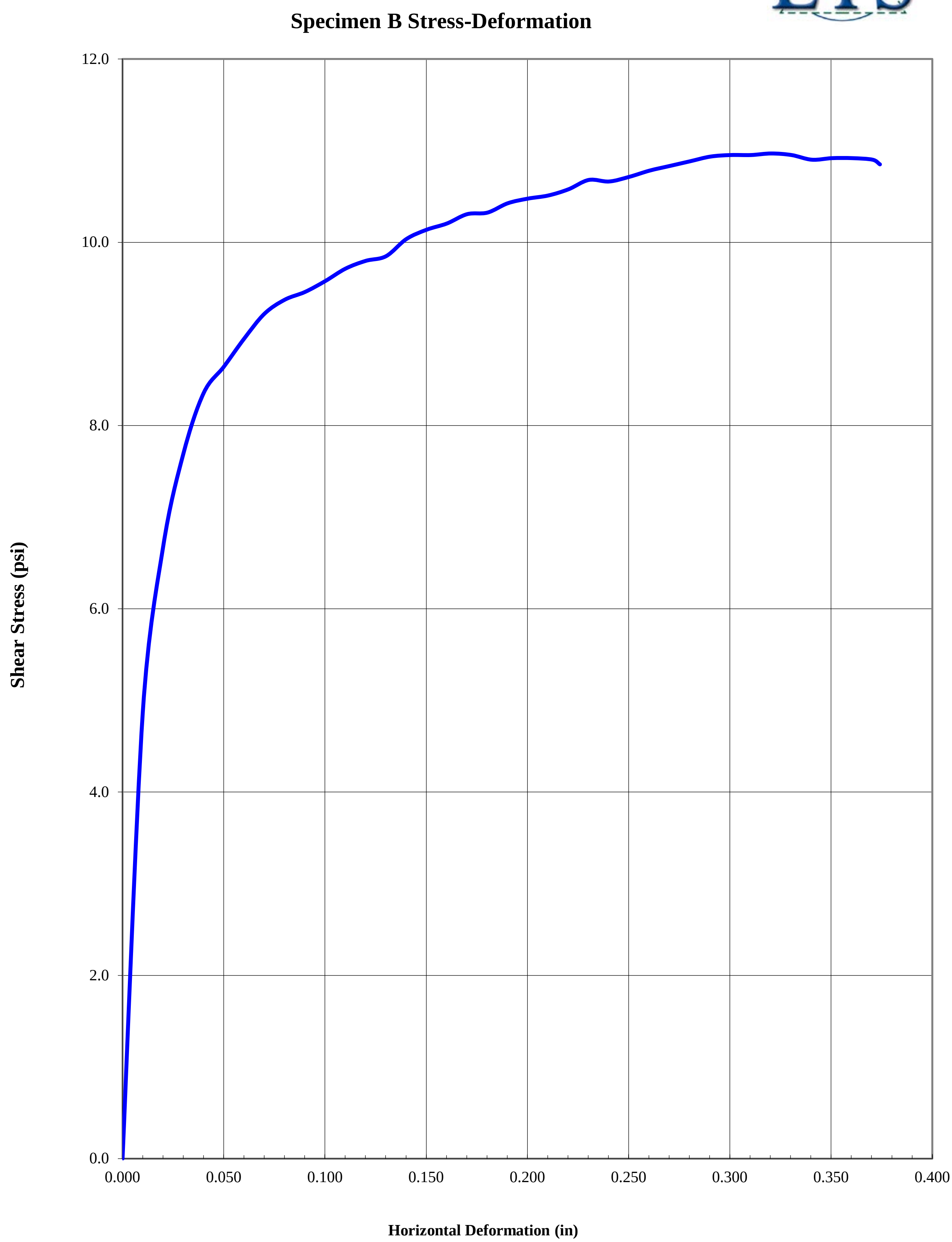
Checked By C.T.N.

8/22/2017

Date

R. Acharya

Tested By



Task 131 R-7 18-20.HSD

Engineering & Testing Services, Inc

Direct Shear Test

Date 9/11/2017

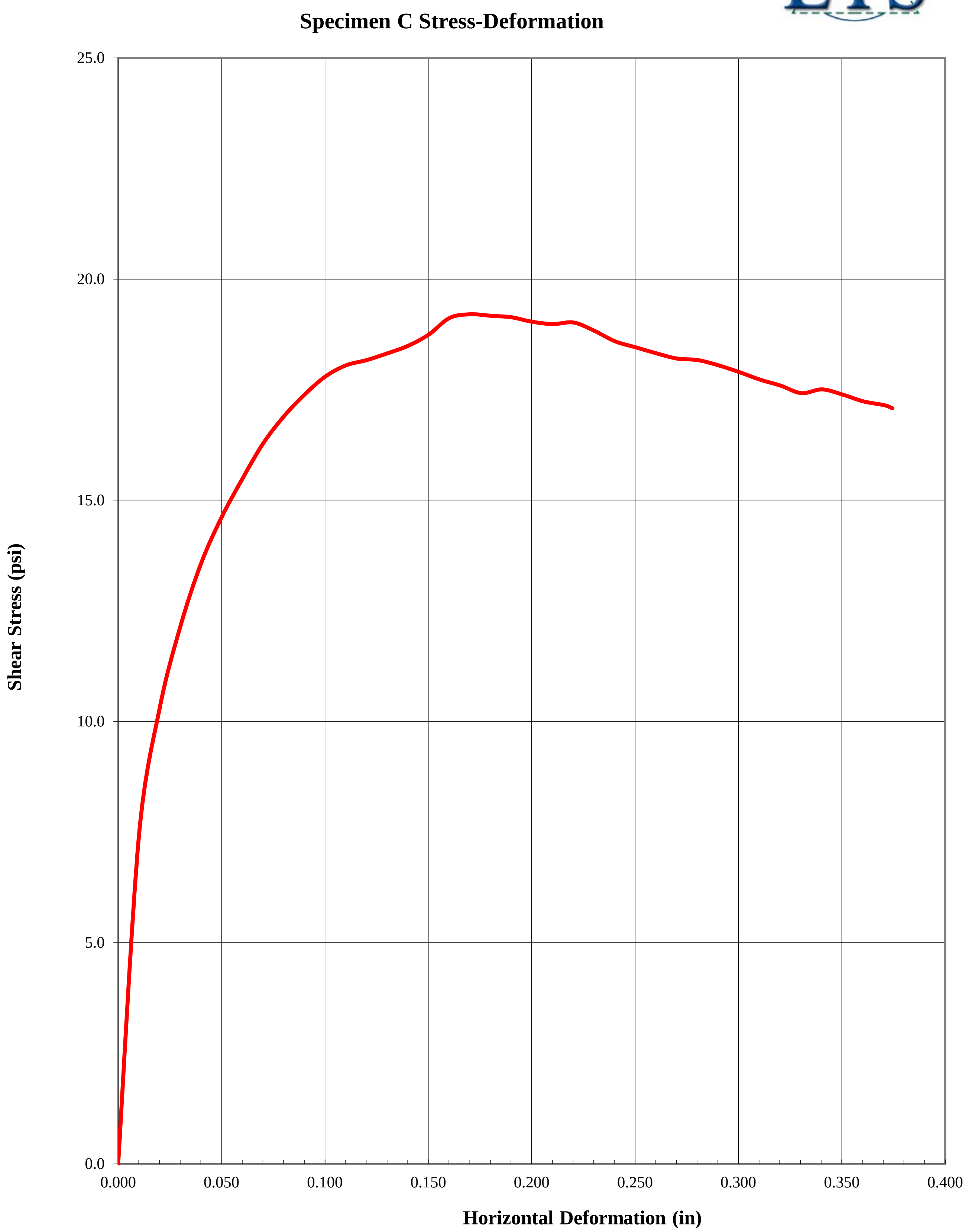
Checked By C.T.N.

8/22/2017

Date

R. Acharya

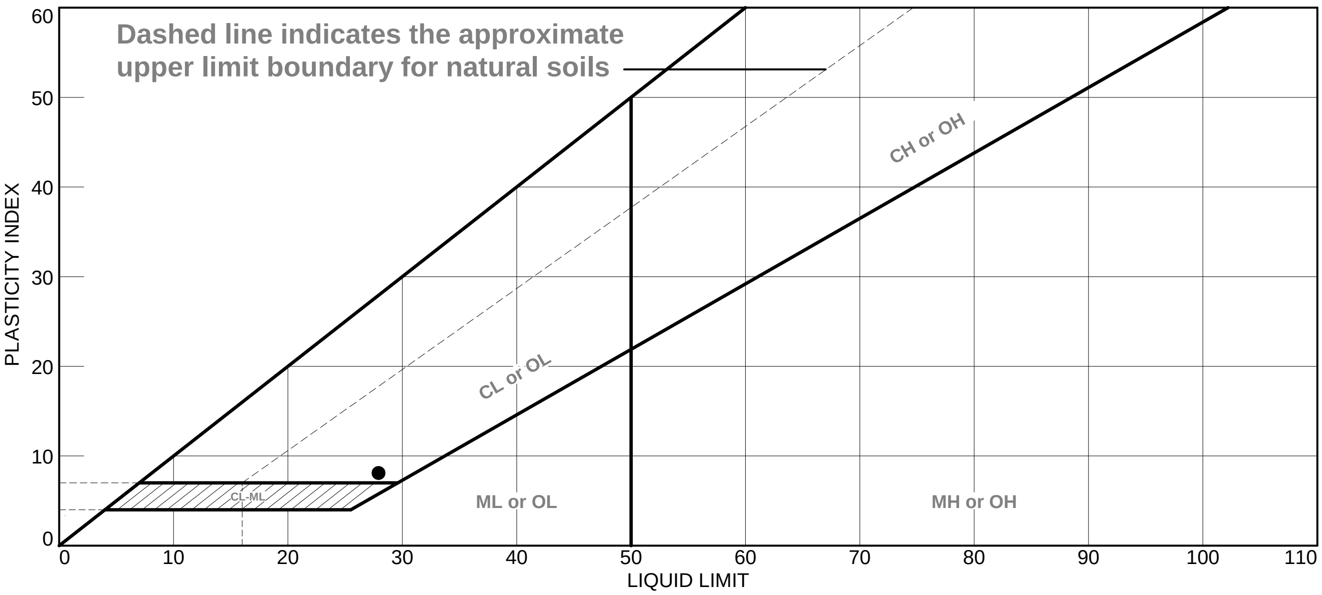
Tested By



Task 131 R-7 18-20.HSD

APPENDIX II – CLASSIFICATION LABORATORY TEST RESULTS

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Gray Lean CLay (CL)	28	20	8		86.4	CL

Project No. ETS-17T219 **Client:** WALTON CORPORATION

Project: TASK-131 T201407404 Wilmington Viaduct Widening

Sample Number: R-7 **Depth:** 18'-20'

ENGINEERING AND TESTING SERVICES, INC.

Virginia Beach, VA

Remarks:

Figure

Tested By: MM **Checked By:** R. Acharya, PhD, PE

ENGINEERING & TESTING SERVICES, INC.

July 27, 2017

To: Walton Corporation
P.O. Box 1097
Newark, Delaware 19715

Attn: Mr. Randy Ferguson

Re: **Reports of Uniaxial Compressive Strength of Intact Rock Core Specimen**
Delaware Department of Transportation
ETS Report No.: ETS-17T219-1
Task 131, T201407404, Wilmington Viaduct Widening

Dear Mr. Ferguson:

The purpose of this report is to submit the results of the compression tests conducted on four intact rock core specimens, designated as R-2, R-3, R-5, and R-6, at Engineering & Testing Services, Inc. (ETS) AASHTO accredited laboratory in Virginia Beach, Virginia. The rock cores were delivered to our laboratory by Walton Corporation personnel on July 19, 2017. The rock cores were prepared, measured, and tested in general accordance with ASTM Standard D7012, Method "C". All laboratory services performed by our personnel were supervised by a Professional Engineer from ETS. The reader is referred to the attached reports of Uniaxial Compressive Strength of Intact Rock Core Specimen for further details.

We appreciate the opportunity to be of service to you on this project. Should you have any questions regarding this letter, do not hesitate to contact our office at 757-306-1040.

Respectfully Submitted,
Engineering & Testing Services, Inc.

Charlie T. Nabhan, PE
Principal Geotechnical Engineer
VA License No.: 25133

Raju Acharya, PhD, PE
Geotechnical Engineer

5226 Indian River Road • Suite 103 • Virginia Beach, VA 23464 • *phone 757.306.1040* • *fax 757.306.1042*

REPORT OF UNIAXIAL COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN

CLIENT: Walton Corporation
PROJECT NAME: Task 131 T201407404 Wilmington Viaduct Widening
ETS Job NO.: ETS-17T219-2
MATERIAL DESCRIPTION: Grey and White, Granite
LOAD DIRECTION: Uniaxial
This Specimen was not prepared in accordance with ASTM D4543
Engineer: Raju Acharya, PhD, PE

COMPRESSION TEST RESULTS (ASTM D7012) - Method "C"

LABORATORY NUMBER	SPECIMEN IDENTIFICATION OR SET NO.	DEPTH (FT.)	DATE OF TEST	SPECIMEN DIAMETER (IN.)	SPECIMEN HEIGHT (IN.)	SPECIMEN AREA (SQ. IN.)	TOTAL LOAD (LBS.)	COMPRESSIVE STRENGTH (PSI)	AMBIENT TEMPERATURE AT TEST (F°)	BREAK TYPE	TIME TO FAILURE (MIN.)
Task 131	R-2	45-50	7/26/2017	2.0	4.0	3.1416	111,640	35,536	80	3	3.0

Respectfully Submitted,
Charlie T. Nabhan, PE
Principal Geotechnical Engineer

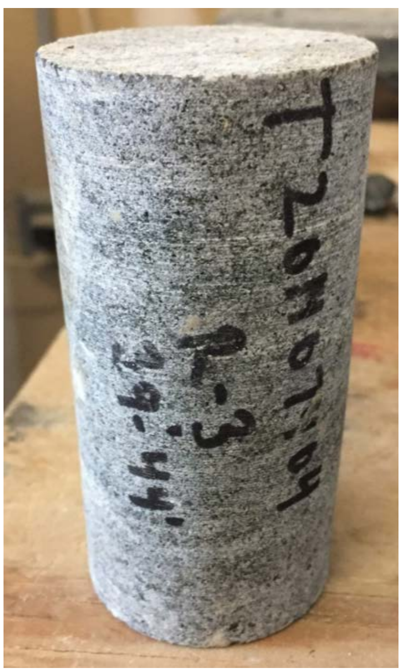
Respectfully Submitted,
Raju Acharya, PhD, PE
Geotechnical Engineer

REPORT OF UNIAXIAL COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN

CLIENT: Walton Corporation
PROJECT NAME: Task 131 T201407404 Wilmington Viaduct Widening
ETS Job NO.: ETS-17T219-3
MATERIAL DESCRIPTION: Grey and White, Granite
LOAD DIRECTION: Uniaxial
This Specimen was not prepared in accordance with ASTM D4543
Engineer: Raju Acharya, PhD, PE

COMPRESSION TEST RESULTS (ASTM D7012) - Method "C"

LABORATORY NUMBER	SPECIMEN IDENTIFICATION OR SET NO.	DEPTH (FT.)	DATE OF TEST	SPECIMEN DIAMETER (IN.)	SPECIMEN HEIGHT (IN.)	SPECIMEN AREA (SQ. IN.)	TOTAL LOAD (LBS.)	COMPRESSIVE STRENGTH (PSI)	AMBIENT TEMPERATURE AT TEST (F°)	BREAK TYPE	TIME TO FAILURE (MIN.)
Task 131	R-3	39-44	7/26/2017	2.0	4.0	3.1416	59,740	19,015	80	3	2.0



Respectfully Submitted,
Charlie T. Nabhan, PE
Principal Geotechnical Engineer

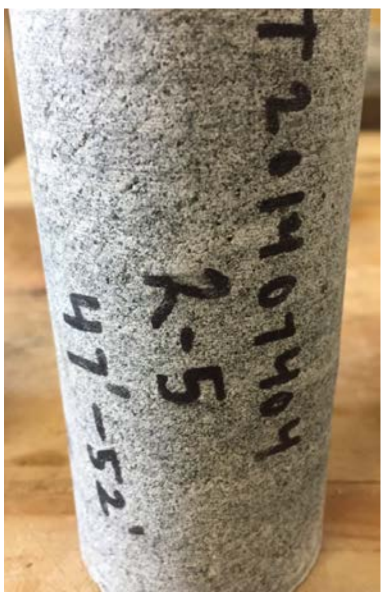
Respectfully Submitted,
Raju Acharya, PhD, PE
Geotechnical Engineer

5226 Indian River Road • Suite 103 • Virginia Beach, VA 23464 • *phone 757.306.1040* • *fax 757.306.1042*
www.etsva.com

REPORT OF UNIAXIAL COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN
CLIENT: Walton Corporation
PROJECT NAME: Task 131 T201407404 Wilmington Viaduct Widening
ETS Job NO.: ETS-17T219-4
MATERIAL DESCRIPTION: Grey and White, Granite
LOAD DIRECTION: Uniaxial
This Specimen was not prepared in accordance with ASTM D4543
Engineer: Raju Acharya, PhD, PE

COMPRESSION TEST RESULTS (ASTM D7012) - Method "C"

LABORATORY NUMBER	SPECIMEN IDENTIFICATION OR SET NO.	DEPTH (FT.)	DATE OF TEST	SPECIMEN DIAMETER (IN.)	SPECIMEN HEIGHT (IN.)	SPECIMEN AREA (SQ. IN.)	TOTAL LOAD (LBS.)	COMPRESSIVE STRENGTH (PSI)	AMBIENT TEMPERATURE AT TEST (F°)	BREAK TYPE	TIME TO FAILURE (MIN.)
Task 131	R-5	47-52	7/26/2017	2.0	4.0	3.1416	79,630	25,346	80	3	3.0



Respectfully Submitted,
Charlie T. Nabhan, PE
Principal Geotechnical Engineer

Respectfully Submitted,
Raju Acharya, PhD, PE
Geotechnical Engineer

5226 Indian River Road • Suite 103 • Virginia Beach, VA 23464 • *phone 757.306.1040* • *fax 757.306.1042*
www.etsva.com

REPORT OF UNIAXIAL COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN

CLIENT: Walton Corporation

PROJECT NAME: Task 131 T201407404 Wilmington Viaduct Widening

ETS Job NO.: ETS-17T219-5

MATERIAL DESCRIPTION: Grey and White, Granite

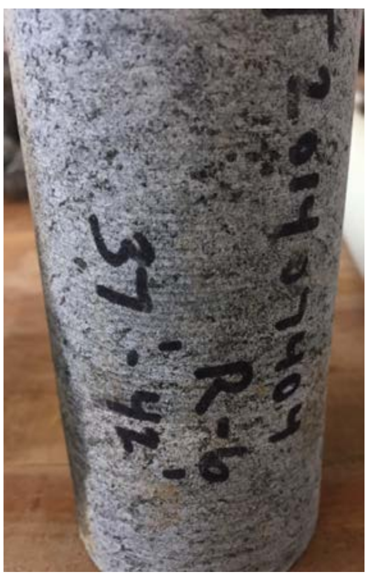
LOAD DIRECTION: Uniaxial

This Specimen was not prepared in accordance with ASTM D4543

Engineer: Raju Acharya, PhD, PE

COMPRESSION TEST RESULTS (ASTM D7012) - Method "C"

LABORATORY NUMBER	SPECIMEN IDENTIFICATION OR SET NO.	DEPTH (FT.)	DATE OF TEST	SPECIMEN DIAMETER (IN.)	SPECIMEN HEIGHT (IN.)	SPECIMEN AREA (SQ. IN.)	TOTAL LOAD (LBS.)	COMPRESSIVE STRENGTH (PSI)	AMBIENT TEMPERATURE AT TEST (F°)	BREAK TYPE	TIME TO FAILURE (MIN.)
Task 131	R-6	37-42	7/26/2017	2.0	4.0	3.1416	49,790	15,848	80	3	2.0



Respectfully Submitted,
Charlie T. Nabhan, PE
Principal Geotechnical Engineer

Respectfully Submitted,
Raju Acharya, PhD, PE
Geotechnical Engineer

ENGINEERING & TESTING SERVICES, INC.

August 29, 2017

To: Walton Corporation
P.O. Box 1097
Newark, Delaware 19715

Attn: Mr. Randy Ferguson

Re: **Reports of Uniaxial Compressive Strength of Intact Rock Core Specimen**
Delaware Department of Transportation
ETS Report No.: ETS-17T219-7
Task 131, T201407404, Wilmington Viaduct Widening

Dear Mr. Ferguson:

The purpose of this report is to submit the results of the compression tests conducted on two intact rock core specimens, designated as R-9 and R-10, at Engineering & Testing Services, Inc. (ETS) AASHTO accredited laboratory in Virginia Beach, Virginia. The rock cores were delivered to our laboratory by Walton Corporation personnel on August 21, 2017. The rock cores were prepared, measured, and tested in general accordance with ASTM Standard D7012, Method "C". All laboratory services performed by our personnel were supervised by a Professional Engineer from ETS. The reader is referred to the attached reports of Uniaxial Compressive Strength of Intact Rock Core Specimen for further details.

We appreciate the opportunity to be of service to you on this project. Should you have any questions regarding this letter, do not hesitate to contact our office at 757-306-1040.

Respectfully Submitted,
Engineering & Testing Services, Inc.

Charlie T. Nabhan, PE
Principal Geotechnical Engineer
VA License No.: 25133

Raju Acharya, PhD, PE
Geotechnical Engineer

5226 Indian River Road • Suite 103 • Virginia Beach, VA 23464 • *phone 757.306.1040* • *fax 757.306.1042*

REPORT OF UNIAXIAL COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN

CLIENT: Walton Corporation

PROJECT NAME: Task 131 T201407404 Wilmington Viaduct Widening

ETS Job NO.: ETS-17T219

MATERIAL DESCRIPTION: Grey and White, Granite

LOAD DIRECTION: Uniaxial

This Specimen was not prepared in accordance with ASTM D4543

Engineer: Raju Acharya, PhD, PE

COMPRESSION TEST RESULTS (ASTM D7012) - Method “C”

LABORATORY NUMBER	SPECIMEN IDENTIFICATION OR SET NO.	DEPTH (FT.)	DATE OF TEST	SPECIMEN DIAMETER (IN.)	SPECIMEN HEIGHT (IN.)	SPECIMEN AREA (SQ. IN.)	TOTAL LOAD (LBS.)	COMPRESSIVE STRENGTH (PSI)	AMBIENT TEMPERATURE AT TEST (F°)	BREAK TYPE	TIME TO FAILURE (MIN.)
Task 131	R-9	42-47	8/29/2017	2.0	4.0	3.1416	70,360	22,396	76	3	3.0

Respectfully Submitted,
Charlie T. Nabhan, PE
Principal Geotechnical Engineer

Respectfully Submitted,
Raju Acharya, PhD, PE
Geotechnical Engineer

REPORT OF UNIAXIAL COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMEN

CLIENT: Walton Corporation

PROJECT NAME: Task 131 T201407404 Wilmington Viaduct Widening

ETS Job NO.: ETS-17T219

MATERIAL DESCRIPTION: Grey and White, Granite

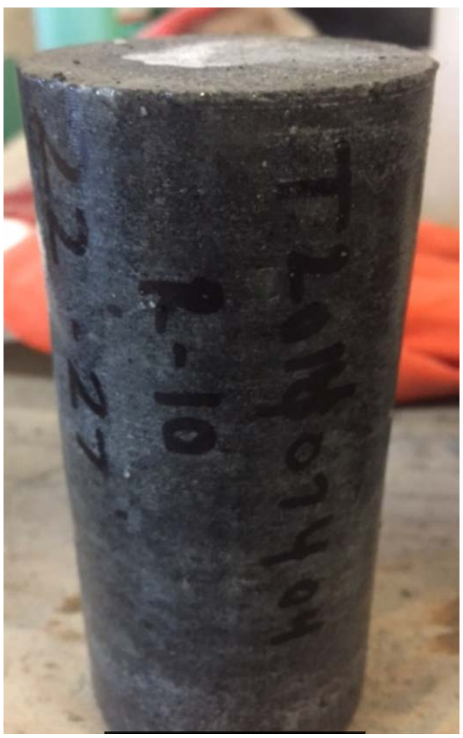
LOAD DIRECTION: Uniaxial

This Specimen was not prepared in accordance with ASTM D4543

Engineer: Raju Acharya, PhD, PE

COMPRESSION TEST RESULTS (ASTM D7012) - Method "C"

LABORATORY NUMBER	SPECIMEN IDENTIFICATION OR SET NO.	DEPTH (FT.)	DATE OF TEST	SPECIMEN DIAMETER (IN.)	SPECIMEN HEIGHT (IN.)	SPECIMEN AREA (SQ. IN.)	TOTAL LOAD (LBS.)	COMPRESSIVE STRENGTH (PSI)	AMBIENT TEMPERATURE AT TEST (F°)	BREAK TYPE	TIME TO FAILURE (MIN.)
Task 131	R-10	22-27	8/29/2017	2.0	4.0	3.1416	45,450	14,467	76	3	2.0



Respectfully Submitted,
Charlie T. Nabhan, PE
Principal Geotechnical Engineer

Respectfully Submitted,
Raju Acharya, PhD, PE
Geotechnical Engineer

5226 Indian River Road • Suite 103 • Virginia Beach, VA 23464 • *phone 757.306.1040* • *fax 757.306.1042*
www.etsva.com

ENGINEERING & TESTING SERVICES, INC.

September 13, 2017

To: Walton Corporation
P.O. Box 1097
Newark, Delaware 19715

Attn: Mr. Randolph Ferguson

Re: **pH and Corrosion Test Results**
ETS Report No.: ETS-17T219-11
Task 131, T201407404 Wilmington Viaduct Widening

Dear Mr. Ferguson:

Engineering & Testing Services, Inc. (ETS) is pleased to submit the completed pH and corrosion test results for three soils samples delivered to our office by Walton Corporation personnel on August 21, 2017. The samples were labeled as shown in Table 1 and were part of Task 131, T 201407404 Wilmington Viaduct Widening. The pH and corrosion tests were performed as per AASHTO T-289 and AASHTO T-288 standard test procedures, respectively. The test results are summarized as shown below in Table 1.

Table 1. Summary of pH and Corrosion Test Results

Boring	Sample	Sample type	pH	Resistivity (ohms/cm)
R-9	S-5	Spoon	7.8	1490
R-10	S-3	Spoon	6.8	2300

We appreciate the opportunity to be of service to you on this project. Should you have any questions regarding this report, do not hesitate to call our office at 757-306-1040.

Respectfully Submitted,
Engineering & Testing Services, Inc.

Charlie T. Nabhan, PE
Principal Geotechnical Engineer
VA License : 25133

Raju Acharya, PhD, PE
Geotechnical Engineer

ENGINEERING & TESTING SERVICES, INC.

August 4, 2017

To: Walton Corporation
P.O. Box 1097
Newark, Delaware 19715

Attn: Mr. Randolph Ferguson

Re: **pH and Corrosion Test Results**
ETS Report No.: ETS-17T219-6
Task 131, T201407404 Wilmington Viaduct Widening

Dear Mr. Ferguson:

Engineering & Testing Services, Inc. (ETS) is pleased to submit the completed pH and corrosion test results for three soils samples delivered to our office by Walton Corporation personnel on July 19, 2017. The samples were labeled as shown in Table 1 and were part of Task 131, T 201407404 Wilmington Viaduct Widening. The pH and corrosion tests were performed as per AASHTO T-289 and AASHTO T-288 standard test procedures, respectively. The test results are summarized as shown below in Table 1.

Table 1. Summary of pH and Corrosion Test Results

Boring	Sample	Sample type	pH	Resistivity (µmhos/cm)
R-2	S-6	Spoon	6.83	2,045
R-4	S-6	Spoon	5.72	2,421
R-6	S-2	Spoon	5.78	1,650

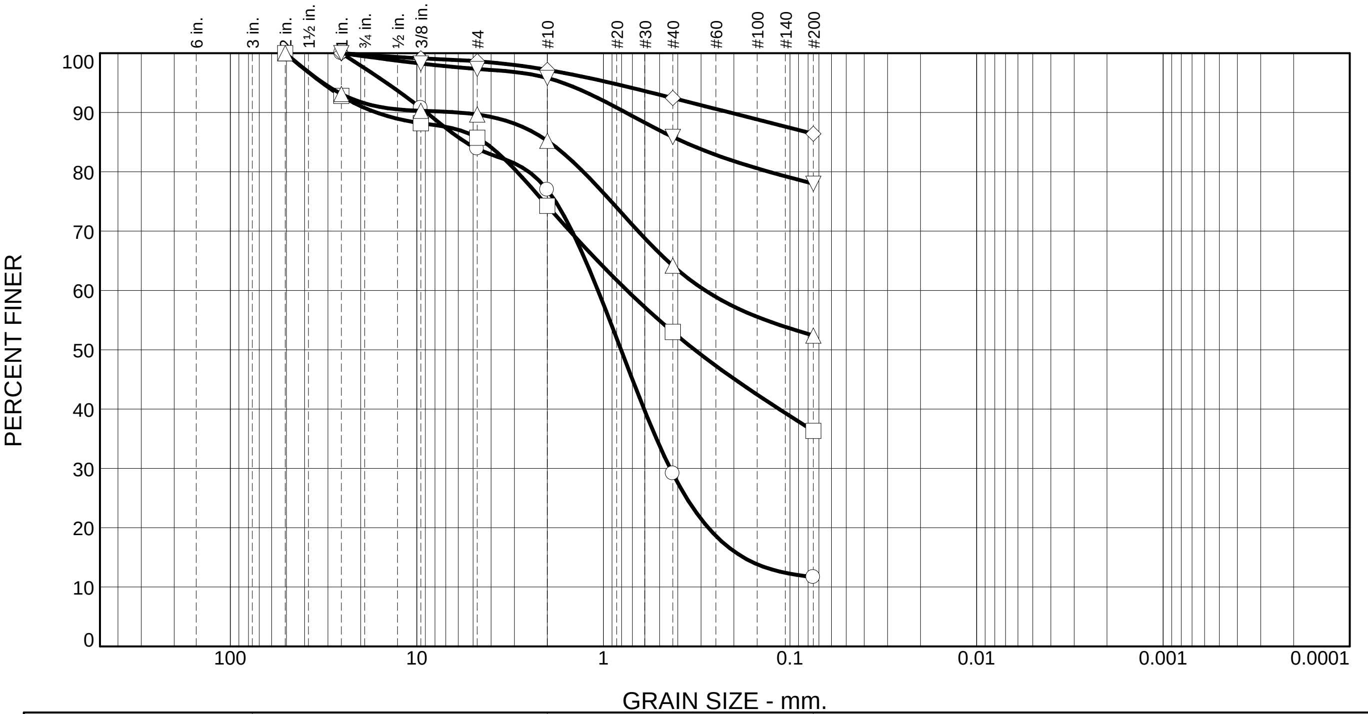
We appreciate the opportunity to be of service to you on this project. Should you have any questions regarding this report, do not hesitate to call our office at 757-306-1040.

Respectfully Submitted,
Engineering & Testing Services, Inc.

Charlie T. Nabhan, PE
Principal Geotechnical Engineer
VA License

Raju Acharya, PhD, PE
Geotechnical Engineer

Particle Size Distribution Report



	% +3"	% Gravel	% Sand Coarse	% Sand Fine	% Fines Silt	% Fines Clay
○	0.0	23.0	47.9	17.4	11.7	
□	0.0	25.7	21.3	16.7	36.3	
△	0.0	14.8	21.0	11.8	52.4	
◇	0.0	2.8	4.8	6.0	86.4	
▽	0.0	4.2	9.9	7.9	78.0	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	NV	NP	5.4797	1.0700	0.8050	0.4388	0.1778			
□	NV	NP	4.3554	0.7470	0.3241					
△	29.4	18.2	1.9518	0.2845						
◇	30.7	20.4								
▽	24.7	20.3	0.3692							

Material Description	USCS	AASHTO
○ Moist brown fine gravelly coarse sand w/some fine sand and silt.	SP-SM	A-1-b
□ Moist stiff brown coarse sandy fine gravelly silt w/some fine sand.	SM	A-4(0)
△ Wet stiff brown coarse sandy clay w/some fine sand, fine gravel and silt.	CL	A-6(3)
◇ Wet stiff brown silty clay w/trace fine to coarse sand and fine gravel.	CL	A-6(9)
▽ Wet firm brown silt w/some clay, trace of coarse to fine sand.	CL-ML	A-4(2)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-1 **Depth:** 1.0 **Sample Number:** 1

□ **Source of Sample:** R-1 **Depth:** 2.0 **Sample Number:** 2

△ **Source of Sample:** R-1 **Depth:** 4.0 **Sample Number:** 3

◇ **Source of Sample:** R-1 **Depth:** 6.0 **Sample Number:** 4

▽ **Source of Sample:** R-1 **Depth:** 8.0 **Sample Number:** 5

Remarks:

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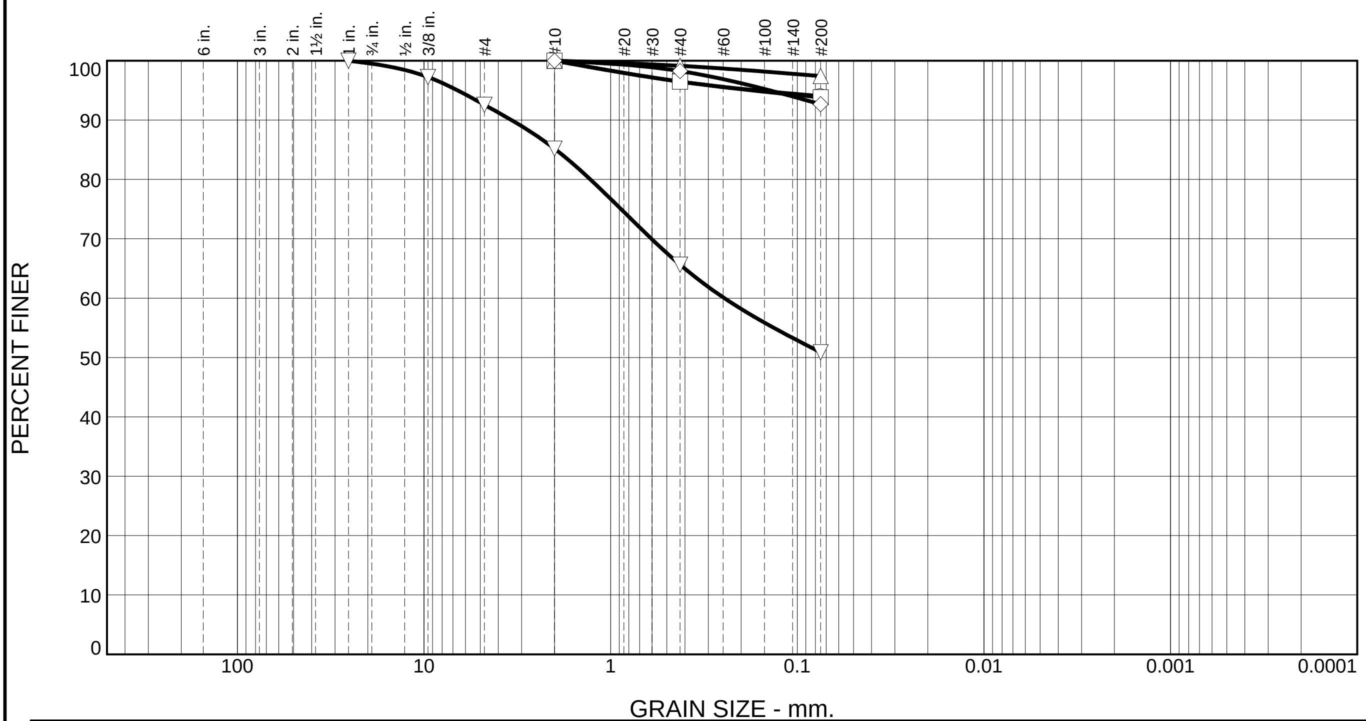
Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand Coarse	% Sand Fine	% Fines Silt	% Fines Clay
○	0.0	0.0	3.5	2.5	94.0	
□	0.0	0.0	3.5	2.7	93.8	
△	0.0	0.0	0.9	1.7	97.4	
◇	0.0	0.0	1.7	5.6	92.7	
▽	0.0	14.8	19.6	14.7	50.9	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	35.9	22.5								
□	35.6	26.5								
△	28.3	NP								
◇	33.5	24.3								
▽	30.0	22.5	1.9648	0.2458						

Material Description	USCS	AASHTO
○ Saturated firm brown silty clay w/trace coarse to fine sand.	CL	A-6(14)
□ Saturated firm brown clayey silt w/trace coarse to fine sand.	ML	A-4(11)
△ Saturated stiff brown silt w/trace fine to coarse sand.	ML	A-4(0)
◇ Saturated very stiff gray clayey silt w/trace fine to coarse sand.	CL	A-4(10)
▽ Saturated very stiff gray clayey silt w/some coarse to fine sand and fine gravel.	CL	A-4(2)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-1 **Depth:** 10.0 **Sample Number:** 6

□ **Source of Sample:** R-1 **Depth:** 12.0 **Sample Number:** 7

△ **Source of Sample:** R-1 **Depth:** 14.0 **Sample Number:** 8

◇ **Source of Sample:** R-1 **Depth:** 16.0 **Sample Number:** 9

▽ **Source of Sample:** R-1 **Depth:** 18.0 **Sample Number:** 10

Remarks:

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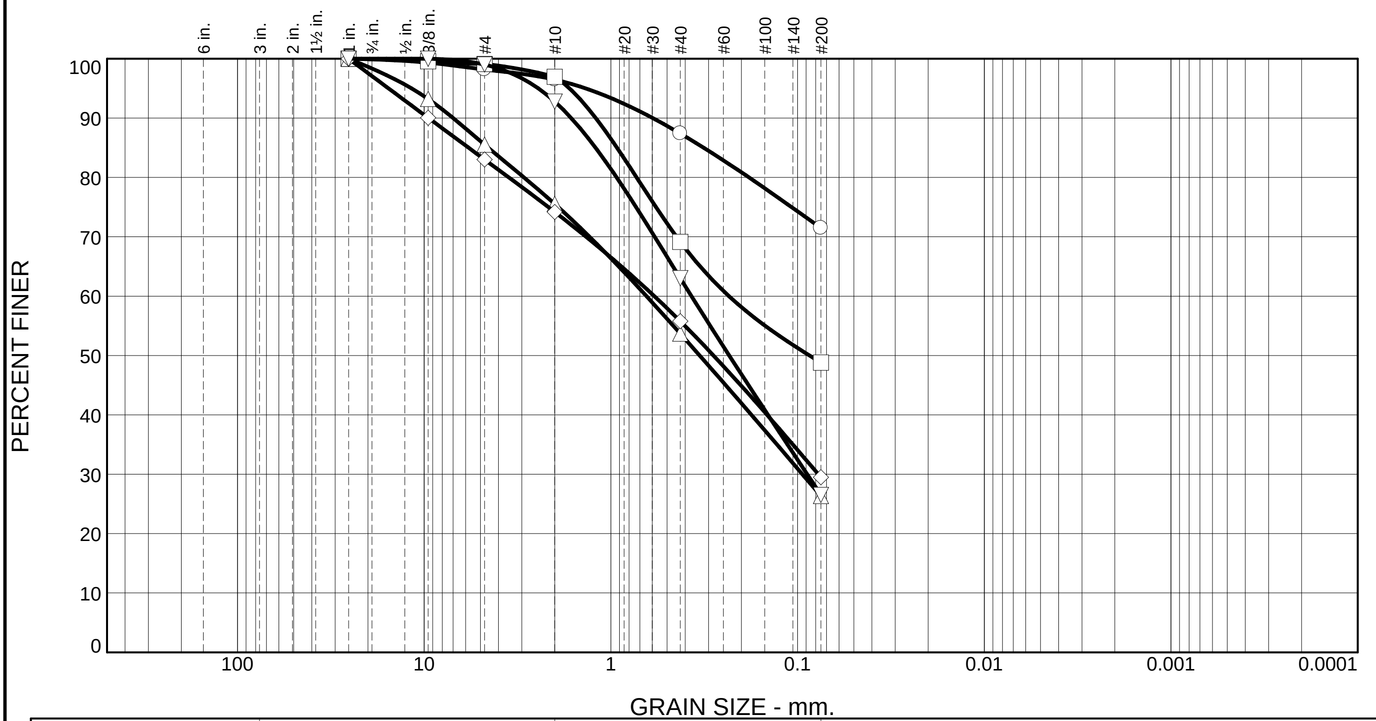
Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand		% Fines	
			Coarse	Fine	Silt	Clay
○	0.0	3.5	9.1	15.9	71.5	
□	0.0	3.0	27.8	20.4	48.8	
△	0.0	24.5	21.8	27.4	26.3	
◇	0.0	25.8	18.4	26.3	29.5	
▽	0.0	7.2	29.8	36.5	26.5	

☒	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	33.4	21.1	0.3184							
□	40.3	27.6	0.9273	0.2330	0.0863					
△	21.9	NP	4.5382	0.6449	0.3344	0.0943				
◇	33.2	NP	5.7796	0.5854	0.2821	0.0774				
▽	49.2	37.4	1.2087	0.3702	0.2324	0.0889				

Material Description	USCS	AASHTO
○ Saturated stiff gray silty clay w/some fine sand, trace of coarse sand and fine gravel.	CL	A-6(7)
□ Saturated stiff gray coarse sandy clay w/some fine sand and silt, trace of fine gravel.	SM	A-6(3)
△ Saturated medium dense gray silty fine to coarse sand and fine gravel.	SM	A-2-4(0)
◇ Saturated very dense gray silty fine sand and fine gravel w/some coarse sand.	SM	A-2-4(0)
▽ Saturated very stiff gray clayey fine to coarse sand w/some silt, trace of fine gravel.	SM	A-2-7(0)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-1 **Depth:** 24.0 **Sample Number:** 11

□ **Source of Sample:** R-1 **Depth:** 29.0 **Sample Number:** 12

△ **Source of Sample:** R-1 **Depth:** 34.0 **Sample Number:** 13

◇ **Source of Sample:** R-1 **Depth:** 39.0 **Sample Number:** 14

▽ **Source of Sample:** R-1 **Depth:** 47.5 **Sample Number:** 16

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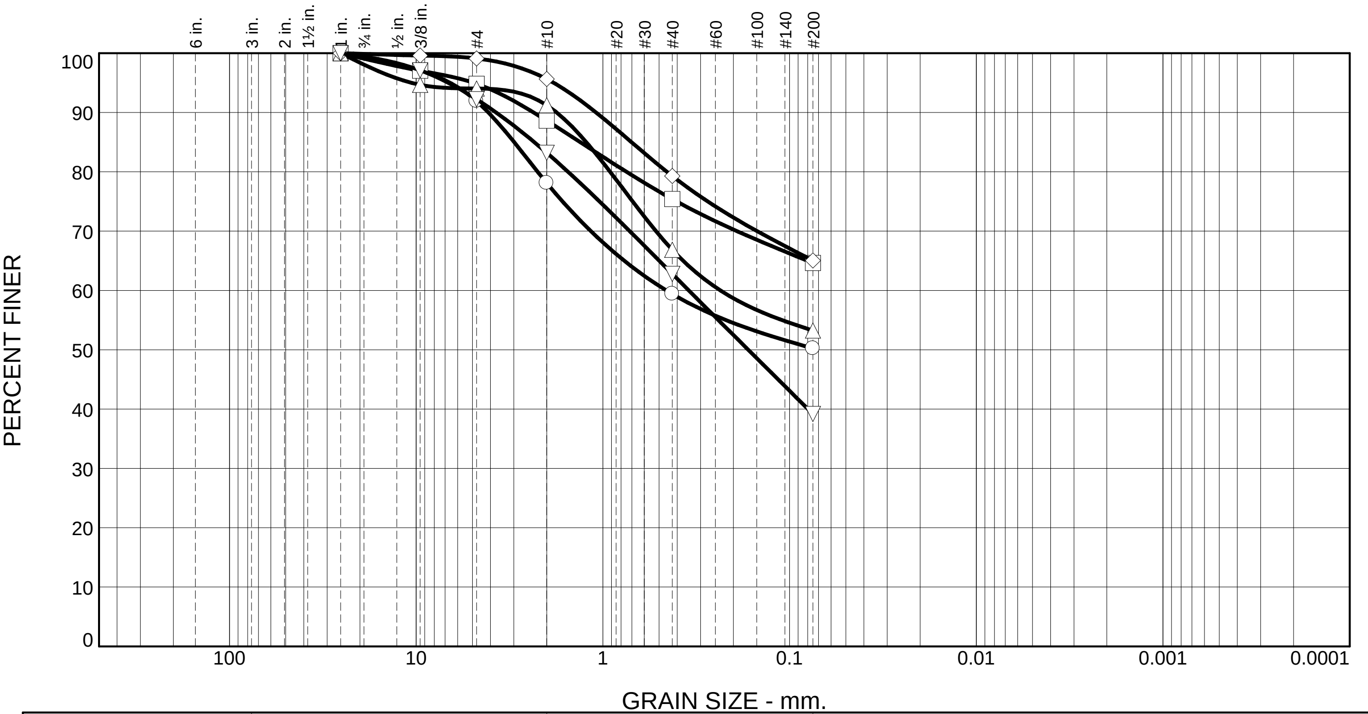
Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand Coarse	% Sand Fine	% Fines Silt	% Fines Clay
○	0.0	21.9	18.7	9.2	50.2	
□	0.0	11.4	13.2	10.8	64.6	
△	0.0	8.8	24.4	13.6	53.2	
◇	0.0	4.4	16.3	14.2	65.1	
▽	0.0	16.8	20.4	23.6	39.2	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	23.8	21.0	2.9886	0.4571						
□	29.0	20.1	1.3224							
△	29.8	20.0	1.2282	0.2347						
◇	25.0	21.2	0.7026							
▽	19.6	NP	2.3328	0.3467	0.1666					

Material Description	USCS	AASHTO
○ Moist hard brown fine gravelly silt w/some coarse sand, trace of fine sand and clay.	ML	A-4(0)
□ Moist very stiff gray clayey silt w/some coarse to fine sand and fine gravel.	CL	A-4(4)
△ Moist very stiff brown clayey coarse sandy silt w/some fine sand, trace of fine gravel.	CL	A-4(3)
◇ Moist stiff gray silt w/some coarse to fine sand and clay, trace of fine gravel.	CL-ML	A-4(1)
▽ Moist very stiff brown fine sandy silt w/some coarse sand and fine gravel.	SM	A-4(0)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-2 **Depth:** 2.0 **Sample Number:** 2

□ **Source of Sample:** R-2 **Depth:** 4.0 **Sample Number:** 3

△ **Source of Sample:** R-2 **Depth:** 6.0 **Sample Number:** 4

◇ **Source of Sample:** R-2 **Depth:** 8.0 **Sample Number:** 5

▽ **Source of Sample:** R-2 **Depth:** 10.0 **Sample Number:** 6

Remarks:

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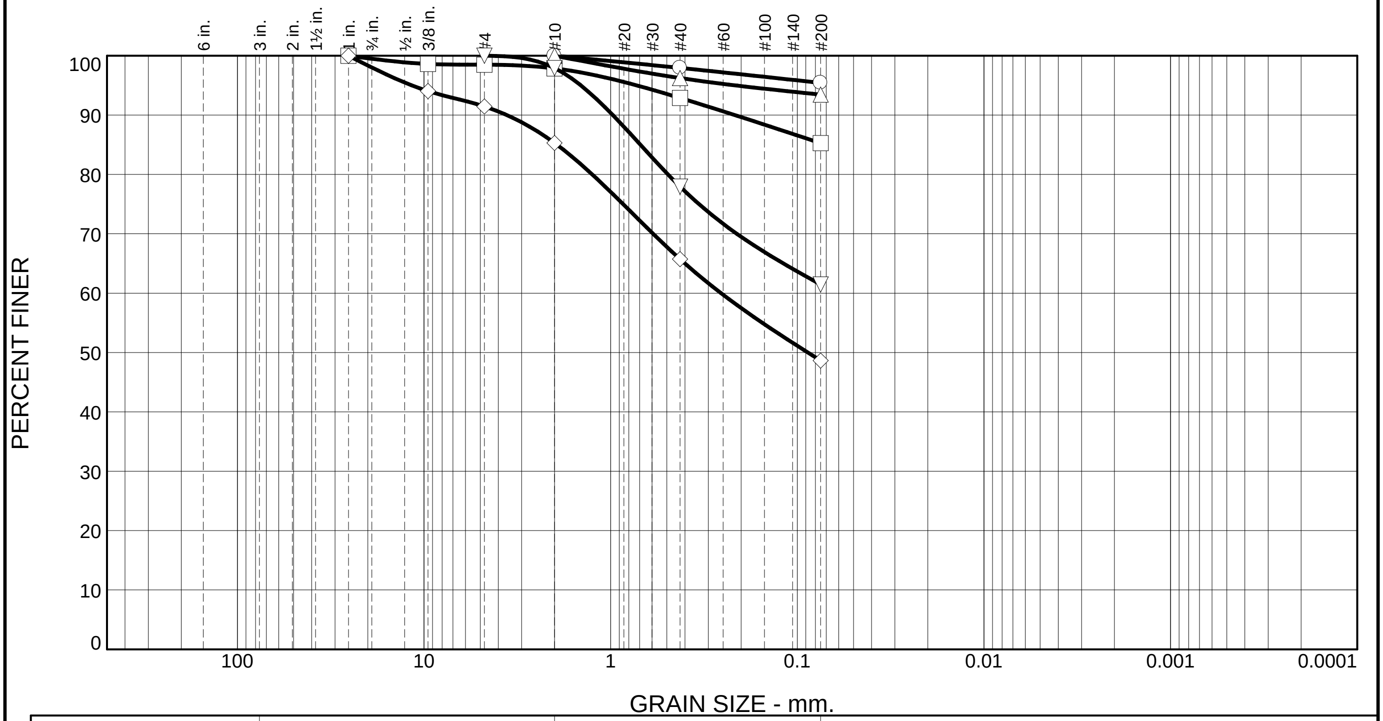
Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand Coarse	% Sand Fine	% Fines Silt	% Fines Clay
○	0.0	0.0	2.1	2.5	95.4	
□	0.0	2.1	5.0	7.6	85.3	
△	0.0	0.0	3.8	2.8	93.4	
◇	0.0	14.7	19.6	17.1	48.6	
▽	0.0	2.1	19.9	16.5	61.5	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	37.6	26.2								
□	32.0	24.0								
△	30.4	24.7								
◇	28.8	18.6	1.9407	0.2563	0.0878					
▽	36.1	26.2	0.6925							

Material Description	USCS	AASHTO
○ Saturated firm gray silty clay w/trace fine to coarse sand.	ML	A-6(13)
□ Saturated stiff gray clayey silt w/trace fine to coarse sand and fine gravel.	ML	A-4(7)
△ Saturated firm gray clayey silt w/trace coarse to fine sand.	ML	A-4(5)
◇ Saturated firm gray clayey silt w/some coarse to fine sand and fine gravel.	SC	A-4(2)
▽ Saturated very stiff gray clayey silt w/some coarse to fine sand, trace of fine gravel.	ML	A-4(5)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-2 **Depth:** 14.0 **Sample Number:** 8

□ **Source of Sample:** R-2 **Depth:** 16.0 **Sample Number:** 9

△ **Source of Sample:** R-2 **Depth:** 18.0 **Sample Number:** 10

◇ **Source of Sample:** R-2 **Depth:** 24.0 **Sample Number:** 11

▽ **Source of Sample:** R-2 **Depth:** 29.0 **Sample Number:** 12

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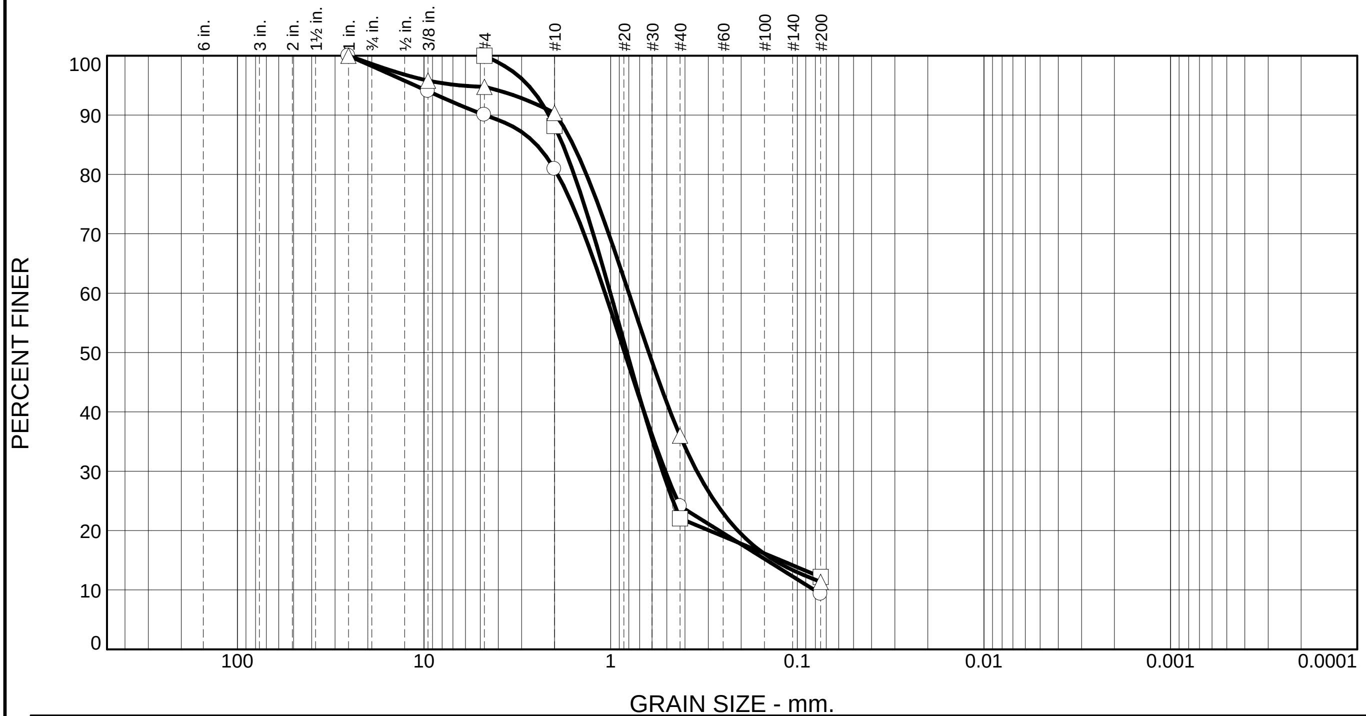
Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand Coarse	% Sand Fine	% Fines Silt	% Fines Clay
○	0.0	19.1	56.8	14.8	9.3	
□	0.0	11.8	66.1	9.9	12.2	
△	0.0	9.6	54.4	24.6	11.4	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	NV	NP	2.4956	1.0717	0.8428	0.5094	0.1459	0.0812	2.98	13.21
□	NV	NP	1.8043	1.0037	0.8178	0.5279	0.1228			
△	NV	NP	1.5881	0.7990	0.6245	0.3451	0.1330			

Material Description	USCS	AASHTO
○ Saturated loose gray coarse sand w/some fine sand and fine gravel, trace of silt.	SW-SM	A-1-b
□ Saturated medium dense gray coarse sand w/some fine gravel and silt, trace of fine sand.	SM	A-1-b
△ Saturated very dense gray coarse to fine sand w/some silt, trace of fine gravel.	SP-SM	A-1-b

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-2 **Depth:** 34.0 **Sample Number:** 13

□ **Source of Sample:** R-2 **Depth:** 39.0 **Sample Number:** 14

△ **Source of Sample:** R-2 **Depth:** 44.0 **Sample Number:** 15

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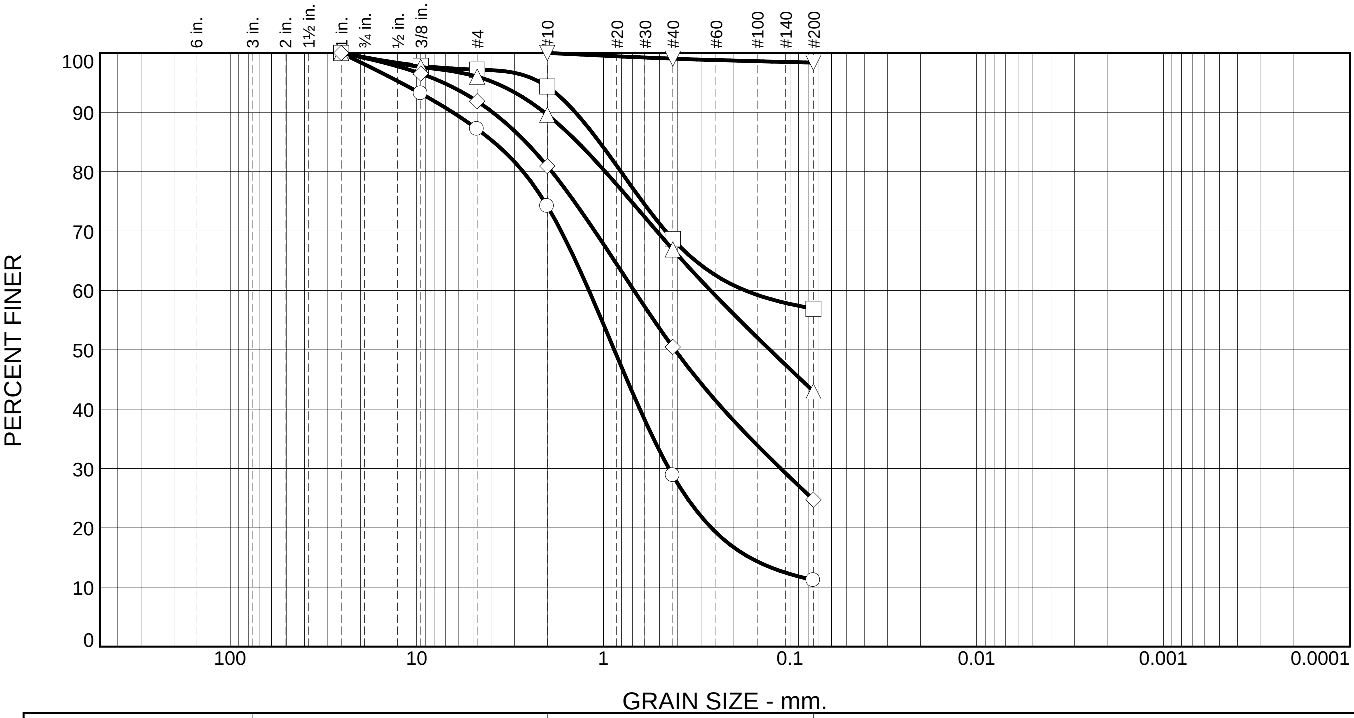
Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand Coarse	% Sand Fine	% Fines Silt	% Fines Clay
○	0.0	25.8	45.4	17.6	11.2	
□	0.0	5.7	25.7	11.7	56.9	
△	0.0	10.4	22.7	23.9	43.0	
◇	0.0	19.1	30.4	25.8	24.7	
▽	0.0	0.0	1.0	0.6	98.4	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	NV	NP	3.8701	1.1964	0.8752	0.4459	0.1647			
□	29.6	21.7	1.0507	0.1742						
△	21.8	17.8	1.3787	0.2669	0.1284					
◇	16.0	NP	2.6087	0.6863	0.4143	0.1123				
▽	39.4	28.4								

Material Description	USCS	AASHTO
○ Moist brown fine gravelly coarse sand w/some fine sand and silt.	SP-SM	A-1-b
□ Moist very stiff brown clayey coarse sandy silt w/some fine sand, trace of fine gravel.	CL	A-4(2)
△ Moist stiff brown fine to coarse sandy silt w/some clay, trace of fine gravel.	SC-SM	A-4(0)
◇ Moist medium dense brown silty coarse to fine sand w/some fine gravel.	SM	A-1-b
▽ Saturated very stiff brown silty clay w/trace fine to coarse sand.	ML	A-6(13)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-3 **Depth:** 0.5 **Sample Number:** 1

□ **Source of Sample:** R-3 **Depth:** 4.0 **Sample Number:** 3

△ **Source of Sample:** R-3 **Depth:** 6.0 **Sample Number:** 4

◇ **Source of Sample:** R-3 **Depth:** 8.0 **Sample Number:** 5

▽ **Source of Sample:** R-3 **Depth:** 10.0 **Sample Number:** 6

Remarks:

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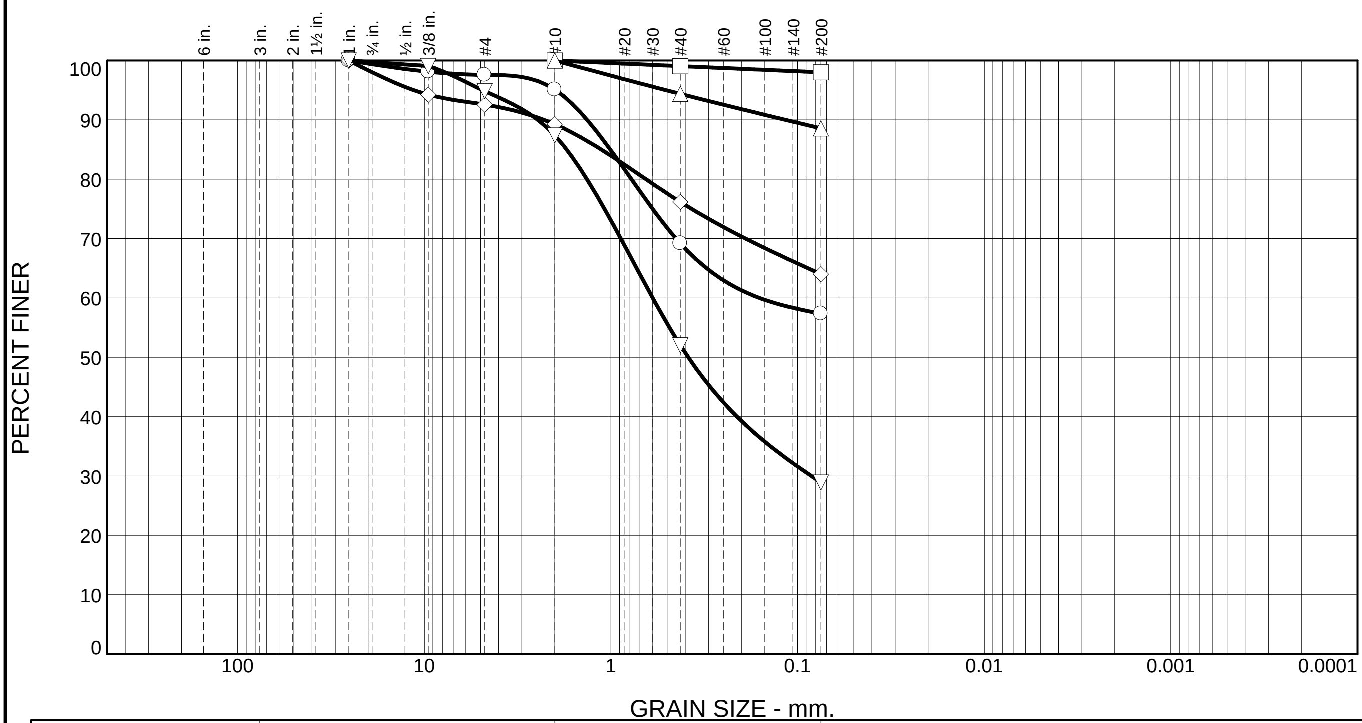
Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand Coarse	% Sand Fine	% Fines Silt	% Fines Clay
○	0.0	4.9	25.9	11.9	57.3	
□	0.0	0.0	0.9	1.1	98.0	
△	0.0	0.0	5.6	5.8	88.6	
◇	0.0	10.7	13.1	12.2	64.0	
▽	0.0	12.6	35.4	23.1	28.9	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	25.2	19.5	1.0079	0.1605						
□	39.2	28.9								
△	32.0	23.3								
◇	34.7	22.2	1.1270							
▽	24.0	16.4	1.7272	0.5974	0.3854	0.0844				

Material Description	USCS	AASHTO
○ Saturated soft gray clayey coarse sandy silt w/some fine sand, trace of fine gravel.	CL-ML	A-4(1)
□ Saturated firm gray clayey silt w/trace fine to coarse sand.	ML	A-4(12)
△ Saturated stiff gray clayey silt w/trace fine to coarse sand.	CL	A-4(8)
◇ Saturated firm gray silty clay w/some coarse to fine sand and fine gravel.	CL	A-6(7)
▽ Saturated medium dense gray clayey coarse to fine sand w/some fine gravel and silt.	SC	A-2-4(0)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-3 **Depth:** 12.0 **Sample Number:** 7

□ **Source of Sample:** R-3 **Depth:** 14.0 **Sample Number:** 8

△ **Source of Sample:** R-3 **Depth:** 16.0 **Sample Number:** 9

◇ **Source of Sample:** R-3 **Depth:** 24.0 **Sample Number:** 11

▽ **Source of Sample:** R-3 **Depth:** 29.0 **Sample Number:** 12

Remarks:

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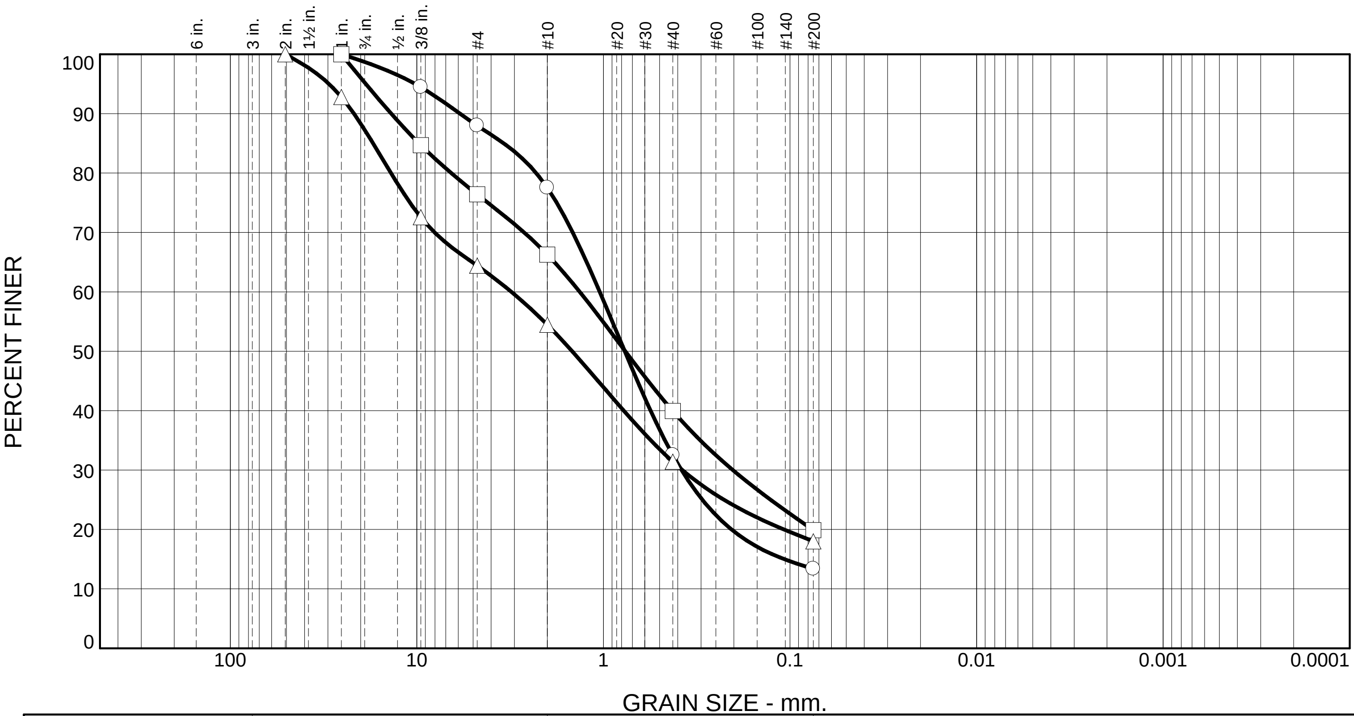
Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand Coarse	% Sand Fine	% Fines Silt	% Fines Clay
○	0.0	22.5	45.0	19.1	13.4	
□	0.0	33.7	26.4	20.0	19.9	
△	0.0	45.5	23.1	13.4	18.0	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	19.8	NP	3.4233	1.0469	0.7688	0.3815	0.1073			
□	37.7	27.4	9.7414	1.3403	0.7635	0.2026				
△	NV	NP	17.1784	3.1155	1.4727	0.3778				

Material Description	USCS	AASHTO
○ Saturated medium dense brown fine gravelly coarse sand w/some fine sand and silt.	SM	A-1-b
□ Saturated very dense gray clayey coarse sand and fine gravel w/some fine sand and silt.	SM	A-2-6(0)
△ Moist loose brown coarse sandy fine gravel w/some fine sand and silt.	SM	A-1-b

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-3 **Depth:** 34.0 **Sample Number:** 13

□ **Source of Sample:** R-3 **Depth:** 39.0 **Sample Number:** 14

△ **Source of Sample:** R-3 **Depth:** 2.0 **Sample Number:** 24

Remarks:

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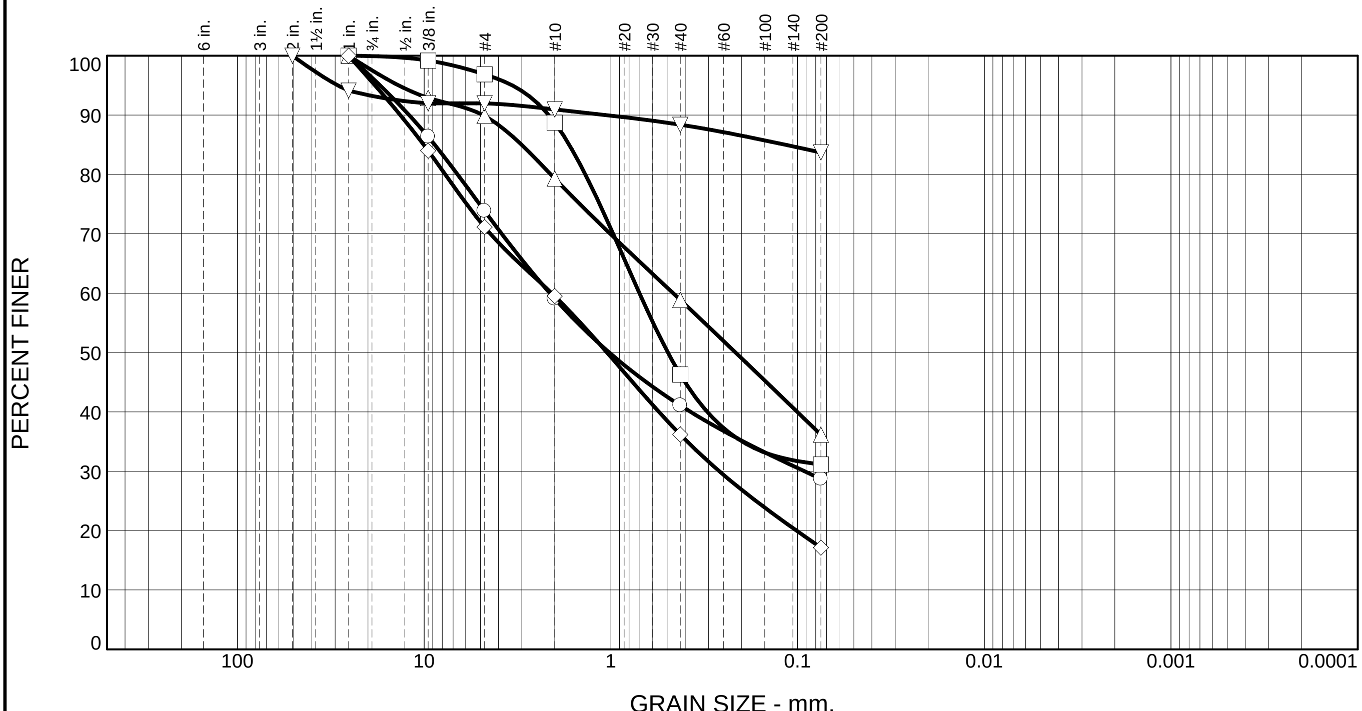
Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand Coarse	% Sand Fine	% Fines Silt	% Fines Clay
○	0.0	40.9	18.0	12.4	28.7	
□	0.0	11.3	42.4	15.2	31.1	
△	0.0	20.7	20.4	22.8	36.1	
◇	0.0	40.5	23.3	19.1	17.1	
▽	0.0	9.1	2.5	4.7	83.7	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	23.7	17.8	8.7855	2.1255	1.0202	0.0919				
□	22.5	16.8	1.6725	0.7034	0.4950					
△	20.0	NP	3.0093	0.4644	0.2149					
◇	16.7	NP	10.0608	2.0769	1.0495	0.2625				
▽	29.5	23.1	0.1163							

	Material Description	USCS	AASHTO
○	Moist medium dense brown clayey fine gravel w/some coarse to fine sand.	SC-SM	A-2-4(0)
□	Moist dense brown clayey coarse sand w/ some fine sand, fine gravel and silt.	SC-SM	A-2-4(0)
△	Moist stiff brown fine sandy fine gravelly silt w/some coarse sand.	SM	A-4(0)
◇	Moist medium dense brown coarse sandy fine gravel w/some fine sand and silt.	SM	A-1-b
▽	Wet firm brown clayey silt w/trace fine to coarse sand and fine gravel.	CL-ML	A-4(5)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-4 **Depth:** 4.0 **Sample Number:** 2

□ **Source of Sample:** R-4 **Depth:** 6.0 **Sample Number:** 3

△ **Source of Sample:** R-4 **Depth:** 8.0 **Sample Number:** 4

◇ **Source of Sample:** R-4 **Depth:** 10.0 **Sample Number:** 5

▽ **Source of Sample:** R-4 **Depth:** 14.0 **Sample Number:** 7

Remarks:

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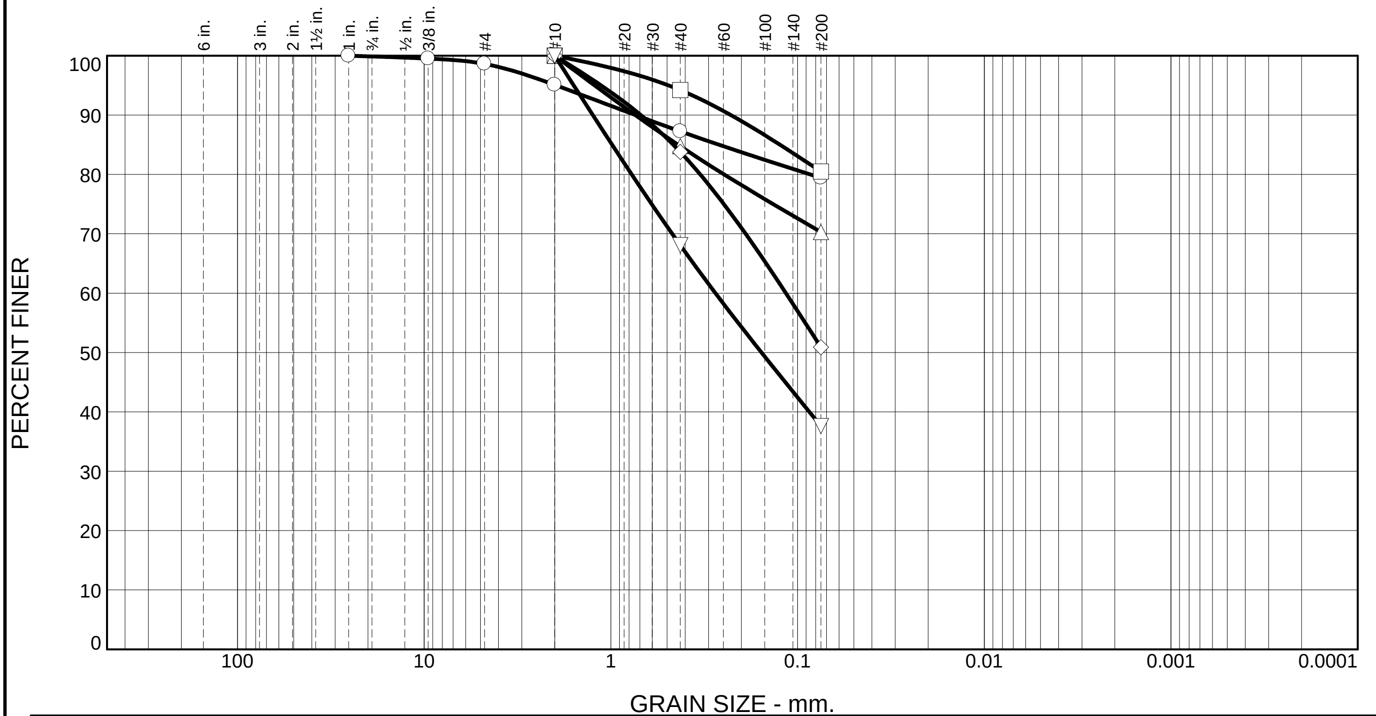
Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand Coarse	% Sand Fine	% Fines Silt	% Fines Clay
○	0.0	4.9	7.8	7.9	79.4	
□	0.0	0.0	5.8	13.7	80.5	
△	0.0	0.0	15.2	14.5	70.3	
◇	0.0	0.0	16.2	32.9	50.9	
▽	0.0	0.0	31.9	30.5	37.6	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	33.9	24.6	0.2638							
□	39.8	25.1	0.1243							
△	31.5	20.2	0.4367							
◇	58.3	44.0	0.4630	0.1155						
▽	52.8	40.4	0.9849	0.2754	0.1561					

Material Description	USCS	AASHTO
○ Wet stiff gray clayey silt w/trace fine to coarse sand and fine gravel.	ML	A-4(7)
□ Wet firm gray silty clay w/some fine sand, trace of coarse sand.	CL	A-6(12)
△ Wet hard gray silty clay w/some fine to coarse sand.	CL	A-6(7)
◇ Saturated very stiff brown fine sandy clay w/some coarse sand and silt.	MH	A-7-5(6)
▽ Saturated hard brown coarse to fine sandy clay w/some silt.	SM	A-7-5(1)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-4 **Depth:** 16.0 **Sample Number:** 8

□ **Source of Sample:** R-4 **Depth:** 18.0 **Sample Number:** 9

△ **Source of Sample:** R-4 **Depth:** 24.0 **Sample Number:** 10

◇ **Source of Sample:** R-4 **Depth:** 28.0 **Sample Number:** 11

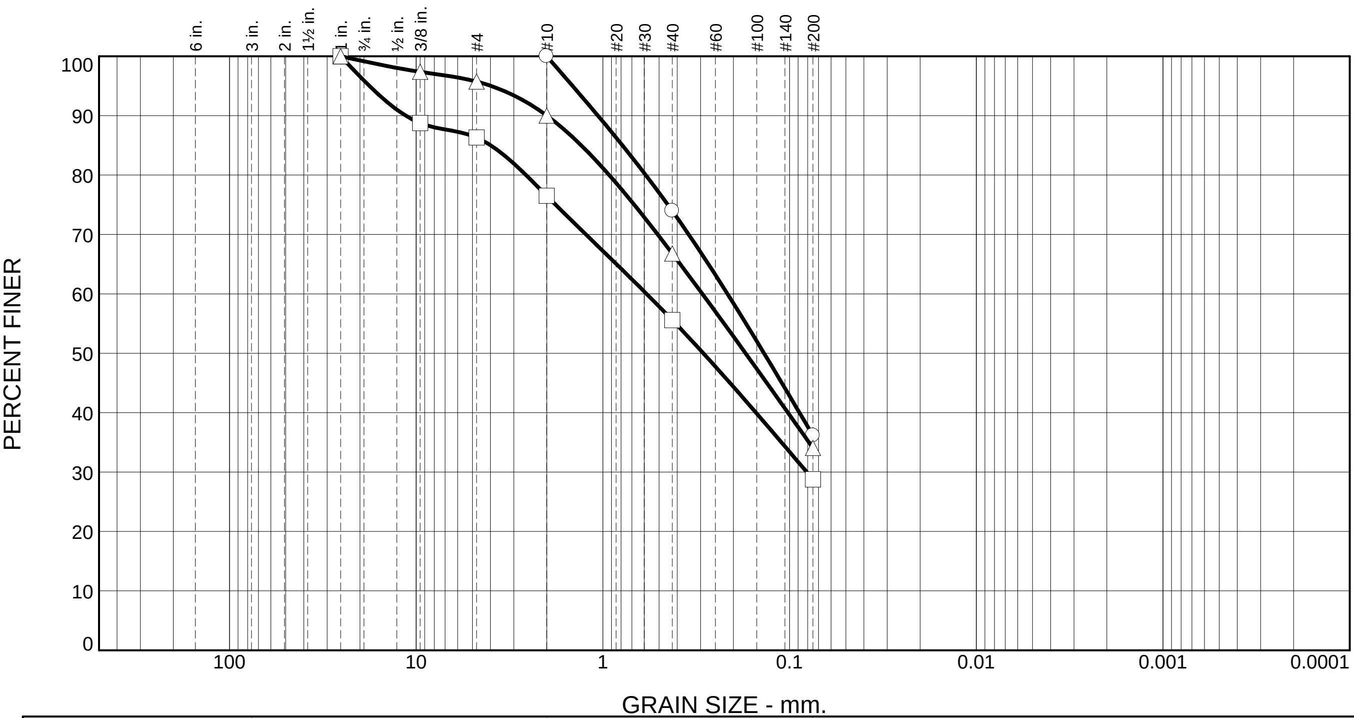
▽ **Source of Sample:** R-4 **Depth:** 34.0 **Sample Number:** 12

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand		% Fines	
			Coarse	Fine	Silt	Clay
○	0.0	0.0	26.0	37.8	36.2	
□	0.0	23.5	20.9	26.8	28.8	
△	0.0	10.0	23.2	32.7	34.1	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	35.7	NP	0.7860	0.2151	0.1371					
□	29.3	NP	3.9837	0.5835	0.2898	0.0810				
△	29.5	NP	1.3071	0.2933	0.1722					

Material Description	USCS	AASHTO
○ Saturated hard brown fine to coarse sandy silt.	SM	A-4(0)
□ Saturated very dense brown silty fine to coarse sand and fine gravel.	SM	A-2-4(0)
△ Saturated dense brown silty fine to coarse sand w/trace fine gravel.	SM	A-2-4(0)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-4 **Depth:** 38.0 **Sample Number:** 13

□ **Source of Sample:** R-4 **Depth:** 44.0 **Sample Number:** 14

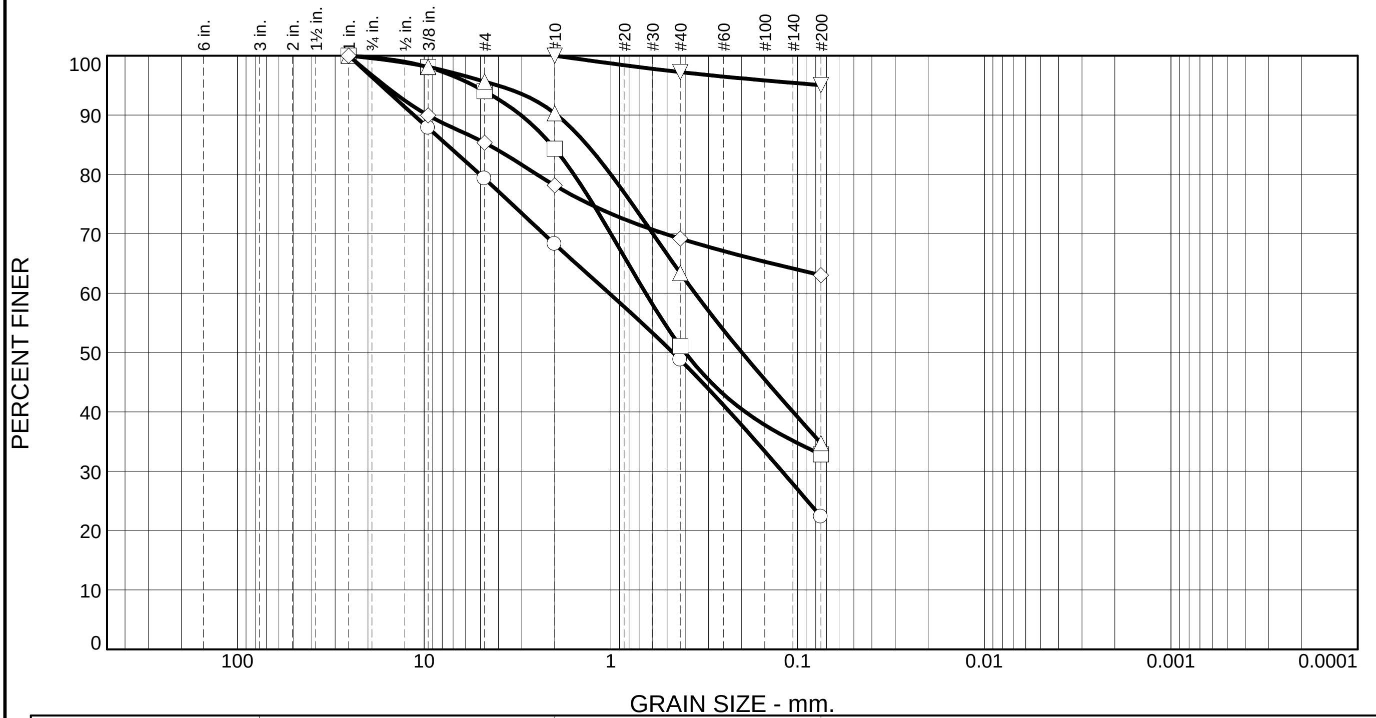
△ **Source of Sample:** R-4 **Depth:** 48.0 **Sample Number:** 15

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand		% Fines	
			Coarse	Fine	Silt	Clay
○	0.0	31.7	19.5	26.5	22.3	
□	0.0	15.7	33.2	18.3	32.8	
△	0.0	9.7	26.9	28.7	34.7	
◇	0.0	21.8	9.0	6.2	63.0	
▽	0.0	0.0	2.8	2.2	95.0	

╳	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	17.4	15.5	7.5507	1.0225	0.4656	0.1212				
□	22.4	17.6	2.0832	0.6510	0.4003					
△	19.1	NP	1.3428	0.3543	0.1989					
◇	23.2	16.8	4.5285							
▽	26.2	18.7								

Material Description	USCS	AASHTO
○ Moist dense brown silty fine sand and fine gravel w/some coarse sand.	SM	A-1-b
□ Wet medium dense brown silty coarse sand w/some fine sand and fine gravel.	SC-SM	A-2-4(0)
△ Wet medium dense brown silty fine to coarse sand w/trace fine gravel.	SM	A-2-4(0)
◇ Wet stiff brown clayey fine clayey silt w/trace coarse to fine sand.	CL-ML	A-4(1)
▽ Wet stiff gray clayey silt w/trace coarse to fine sand.	CL-ML	A-4(5)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-5 **Depth:** 2.0 **Sample Number:** 1

□ **Source of Sample:** R-5 **Depth:** 6.0 **Sample Number:** 3

△ **Source of Sample:** R-5 **Depth:** 8.0 **Sample Number:** 4

◇ **Source of Sample:** R-5 **Depth:** 10.0 **Sample Number:** 5

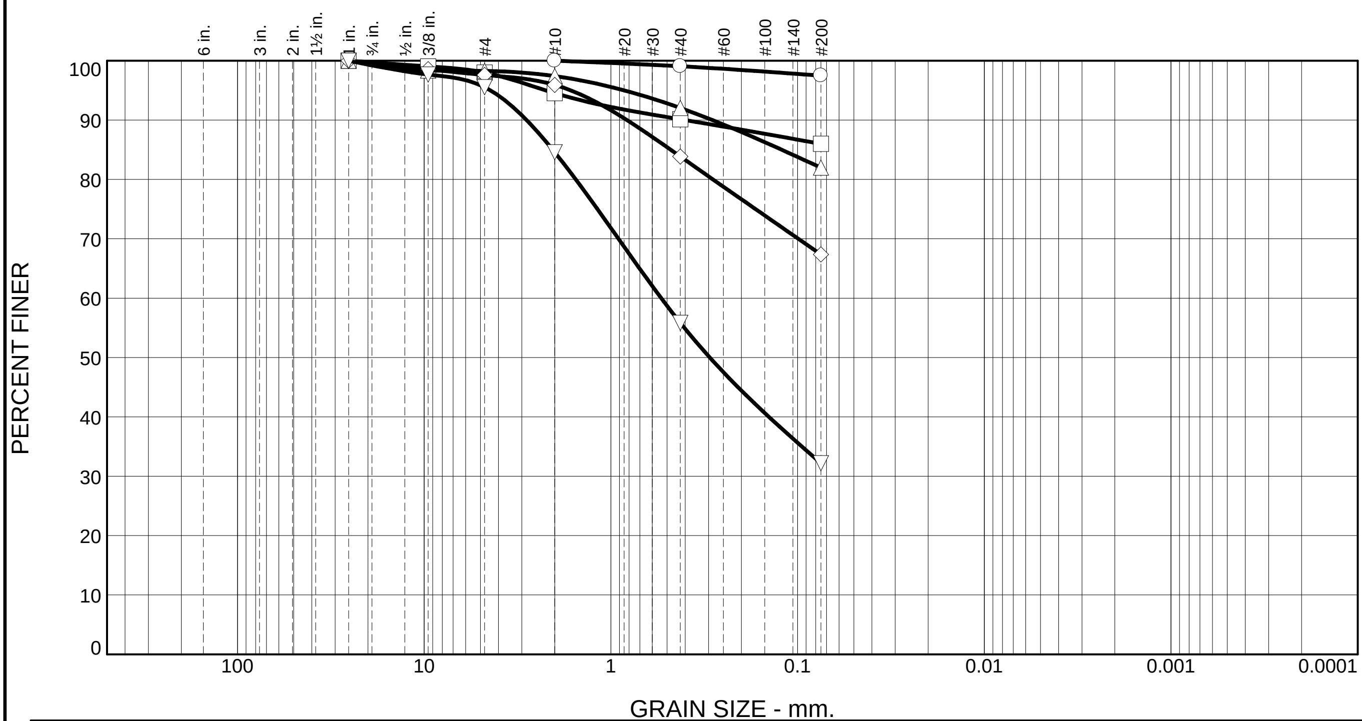
▽ **Source of Sample:** R-5 **Depth:** 12.0 **Sample Number:** 6

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand Coarse	% Sand Fine	% Fines Silt	% Fines Clay
○	0.0	0.0	0.9	1.6	97.5	
□	0.0	5.5	4.4	4.1	86.0	
△	0.0	2.6	5.4	10.0	82.0	
◇	0.0	4.1	12.0	16.5	67.4	
▽	0.0	15.4	28.7	23.7	32.2	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	24.1	20.7								
□	26.6	21.2								
△	28.3	21.3	0.1217							
◇	33.2	19.8	0.4782							
▽	26.7	15.2	2.0505	0.5374	0.2953					

Material Description	USCS	AASHTO
○ Wet stiff gray silt w/trace fine to coarse sand and clay.	ML	A-4(2)
□ Wet very stiff gray silt w/some clay, trace of fine to coarse sand and fine gravel.	CL-ML	A-4(4)
△ Wet stiff brown clayey silt w/trace fine to coarse sand and fine gravel.	CL-ML	A-4(5)
◇ Wet stiff brown silty clay w/some fine to coarse sand, trace of fine gravel.	CL	A-6(7)
▽ Wet medium dense gray clayey coarse to fine sand w/some fine gravel and silt.	SC	A-2-6(0)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-5 **Depth:** 14.0 **Sample Number:** 7

□ **Source of Sample:** R-5 **Depth:** 16.0 **Sample Number:** 8

△ **Source of Sample:** R-5 **Depth:** 18.0 **Sample Number:** 9

◇ **Source of Sample:** R-5 **Depth:** 24.0 **Sample Number:** 10

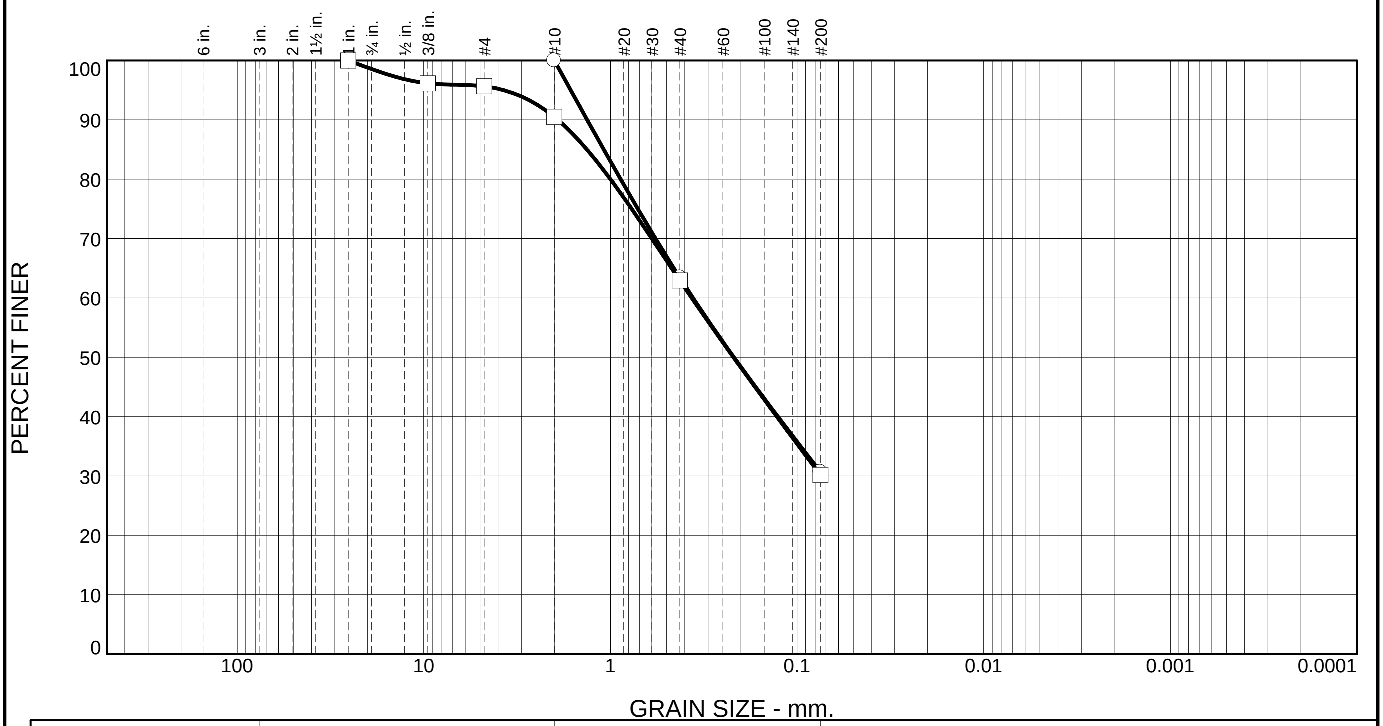
▽ **Source of Sample:** R-5 **Depth:** 28.0 **Sample Number:** 11

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand		% Fines	
			Coarse	Fine	Silt	Clay
○	0.0	0.0	36.6	32.7	30.7	
□	0.0	9.5	27.5	32.8	30.2	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	30.0	NP	1.0857	0.3610	0.2180					
□	32.9	NP	1.3379	0.3668	0.2192					

	Material Description	USCS	AASHTO
○	Saturated medium dense gray silty coarse to fine sand.	SM	A-2-4(0)
□	Saturated dense gray silty fine to coarse sand w/trace fine gravel.	SM	A-2-4(0)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-5 **Depth:** 34.0 **Sample Number:** 12

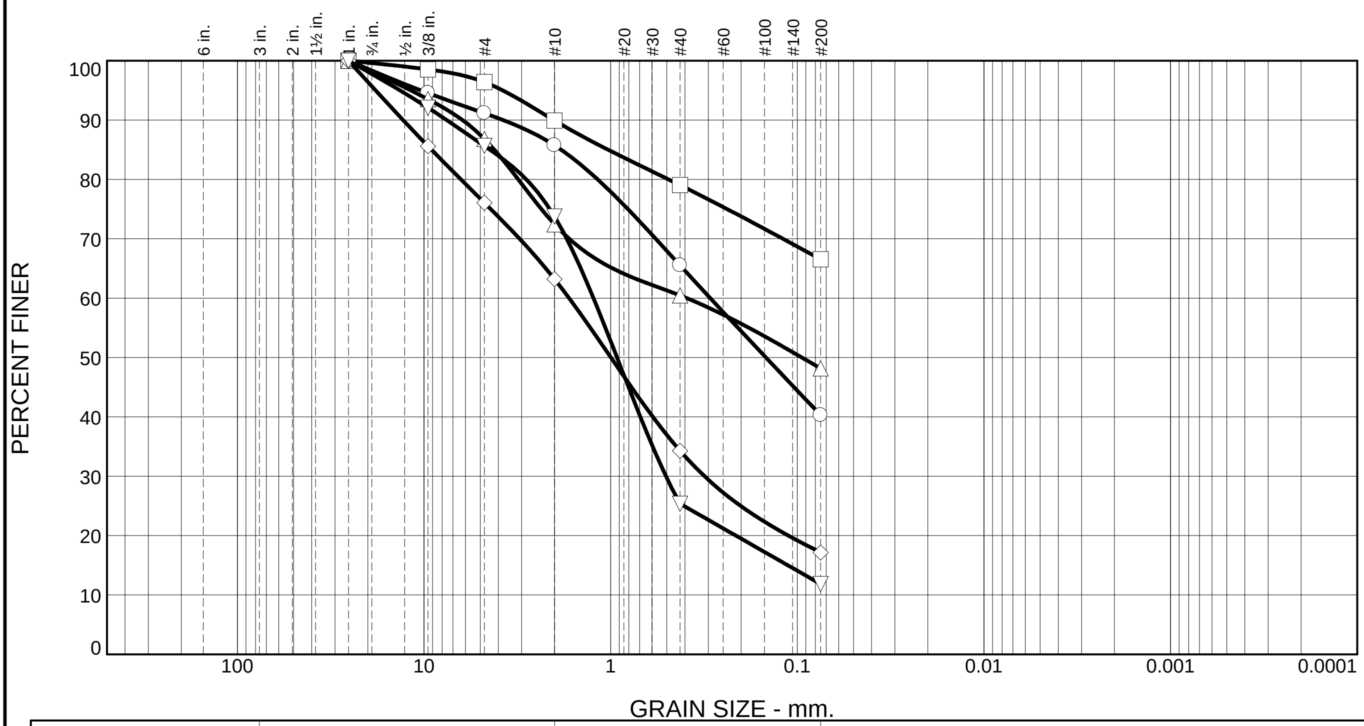
□ **Source of Sample:** R-5 **Depth:** 38.0 **Sample Number:** 13

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand Coarse	% Sand Fine	% Fines Silt	% Fines Clay
○	0.0	14.2	20.3	25.2	40.3	
□	0.0	10.1	10.8	12.5	66.6	
△	0.0	27.6	11.9	12.3	48.2	
◇	0.0	36.8	28.9	17.2	17.1	
▽	0.0	26.2	48.4	13.6	11.8	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	19.6	15.5	1.8425	0.2927	0.1473					
□	25.2	19.3	1.0320							
△	26.1	18.6	4.1976	0.3908	0.0945					
◇	NV	NP	9.1202	1.6705	0.9973	0.3141				
▽	NV	NP	4.4341	1.2365	0.9244	0.5054	0.1125			

Material Description	USCS	AASHTO
○ Moist brown fine sandy silt w/some coarse sand, clay and fine gravel.	SC-SM	A-4(0)
□ Moist very stiff gray clayey silt w/some fine to coarse sand, trace of fine gravel.	CL-ML	A-4(2)
△ Moist very stiff gray clayey fine gravelly silt w/some fine to coarse sand.	SC-SM	A-4(1)
◇ Moist dense brown coarse sand and fine gravel w/some fine sand and silt.	SM	A-1-b
▽ Moist dense brown fine gravelly coarse sand w/some fine sand and silt.	SP-SM	A-1-b

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-6 **Depth:** 0.0 **Sample Number:** 1

□ **Source of Sample:** R-6 **Depth:** 4.0 **Sample Number:** 3

△ **Source of Sample:** R-6 **Depth:** 6.0 **Sample Number:** 4

◇ **Source of Sample:** R-6 **Depth:** 8.0 **Sample Number:** 5

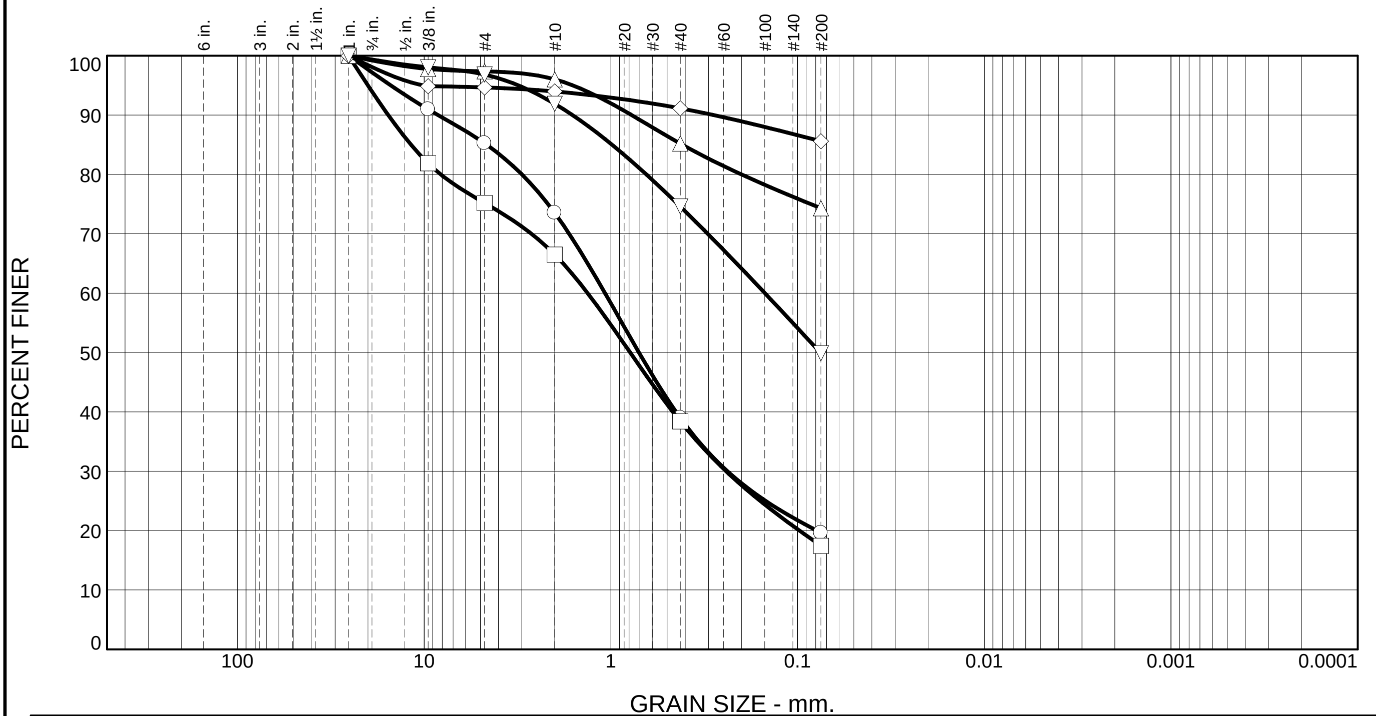
▽ **Source of Sample:** R-6 **Depth:** 10.0 **Sample Number:** 6

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand		% Fines	
			Coarse	Fine	Silt	Clay
○	0.0	26.5	34.5	19.4	19.6	
□	0.0	33.5	28.1	21.0	17.4	
△	0.0	4.0	10.8	10.9	74.3	
◇	0.0	6.0	2.9	5.5	85.6	
▽	0.0	8.1	17.3	24.7	49.9	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	NV	NP	4.6269	1.0750	0.7076	0.2376				
□	NV	NP	11.7608	1.3327	0.7885	0.2415				
△	38.2	26.2	0.4141							
◇	24.5	20.4								
▽	29.9	16.2	0.9854	0.1497	0.0757					

Material Description	USCS	AASHTO
○ Moist medium dense brown fine gravel and coarse sand w/some fine sand and silt.	SM	A-1-b
□ Moist dense brown coarse to fine sand and fine gravel w/some silt.	SM	A-1-b
△ Saturated stiff brown silty clay w/some fine to coarse sand, trace of fine gravel.	ML	A-6(9)
◇ Saturated stiff brown silt w/some clay, trace of fine to coarse sand and fine gravel.	CL-ML	A-4(2)
▽ Saturated firm gray fine sandy clay w/some coarse sand and silt, trace of fine gravel.	SC	A-6(4)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-6 **Depth:** 12.0 **Sample Number:** 7

□ **Source of Sample:** R-6 **Depth:** 14.0 **Sample Number:** 8

△ **Source of Sample:** R-6 **Depth:** 16.0 **Sample Number:** 9

◇ **Source of Sample:** R-6 **Depth:** 18.0 **Sample Number:** 10

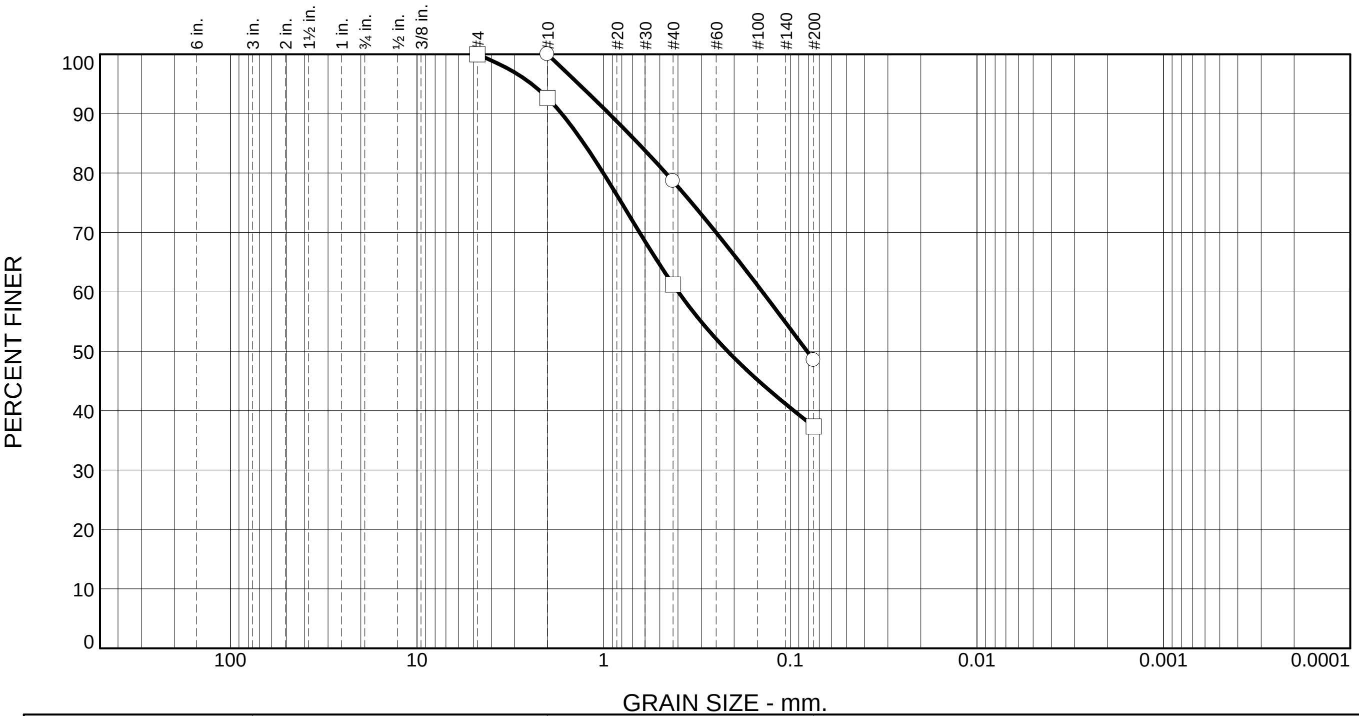
▽ **Source of Sample:** R-6 **Depth:** 24.0 **Sample Number:** 11

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand		% Fines	
			Coarse	Fine	Silt	Clay
○	0.0	0.0	21.4	30.1	48.5	
□	0.0	7.4	31.4	23.9	37.3	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	59.0	34.5	0.6524	0.1411	0.0814					
□	47.5	25.1	1.2778	0.3987	0.2166					

Material Description	USCS	AASHTO
○ Saturated very stiff gray fine to coarse sandy clay w/some silt.	SM	A-7-5(9)
□ Saturated hard brown coarse to fine sandy clay w/some silt, trace of fine gravel.	SC	A-7-6(3)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-6 **Depth:** 29.0 **Sample Number:** 12

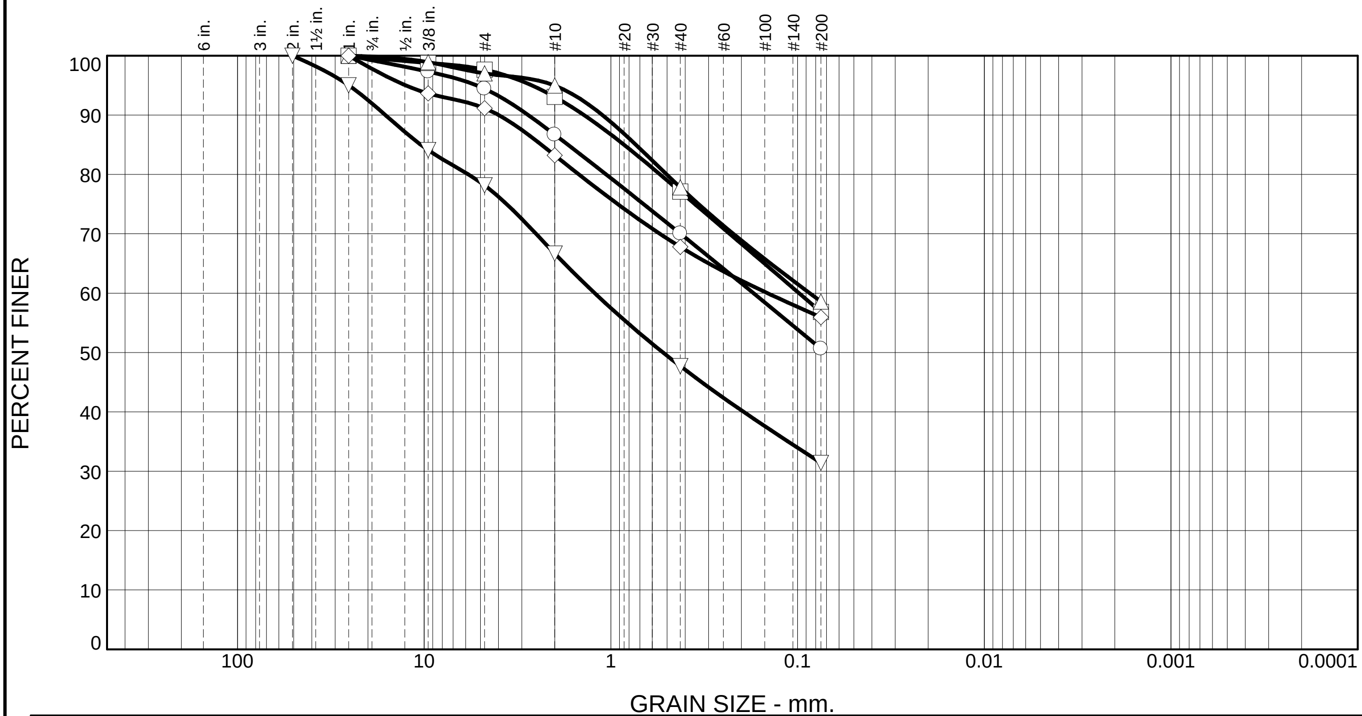
□ **Source of Sample:** R-6 **Depth:** 34.0 **Sample Number:** 13

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand Coarse	% Sand Fine	% Fines Silt	% Fines Clay
○	0.0	13.3	16.7	19.4	50.6	
□	0.0	6.9	16.0	20.3	56.8	
△	0.0	5.0	17.2	19.2	58.6	
◇	0.0	16.8	15.4	11.9	55.9	
▽	0.0	33.3	19.0	16.3	31.4	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	25.3	20.2	1.6996	0.1724						
□	22.6	19.0	0.8502	0.0983						
△	22.6	18.2	0.7347	0.0861						
◇	28.2	21.5	2.3575	0.1451						
▽	20.5	17.6	10.4214	1.2206	0.5256					

Material Description	USCS	AASHTO
○ Wet firm brown silt w/some fine to coarse sand, fine gravel and clay.	CL-ML	A-4(0)
□ Wet very stiff brown silt w/some fine to coarse sand and clay, trace of fine gravel.	CL-ML	A-4(0)
△ Wet very stiff brown silt w/some fine to coarse sand and clay, trace of fine gravel.	CL-ML	A-4(1)
◇ Wet hard brown clayey silt w/some coarse to fine sand and fine gravel.	CL-ML	A-4(1)
▽ Wet dense brown silty fine gravel w/some coarse to fine sand, trace of clay.	SM	A-2-4(0)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-7 **Depth:** 0.0 **Sample Number:** 1

□ **Source of Sample:** R-7 **Depth:** 2.0 **Sample Number:** 2

△ **Source of Sample:** R-7 **Depth:** 4.0 **Sample Number:** 3

◇ **Source of Sample:** R-7 **Depth:** 6.0 **Sample Number:** 4

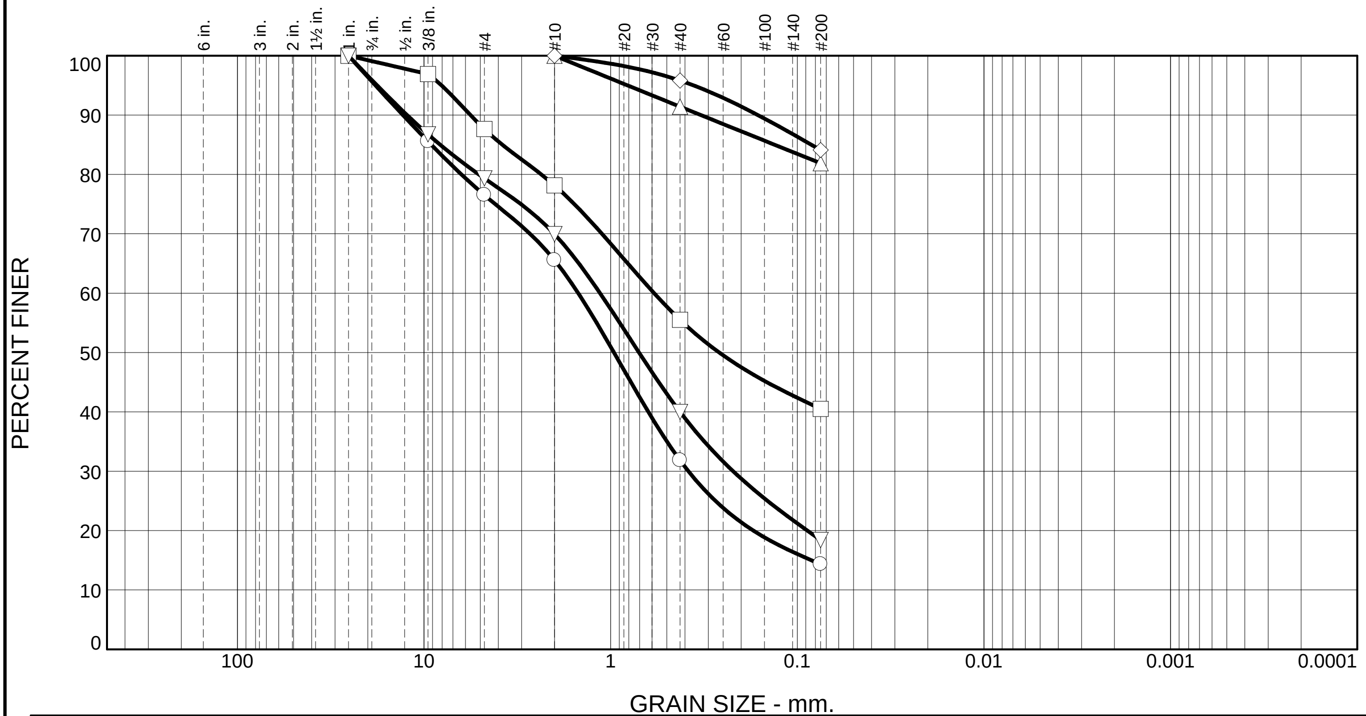
▽ **Source of Sample:** R-7 **Depth:** 8.0 **Sample Number:** 5

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand Coarse	% Sand Fine	% Fines Silt	% Fines Clay
○	0.0	34.4	33.8	17.5	14.3	
□	0.0	21.8	22.7	15.0	40.5	
△	0.0	0.0	8.6	9.5	81.9	
◇	0.0	0.0	4.2	11.7	84.1	
▽	0.0	30.1	29.9	21.6	18.4	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	NV	NP	9.1416	1.4931	0.9603	0.3834	0.0840			
□	25.4	16.6	3.8027	0.5849	0.2619					
△	28.6	19.0	0.1323							
◇	31.9	22.8	0.0841							
▽	NV	NP	8.1916	1.1405	0.7047	0.2206				

Material Description	USCS	AASHTO
○ Wet medium dense brown coarse sand and fine gravel w/some fine sand and silt.	SM	A-1-b
□ Wet stiff brown clayey coarse sandy fine gravelly silt w/some fine sand.	SC	A-4(0)
△ Wet clayey silt w/trace fine to coarse sand.	CL	A-4(7)
◇ Wet very stiff brown clayey silt w/some fine sand, trace of coarse sand.	CL	A-4(7)
▽ Wet medium dense brown coarse to fine sand and fine gravel w/some silt.	SM	A-1-b

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-7 **Depth:** 10.0 **Sample Number:** 6

□ **Source of Sample:** R-7 **Depth:** 12.0 **Sample Number:** 7

△ **Source of Sample:** R-7 **Depth:** 14.0 **Sample Number:** 8

◇ **Source of Sample:** R-7 **Depth:** 16.0 **Sample Number:** 9

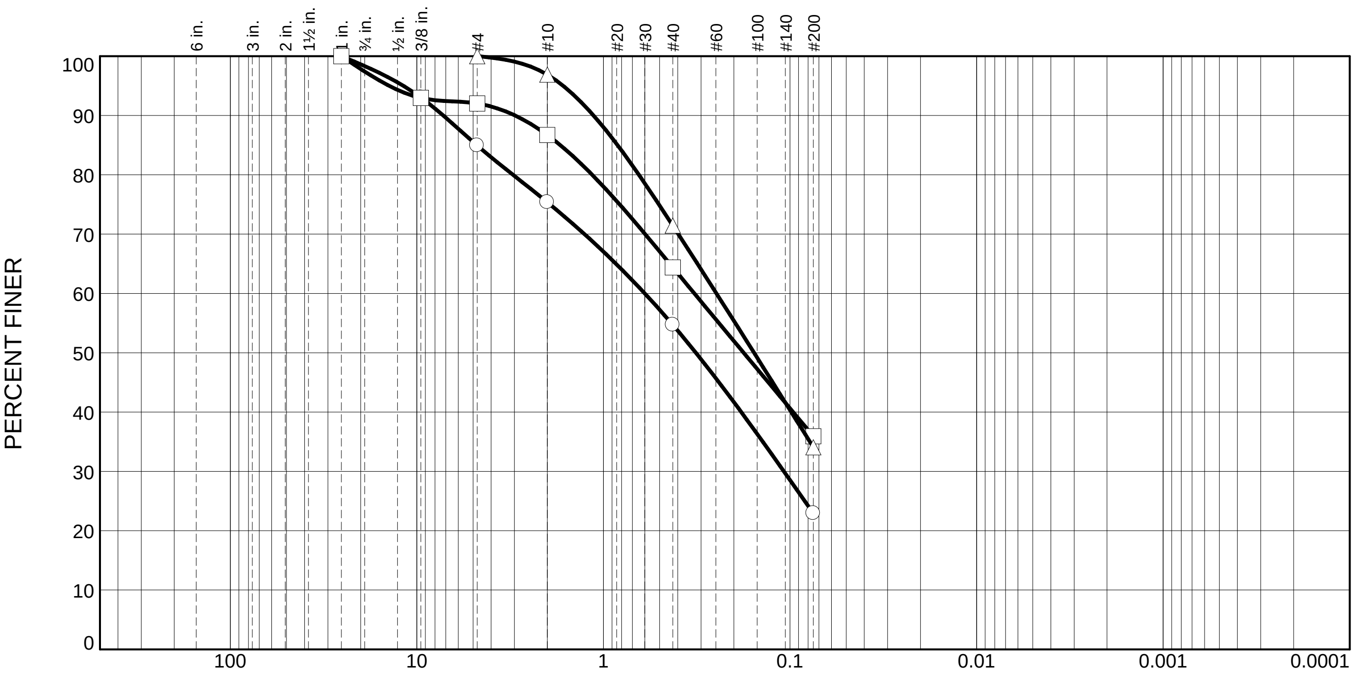
▽ **Source of Sample:** R-7 **Depth:** 20.0 **Sample Number:** 10

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand		% Fines	
			Coarse	Fine	Silt	Clay
○	0.0	24.6	20.7	31.8	22.9	
□	0.0	13.3	22.3	28.5	35.9	
△	0.0	3.1	25.5	37.3	34.1	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	32.8	18.7	4.7708	0.6003	0.3197	0.1079				
□	37.7	26.6	1.7044	0.3257	0.1773					
△	NV	NP	0.8404	0.2480	0.1559					

Material Description	USCS	AASHTO
○ Wet very dense brown clayey fine to coarse sand and fine gravel w/some silt.	SC	A-2-6(0)
□ Wet very stiff brown fine to coarse sandy clay w/some fine gravel and silt.	SM	A-6(0)
△ Wet very dense brown silty fine to coarse sand w/trace fine gravel.	SM	A-2-4(0)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-7 **Depth:** 24.0 **Sample Number:** 11

□ **Source of Sample:** R-7 **Depth:** 29.0 **Sample Number:** 12

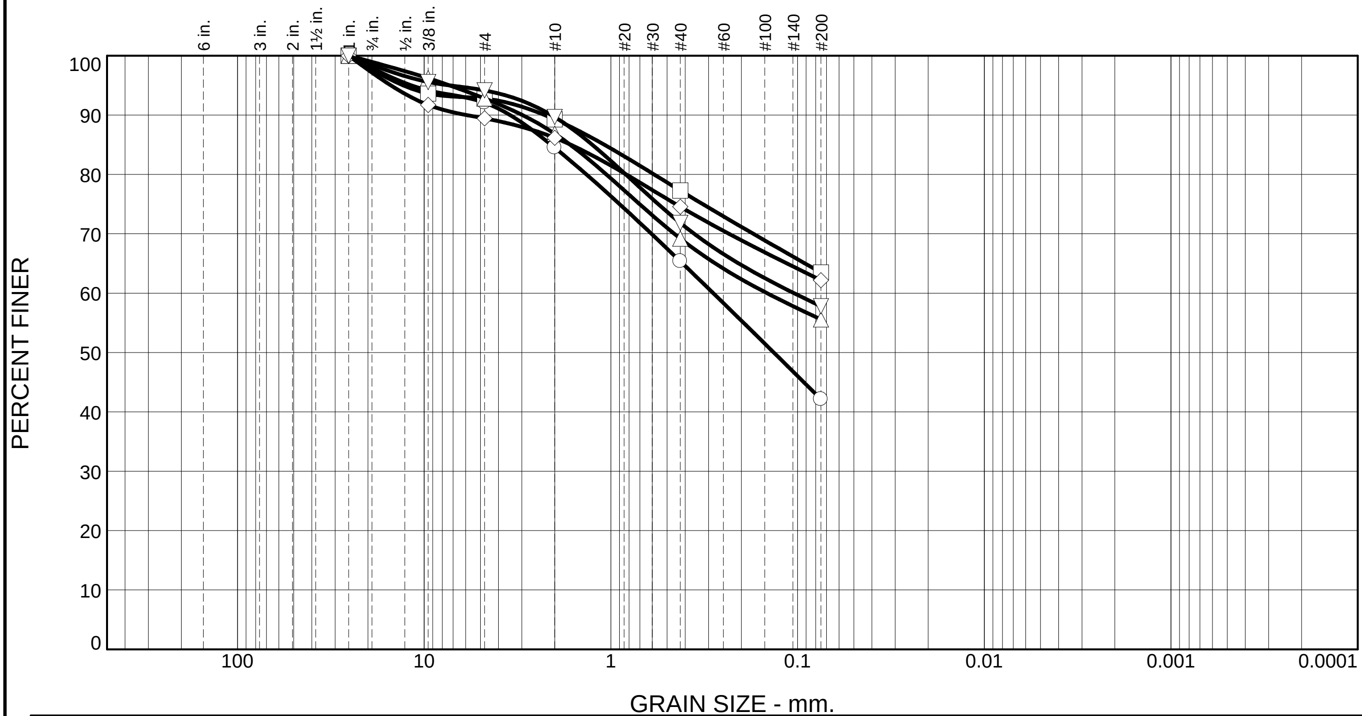
△ **Source of Sample:** R-7 **Depth:** 34.0 **Sample Number:** 13

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand Coarse	% Sand Fine	% Fines Silt	% Fines Clay
○	0.0	15.5	19.1	23.3	42.1	
□	0.0	10.7	12.0	13.8	63.5	
△	0.0	13.1	17.7	13.6	55.6	
◇	0.0	13.8	11.6	12.4	62.2	
▽	0.0	10.3	17.9	14.0	57.8	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	24.6	20.8	2.0907	0.2831	0.1342					
□	24.8	20.0	1.0833							
△	29.9	20.7	1.6504	0.1460						
◇	34.0	19.7	1.6402							
▽	62.5	33.1	1.2581	0.1046						

Material Description	USCS	AASHTO
○ Moist stiff gray fine sandy silt w/some coarse sand, fine gravel and clay.	SC-SM	A-4(0)
□ Moist stiff gray silt w/some fine to coarse sand, fine gravel and clay.	CL-ML	A-4(1)
△ Moist hard gray clayey silt w/some coarse to fine sand and fine gravel.	CL	A-4(3)
◇ Wet stiff gray silty clay w/some fine to coarse sand and fine gravel.	CL	A-6(6)
▽ Saturated firm brown clay w/some coarse to fine sand and silt, trace of fine gravel.	MH	A-7-5(15)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-8 **Depth:** 0.0 **Sample Number:** 1

□ **Source of Sample:** R-8 **Depth:** 2.0 **Sample Number:** 2

△ **Source of Sample:** R-8 **Depth:** 4.0 **Sample Number:** 3

◇ **Source of Sample:** R-8 **Depth:** 6.0 **Sample Number:** 4

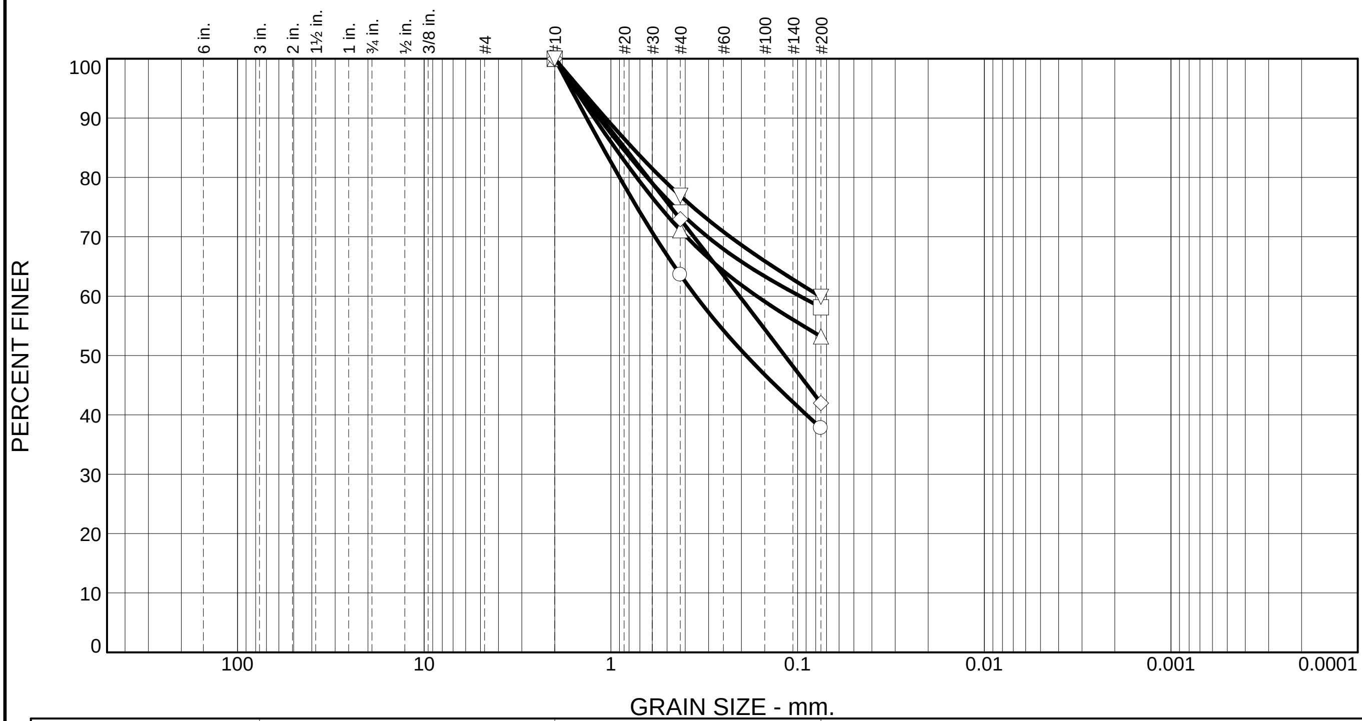
▽ **Source of Sample:** R-8 **Depth:** 8.0 **Sample Number:** 5

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand		% Fines	
			Coarse	Fine	Silt	Clay
○	0.0	0.0	36.4	25.8	37.8	
□	0.0	0.0	25.9	16.0	58.1	
△	0.0	0.0	28.9	17.9	53.2	
◇	0.0	0.0	27.0	31.0	42.0	
▽	0.0	0.0	23.1	17.0	59.9	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	44.7	32.5	1.1037	0.3506	0.1887					
□	69.8	38.4	0.8693	0.0972						
△	54.3	31.5	0.9506	0.1655						
◇	37.2	NP	0.8442	0.2049	0.1172					
▽	63.1	38.1	0.7667	0.0760						

Material Description	USCS	AASHTO
○ Saturated very stiff brown coarse to fine sandy clay w/some silt.	SM	A-7-5(1)
□ Saturated stiff brown coarse sandy clay w/some fine sand and silt.	MH	A-7-5(18)
△ Saturated very stiff brown coarse sandy clay w/some fine sand and silt.	MH	A-7-5(9)
◇ Saturated very stiff brown fine to coarse sandy silt.	SM	A-4(0)
▽ Saturated very stiff brown coarse sandy clay w/some fine sand and silt.	MH	A-7-5(15)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-8 **Depth:** 10.0 **Sample Number:** 6

□ **Source of Sample:** R-8 **Depth:** 12.0 **Sample Number:** 7

△ **Source of Sample:** R-8 **Depth:** 14.0 **Sample Number:** 8

◇ **Source of Sample:** R-8 **Depth:** 16.0 **Sample Number:** 9

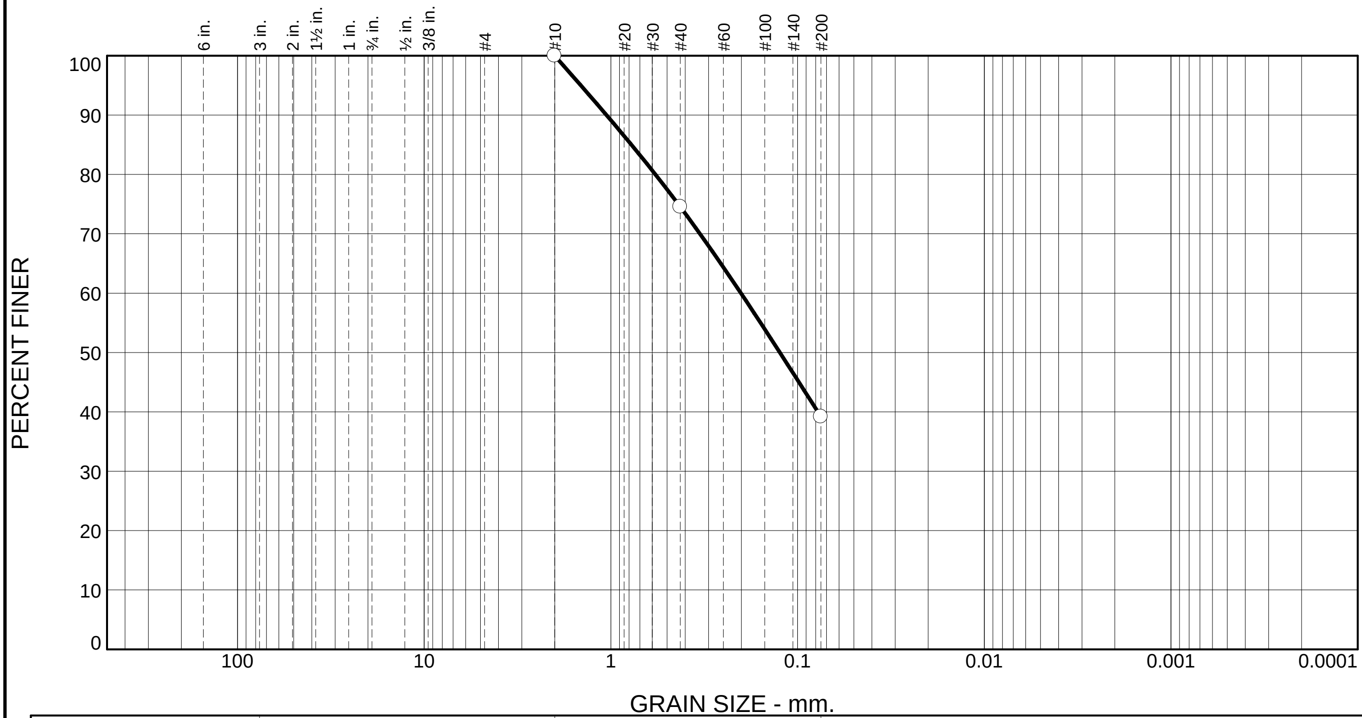
▽ **Source of Sample:** R-8 **Depth:** 18.0 **Sample Number:** 10

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand Coarse	% Sand Fine	% Fines Silt	% Fines Clay
○	0.0	0.0	25.5	35.3	39.2	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	47.6	NP	0.7764	0.2012	0.1245					

Material Description	USCS	AASHTO
○ Saturated stiff gray fine to coarse sandy silt.	SM	A-5(0)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

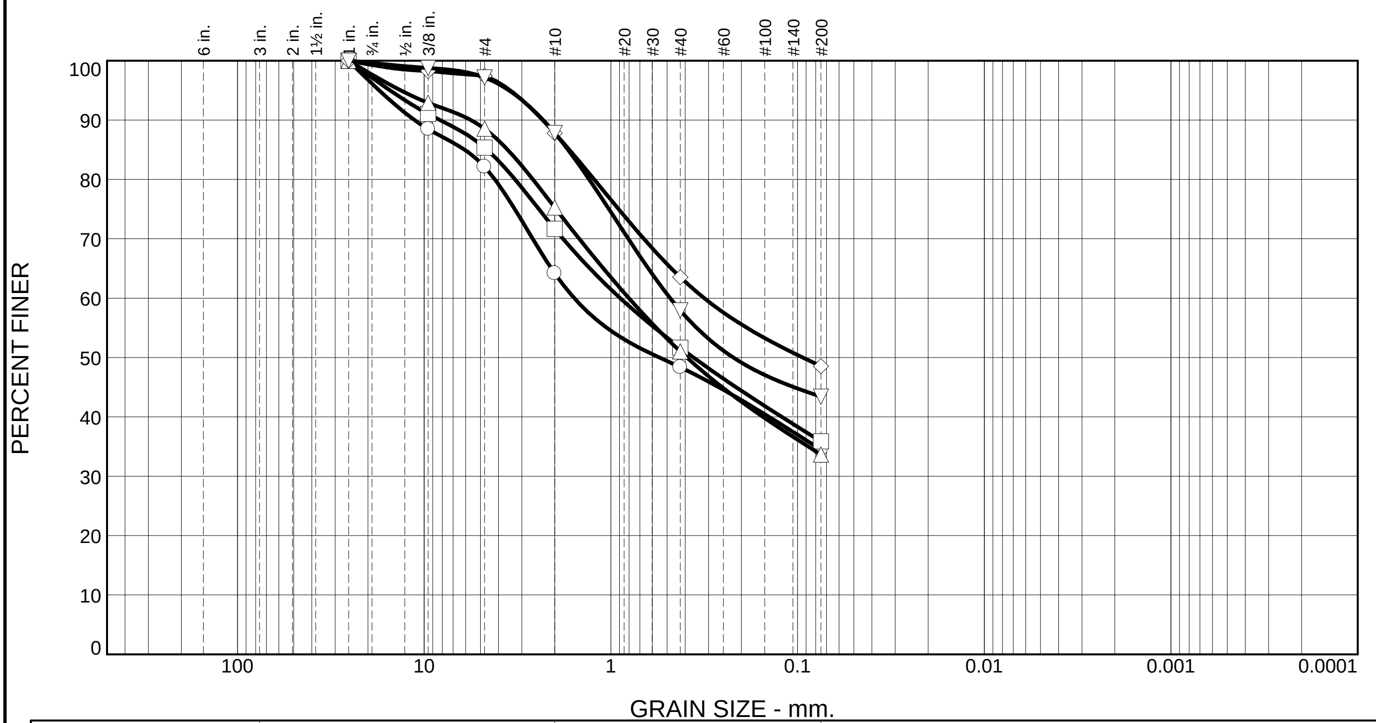
○ **Source of Sample:** R-8 **Depth:** 24.0 **Sample Number:** 11

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand Coarse	% Sand Fine	% Fines Silt	% Fines Clay
○	0.0	35.8	15.8	13.9	34.5	
□	0.0	28.3	20.1	15.7	35.9	
△	0.0	24.7	24.3	17.3	33.7	
◇	0.0	12.2	24.3	15.0	48.5	
▽	0.0	12.2	29.9	14.5	43.4	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	26.0	19.3	6.0666	1.5715	0.5495					
□	28.6	20.3	4.6046	0.8906	0.3611					
△	44.6	26.0	3.5710	0.8011	0.3918					
◇	55.7	27.2	1.6710	0.3161	0.0938					
▽	54.7	26.6	1.6975	0.4814	0.2211					

Material Description	USCS	AASHTO
○ Moist medium dense brown clayey fine gravel w/some coarse to fine sand and silt.	SC-SM	A-2-4(0)
□ Moist very stiff brown clayey fine gravelly silt w/some coarse to fine sand.	SC	A-4(0)
△ Wet loose brown clayey coarse sand and fine gravel w/some fine sand and silt.	SC	A-2-7(2)
◇ Wet firm brown coarse sandy clay w/some fine sand, fine gravel and silt.	SC	A-7-6(10)
▽ Wet stiff brown coarse sandy clay w/some fine sand, fine gravel and silt.	SC	A-7-6(7)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-9 **Depth:** 4.0 **Sample Number:** 3

□ **Source of Sample:** R-9 **Depth:** 6.0 **Sample Number:** 4

△ **Source of Sample:** R-9 **Depth:** 10.0 **Sample Number:** 6

◇ **Source of Sample:** R-9 **Depth:** 12.0 **Sample Number:** 7

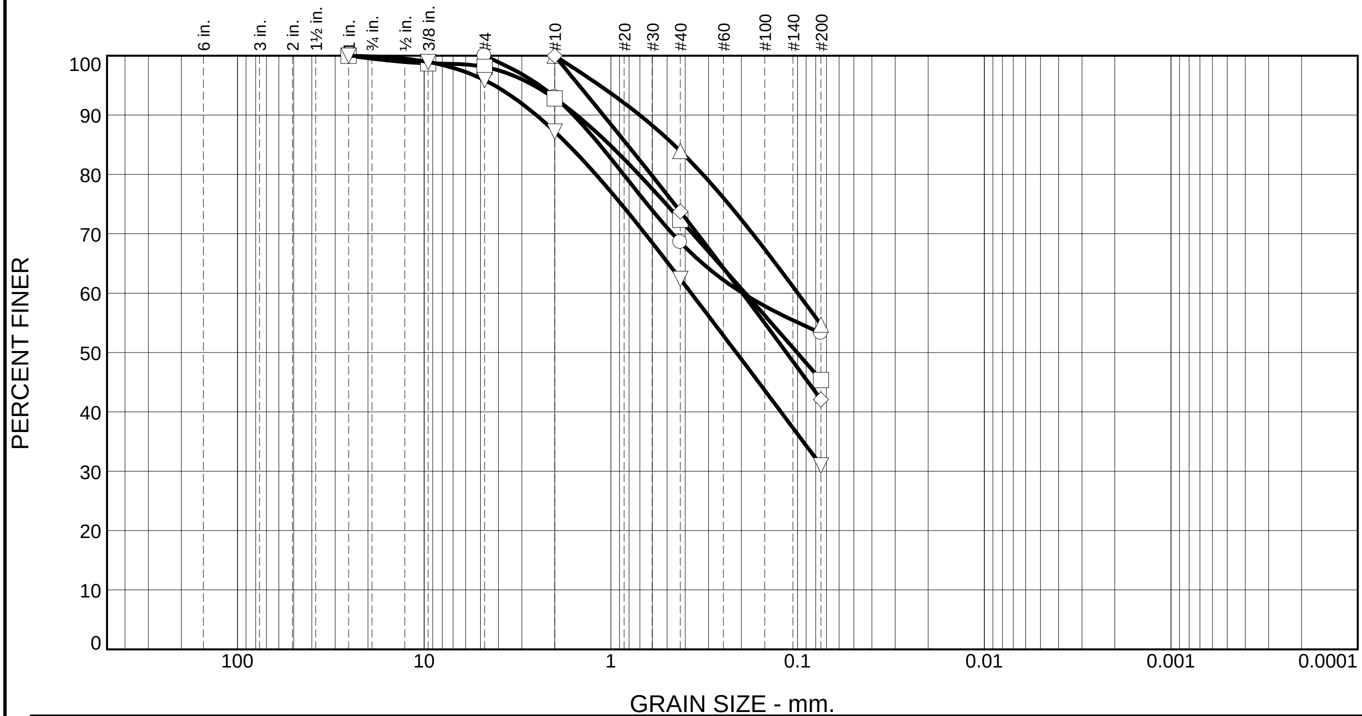
▽ **Source of Sample:** R-9 **Depth:** 14.0 **Sample Number:** 8

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand Coarse	% Sand Fine	% Fines Silt	% Fines Clay
○	0.0	7.0	24.4	15.3	53.3	
□	0.0	7.2	20.4	27.0	45.4	
△	0.0	0.0	16.0	29.3	54.7	
◇	0.0	0.0	26.2	31.7	42.1	
▽	0.0	12.8	24.8	31.4	31.0	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	57.9	28.7	1.1482	0.1967						
□	61.1	31.6	1.0150	0.1909	0.1007					
△	49.1	31.1	0.4602	0.0998						
◇	38.9	28.2	0.8165	0.1976	0.1148					
▽	36.5	26.3	1.6941	0.3716	0.2131					

Material Description	USCS	AASHTO
○ Wet very stiff brown coarse sandy clay w/some fine sand and silt, trace of fine gravel.	CH	A-7-6(12)
□ Wet very stiff gray fine sandy clay w/some coarse sand and silt, trace of fine gravel.	SM	A-7-5(9)
△ Wet very stiff gray fine sandy clay w/some coarse sand and silt.	ML	A-7-5(8)
◇ Wet hard gray fine to coarse sandy clay w/some silt.	SM	A-6(2)
▽ Wet very dense gray clayey fine to coarse sand w/some fine gravel and silt.	SM	A-2-4(0)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-9 **Depth:** 16.0 **Sample Number:** 9

□ **Source of Sample:** R-9 **Depth:** 18.0 **Sample Number:** 10

△ **Source of Sample:** R-9 **Depth:** 24.0 **Sample Number:** 11

◇ **Source of Sample:** R-9 **Depth:** 28.0 **Sample Number:** 12

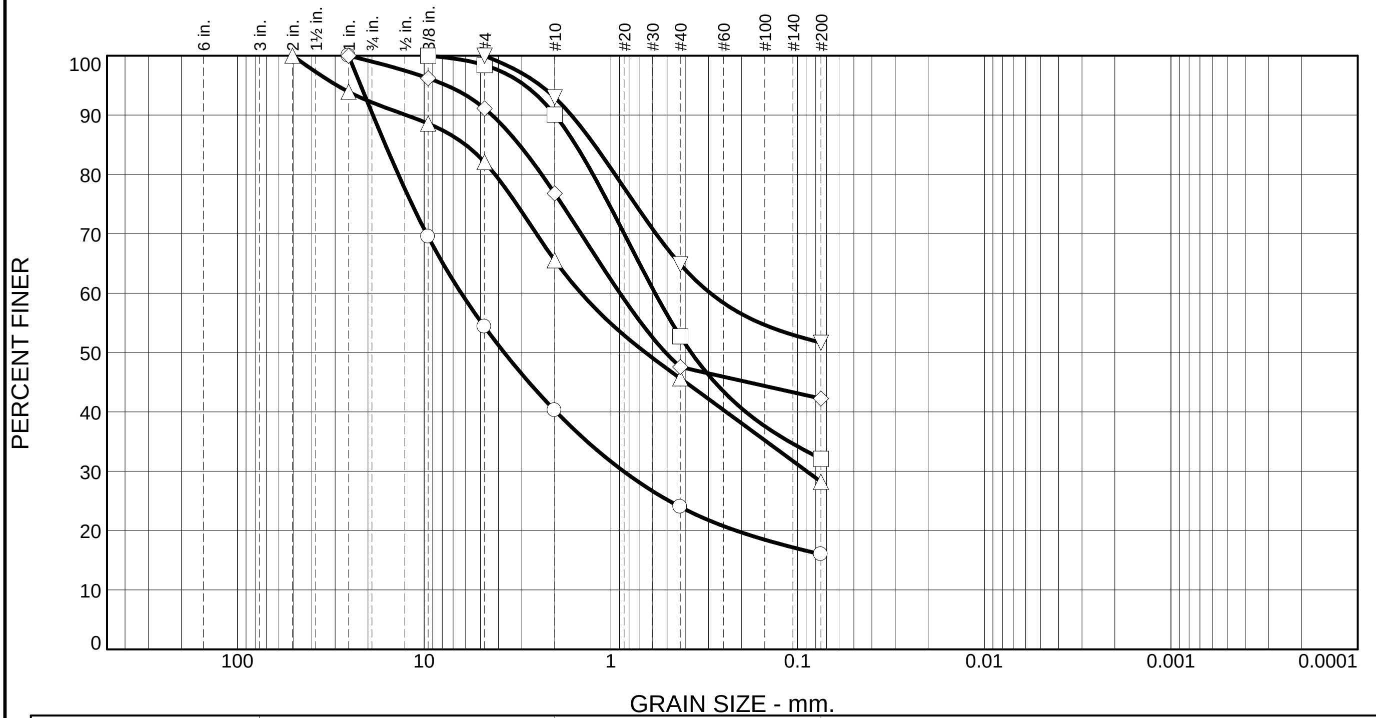
▽ **Source of Sample:** R-9 **Depth:** 34.0 **Sample Number:** 13

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand		% Fines	
			Coarse	Fine	Silt	Clay
○	0.0	59.7	16.3	8.0	16.0	
□	0.0	9.9	37.4	20.6	32.1	
△	0.0	34.5	19.9	17.4	28.2	
◇	0.0	23.2	29.2	5.4	42.2	
▽	0.0	7.0	28.1	13.3	51.6	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	17.2	NP	16.1383	6.3317	3.7300	0.8560				
□	23.2	17.3	1.5444	0.5797	0.3716					
△	27.1	19.1	6.0025	1.4452	0.6532	0.0894				
◇	69.0	32.0	3.0945	0.8949	0.5094					
▽	65.6	32.5	1.2267	0.2933						

Material Description	USCS	AASHTO
○ Moist gray fine gravel w/some coarse sand and silt, trace of fine sand.	GM	A-1-b
□ Moist loose brown clayey coarse to fine sand w/some silt, trace of fine gravel.	SC-SM	A-2-4(0)
△ Moist loose brown clayey fine gravel w/ some coarse to fine sand and silt.	SC	A-2-4(0)
◇ Wet stiff brown coarse sandy fine gravelly clay w/some silt, trace of fine sand.	SC	A-7-5(10)
▽ Saturated very stiff brown coarse sandy clay w/some fine sand and silt, trace of fine gravel.	CH	A-7-5(14)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-10 **Depth:** 0.0 **Sample Number:** 1

□ **Source of Sample:** R-10 **Depth:** 2.0 **Sample Number:** 2

△ **Source of Sample:** R-10 **Depth:** 6.0 **Sample Number:** 4

◇ **Source of Sample:** R-10 **Depth:** 8.0 **Sample Number:** 5

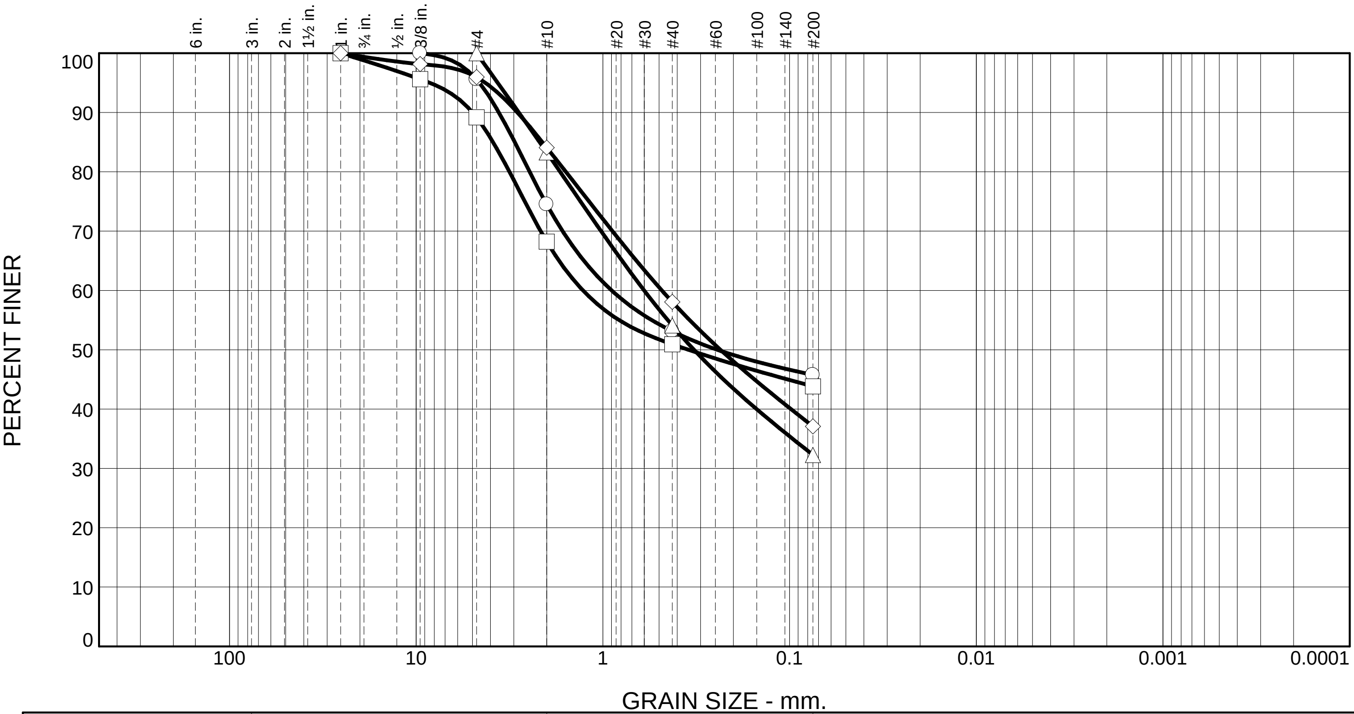
▽ **Source of Sample:** R-10 **Depth:** 10.0 **Sample Number:** 6

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand		% Fines	
			Coarse	Fine	Silt	Clay
○	0.0	25.5	21.4	7.4	45.7	
□	0.0	31.8	17.3	7.1	43.8	
△	0.0	16.7	29.2	21.8	32.3	
◇	0.0	15.9	26.0	21.0	37.1	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	64.8	35.7	2.9675	0.8980	0.2438					
□	68.3	37.9	3.8737	1.2782	0.3484					
△	53.2	31.5	2.1826	0.6011	0.3252					
◇	36.6	26.3	2.1103	0.4831	0.2346					

Material Description	USCS	AASHTO
○ Saturated stiff brown coarse sandy fine gravelly clay w/some silt, trace of fine sand.	SM	A-7-5(9)
□ Saturated very stiff brown fine gravelly clay w/some coarse sand and silt, trace of fine sand.	SM	A-7-5(9)
△ Saturated dense brown clayey coarse to fine sand w/some fine gravel and silt.	SM	A-2-7(2)
◇ Saturated hard gray coarse to fine sandy clay w/some silt and fine gravel.	SM	A-6(1)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-10 **Depth:** 12.0 **Sample Number:** 7

□ **Source of Sample:** R-10 **Depth:** 14.0 **Sample Number:** 8

△ **Source of Sample:** R-10 **Depth:** 16.0 **Sample Number:** 9

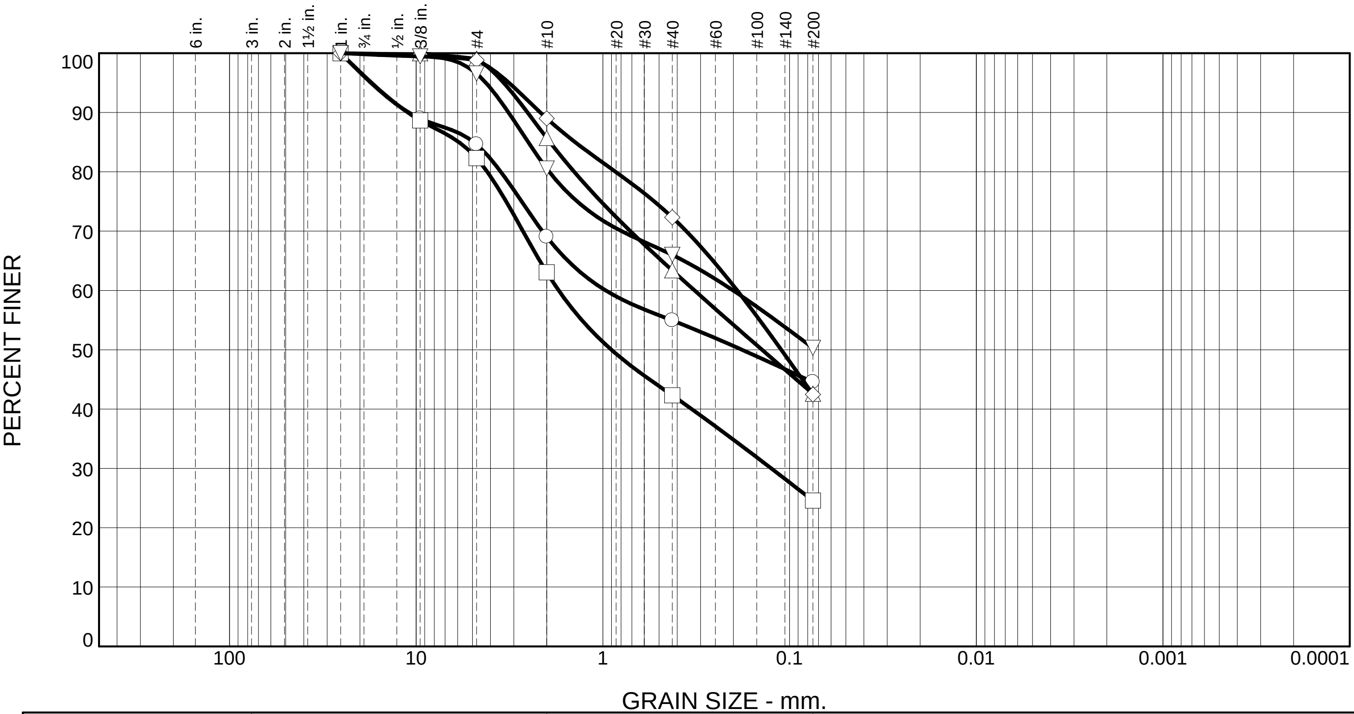
◇ **Source of Sample:** R-10 **Depth:** 18.0 **Sample Number:** 10

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand		% Fines	
			Coarse	Fine	Silt	Clay
○	0.0	31.0	14.1	10.4	44.5	
□	0.0	36.9	20.8	17.7	24.6	
△	0.0	14.3	22.3	20.8	42.6	
◇	0.0	11.0	16.7	29.8	42.5	
▽	0.0	19.4	14.6	15.7	50.3	

╳	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	55.2	26.8	4.9196	0.9656	0.1798					
□	NV	NP	5.9057	1.7261	0.9004	0.1254				
△	60.1	35.0	1.9219	0.3229	0.1405					
◇	58.3	30.7	1.4087	0.1910	0.1109					
▽	57.2	31.4	2.5125	0.2007						

Material Description	USCS	AASHTO
○ Wet gray fine gravelly clay w/some coarse sand and silt, trace of fine sand.	SC	A-7-6(8)
□ Wet medium dense brown silty coarse sandy fine gravel w/some fine sand.	SM	A-1-b
△ Wet very stiff brown coarse to fine sandy clay w/some fine gravel and silt.	SM	A-7-5(7)
◇ Wet hard brown fine sandy clay w/some coarse sand, fine gravel and silt.	SM	A-7-5(7)
▽ Wet very stiff brown clay w/some fine to coarse sand, fine gravel and silt.	MH	A-7-5(10)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-11 **Depth:** 0.0 **Sample Number:** 1

□ **Source of Sample:** R-11 **Depth:** 2.0 **Sample Number:** 2

△ **Source of Sample:** R-11 **Depth:** 4.0 **Sample Number:** 3

◇ **Source of Sample:** R-11 **Depth:** 6.0 **Sample Number:** 4

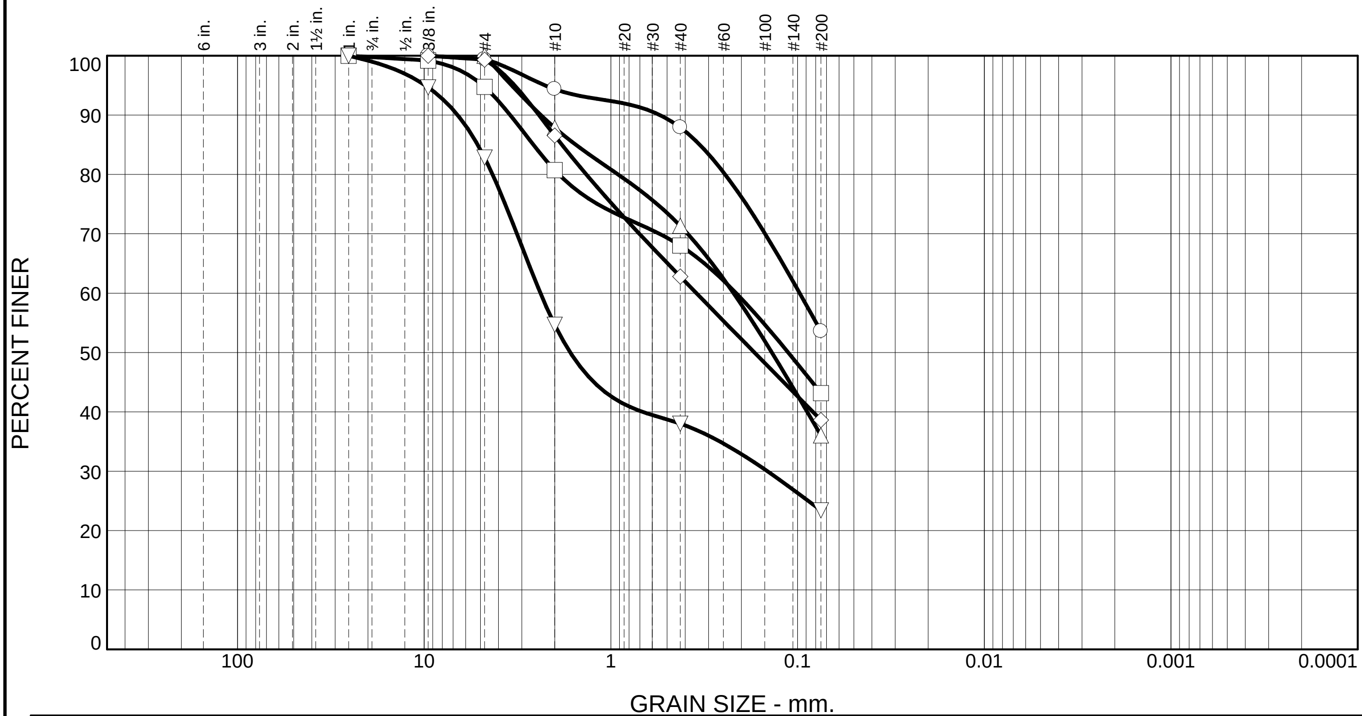
▽ **Source of Sample:** R-11 **Depth:** 8.0 **Sample Number:** 5

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand Coarse	% Sand Fine	% Fines Silt	% Fines Clay
○	0.0	5.6	6.5	34.3	53.6	
□	0.0	19.3	12.7	24.8	43.2	
△	0.0	12.1	16.6	35.3	36.0	
◇	0.0	13.5	23.7	24.2	38.6	
▽	0.0	45.3	16.7	14.6	23.4	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	45.4	26.7	0.3341	0.0975						
□	52.0	28.5	2.6020	0.2135	0.1123					
△	59.0	36.6	1.5417	0.2203	0.1374					
◇	62.3	37.8	1.8299	0.3484	0.1703					
▽	65.0	35.4	5.1676	2.3846	1.6506	0.1452				

Material Description	USCS	AASHTO
○ Wet hard brown fine sandy clay w/some silt, trace of coarse sand and fine gravel.	CL	A-7-6(7)
□ Wet very stiff brown fine sandy clay w/ some coarse sand, fine gravel and silt.	SC	A-7-6(6)
△ Wet very stiff brown fine sandy clay w/ some coarse sand, fine gravel and silt.	SM	A-7-5(3)
◇ Wet hard brown fine to coarse sandy clay w/some fine gravel and silt.	SM	A-7-5(5)
▽ Wet dense gray clayey fine gravel w/some coarse to fine sand and silt.	SM	A-2-7(2)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-11 **Depth:** 10.0 **Sample Number:** 6

□ **Source of Sample:** R-11 **Depth:** 12.0 **Sample Number:** 7

△ **Source of Sample:** R-11 **Depth:** 14.0 **Sample Number:** 8

◇ **Source of Sample:** R-11 **Depth:** 16.0 **Sample Number:** 9

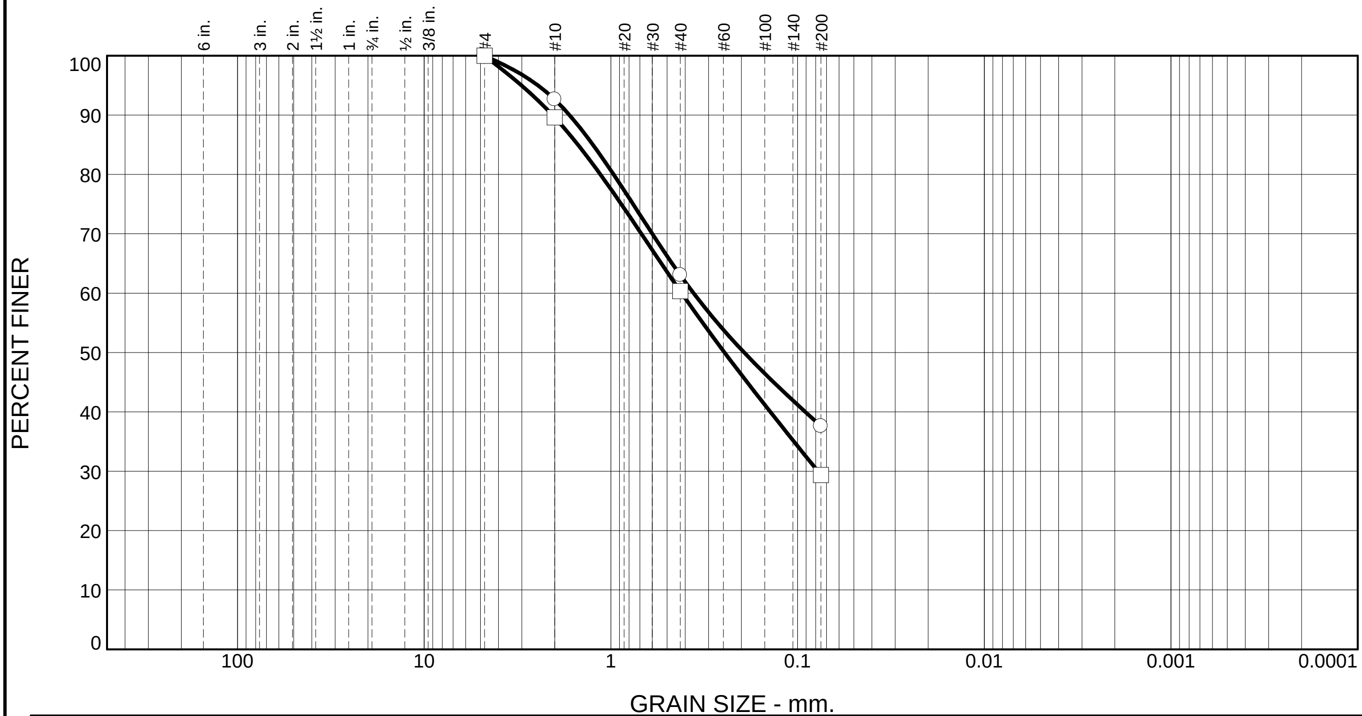
▽ **Source of Sample:** R-11 **Depth:** 18.0 **Sample Number:** 10

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand		% Fines	
			Coarse	Fine	Silt	Clay
○	0.0	7.4	29.6	25.4	37.6	
□	0.0	10.4	29.3	30.9	29.4	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	57.8	39.3	1.2481	0.3607	0.1934					
□	55.7	38.5	1.5009	0.4178	0.2462	0.0779				

Material Description	USCS	AASHTO
○ Saturated hard gray coarse to fine sandy clay w/some silt, trace of fine gravel.	SM	A-7-5(3)
□ Saturated very dense brown clayey fine to coarse sand w/some silt, trace of fine gravel.	SM	A-2-7(1)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-11 **Depth:** 24.0 **Sample Number:** 11

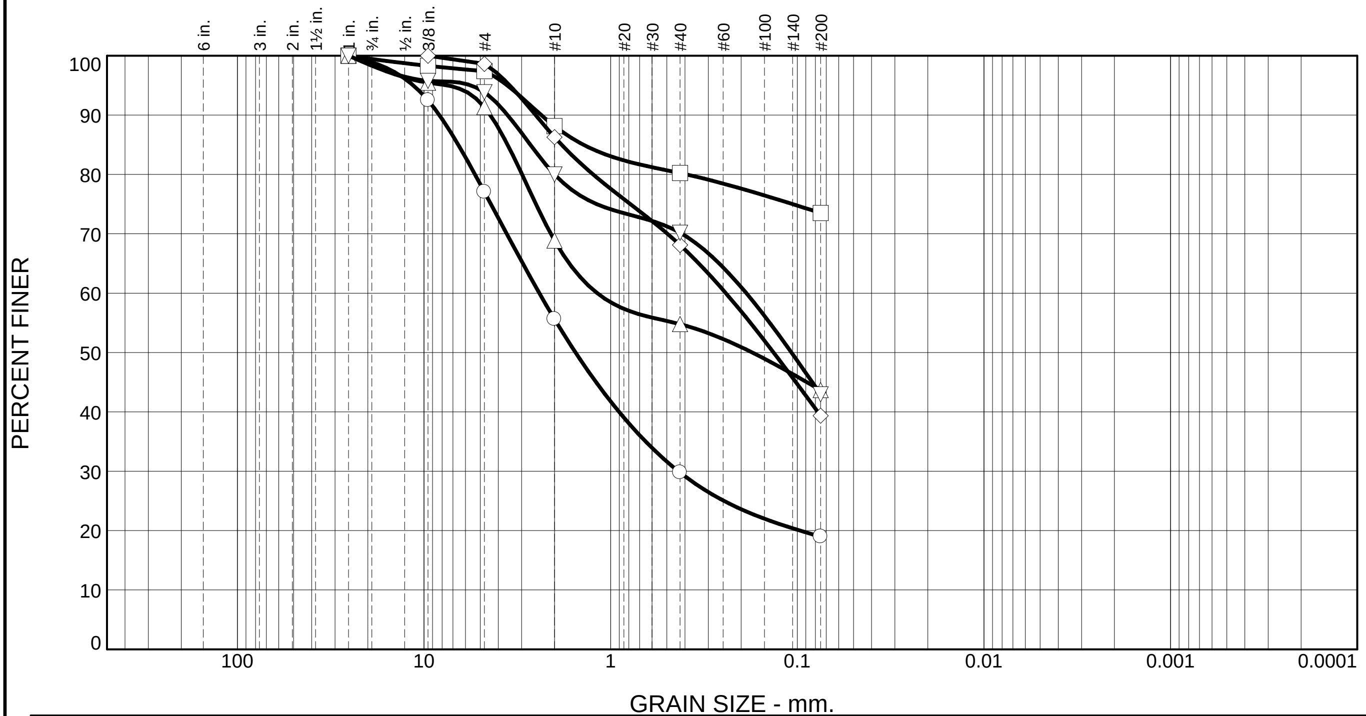
□ **Source of Sample:** R-11 **Depth:** 29.0 **Sample Number:** 12

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand Coarse	% Sand Fine	% Fines Silt	% Fines Clay
○	0.0	44.4	25.8	10.8	19.0	
□	0.0	11.9	7.9	6.7	73.5	
△	0.0	31.1	14.1	11.1	43.7	
◇	0.0	13.7	18.2	28.7	39.4	
▽	0.0	20.0	9.8	27.2	43.0	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	NV	NP	6.5553	2.4112	1.5433	0.4335				
□	29.8	21.8	1.3979							
△	49.2	26.1	3.5666	1.1811	0.1745					
◇	53.6	30.8	1.8336	0.2416	0.1341					
▽	58.3	37.4	2.6936	0.1877	0.1076					

Material Description	USCS	AASHTO
○ Moist brown coarse sandy fine gravel w/ some fine sand and silt.	SM	A-1-b
□ Wet very stiff brown clayey silt w/some fine gravel, trace of coarse to fine sand.	CL	A-4(5)
△ Wet stiff brown fine gravelly clay w/some coarse to fine sand and silt.	SC	A-7-6(6)
◇ Wet very stiff brown fine sandy clay w/ some coarse sand, fine gravel and silt.	SM	A-7-5(4)
▽ Wet stiff brown fine sandy clay w/some fine gravel and silt, trace of coarse sand.	SM	A-7-5(5)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-12 **Depth:** 0.0 **Sample Number:** 1

□ **Source of Sample:** R-12 **Depth:** 2.0 **Sample Number:** 2

△ **Source of Sample:** R-12 **Depth:** 4.0 **Sample Number:** 3

◇ **Source of Sample:** R-12 **Depth:** 6.0 **Sample Number:** 4

▽ **Source of Sample:** R-12 **Depth:** 8.0 **Sample Number:** 5

Remarks:

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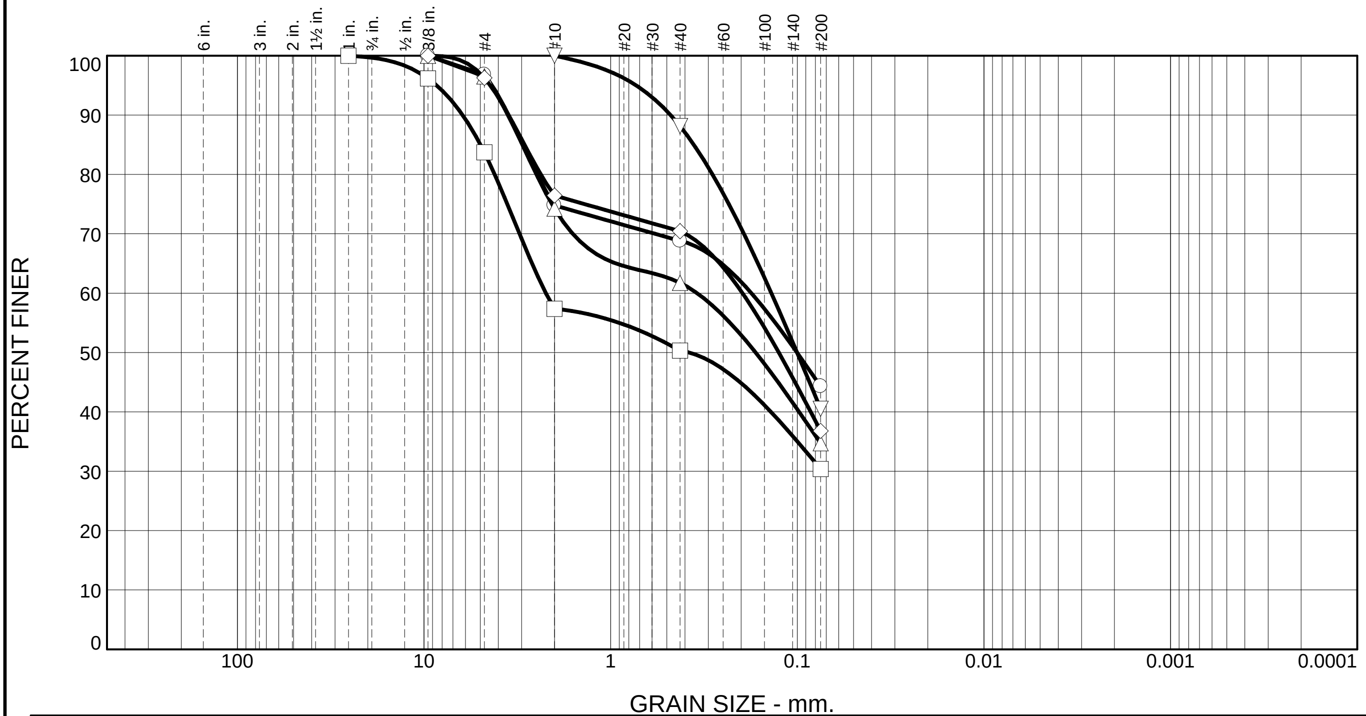
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand Coarse	% Sand Fine	% Fines Silt	% Fines Clay
○	0.0	25.2	6.0	24.5	44.3	
□	0.0	42.6	7.1	19.9	30.4	
△	0.0	25.8	12.5	27.0	34.7	
◇	0.0	23.6	6.0	33.6	36.8	
▽	0.0	0.0	11.9	47.6	40.5	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	64.2	41.6	2.9531	0.1763	0.1004					
□	61.1	37.4	4.9826	2.2247	0.3797					
△	64.2	39.1	2.9738	0.3445	0.1673					
◇	60.8	38.1	2.8966	0.1967	0.1257					
▽	54.9	30.1	0.3586	0.1378	0.1004					

Material Description	USCS	AASHTO
○ Saturated very stiff brown fine sandy fine gravelly clay w/some silt, trace of coarse sand.	SM	A-7-5(6)
□ Saturated medium dense brown silty fine gravel w/some fine sand, trace of coarse sand.	SM	A-2-7(2)
△ Saturated medium dense brown clayey fine sand and fine gravel w/some coarse sand and silt.	SM	A-2-7(3)
◇ Saturated very stiff brown fine sandy fine gravelly clay w/some silt, trace of coarse sand.	SM	A-7-5(3)
▽ Saturated very stiff brown fine sandy clay w/some coarse sand and silt.	SM	A-7-5(5)

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-12 **Depth:** 10.0 **Sample Number:** 6

□ **Source of Sample:** R-12 **Depth:** 12.0 **Sample Number:** 7

△ **Source of Sample:** R-12 **Depth:** 14.0 **Sample Number:** 8

◇ **Source of Sample:** R-12 **Depth:** 16.0 **Sample Number:** 9

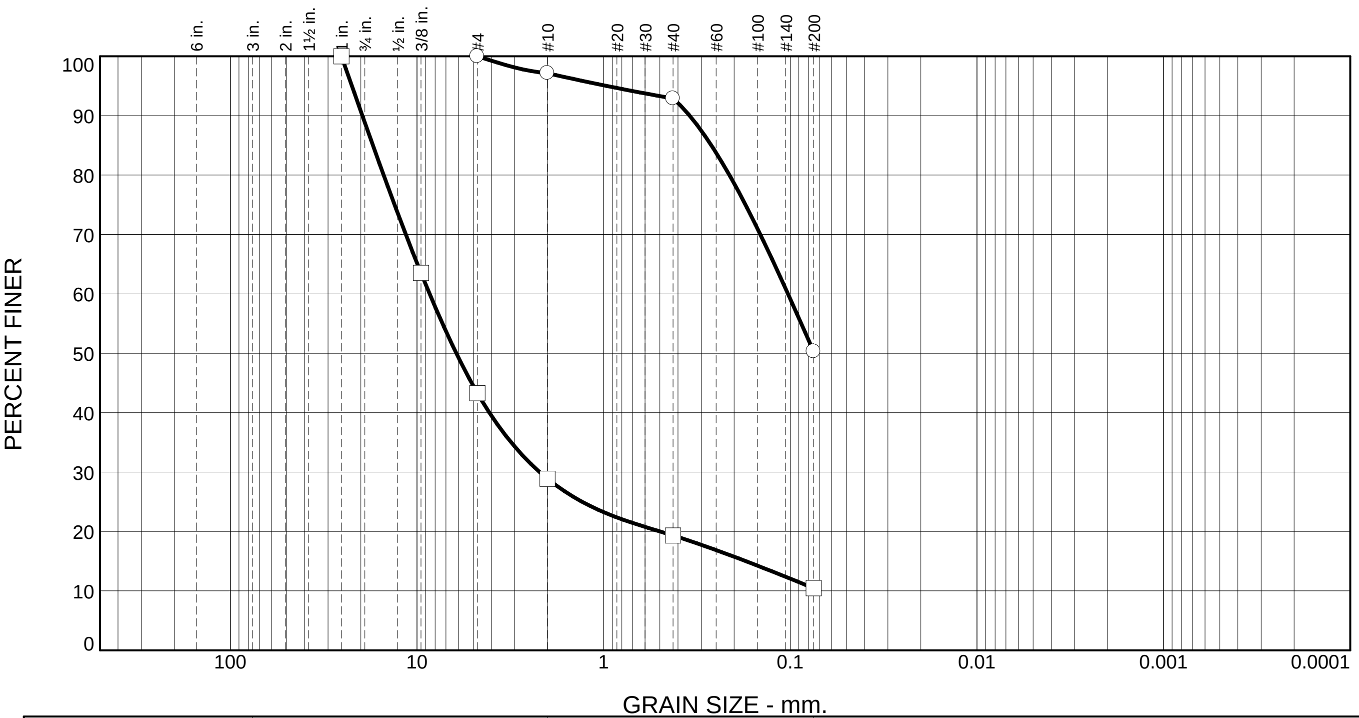
▽ **Source of Sample:** R-12 **Depth:** 18.0 **Sample Number:** 10

Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Particle Size Distribution Report



	% +3"	% Gravel	% Sand		% Fines	
			Coarse	Fine	Silt	Clay
○	0.0	2.9	4.2	42.6	50.3	
□	0.0	71.1	9.6	8.8	10.5	

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	47.4	34.9	0.2647	0.1030						
□	NV	NP	17.2295	8.5625	6.1593	2.2043	0.1731			

Material Description	USCS	AASHTO
○ Saturated hard brown fine sandy clay w/ some silt, trace of coarse sand and fine gravel.	ML	A-7-5(4)
□ Saturated very dense gray fine gravel w/some silt, trace of coarse to fine sand.	GP-GM	A-1-a

Project No. T201407404 **Client:**

Project: Wilmington Viaduct

○ **Source of Sample:** R-12 **Depth:** 24.0 **Sample Number:** 11

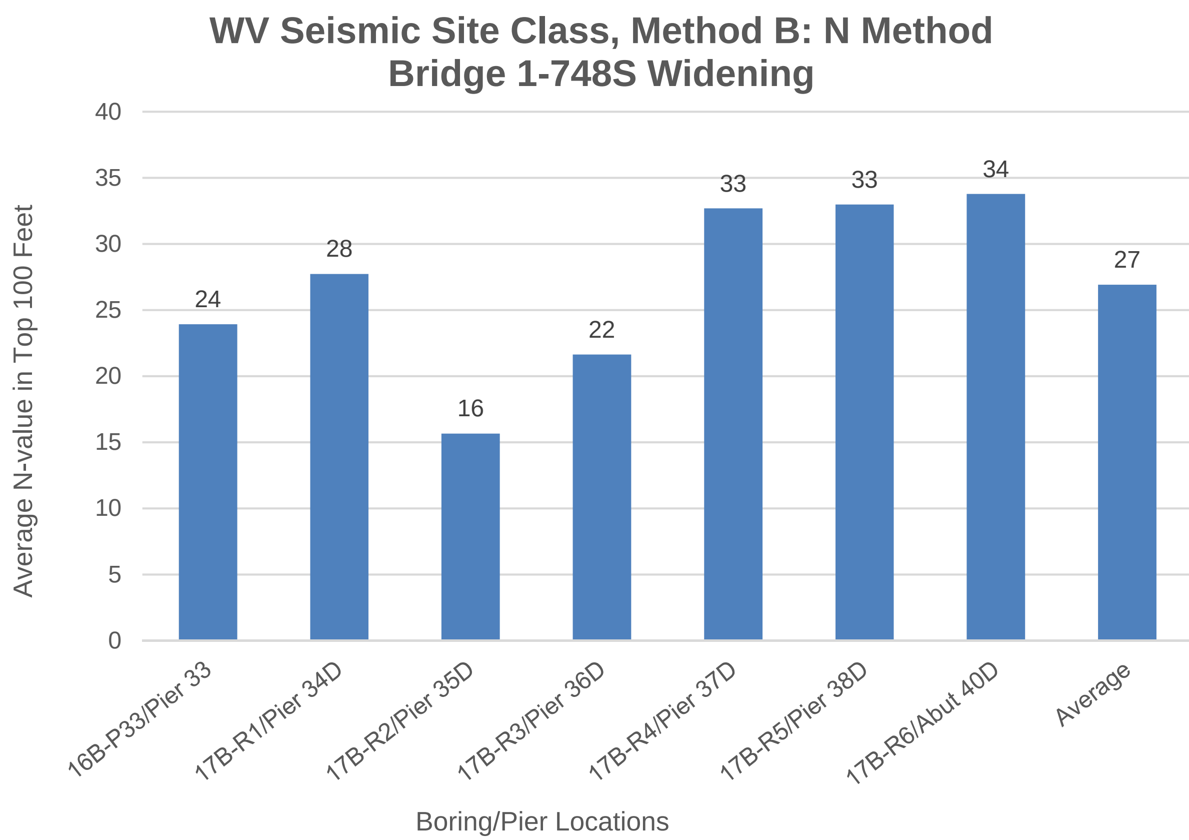
□ **Source of Sample:** R-12 **Depth:** 28.0 **Sample Number:** 12

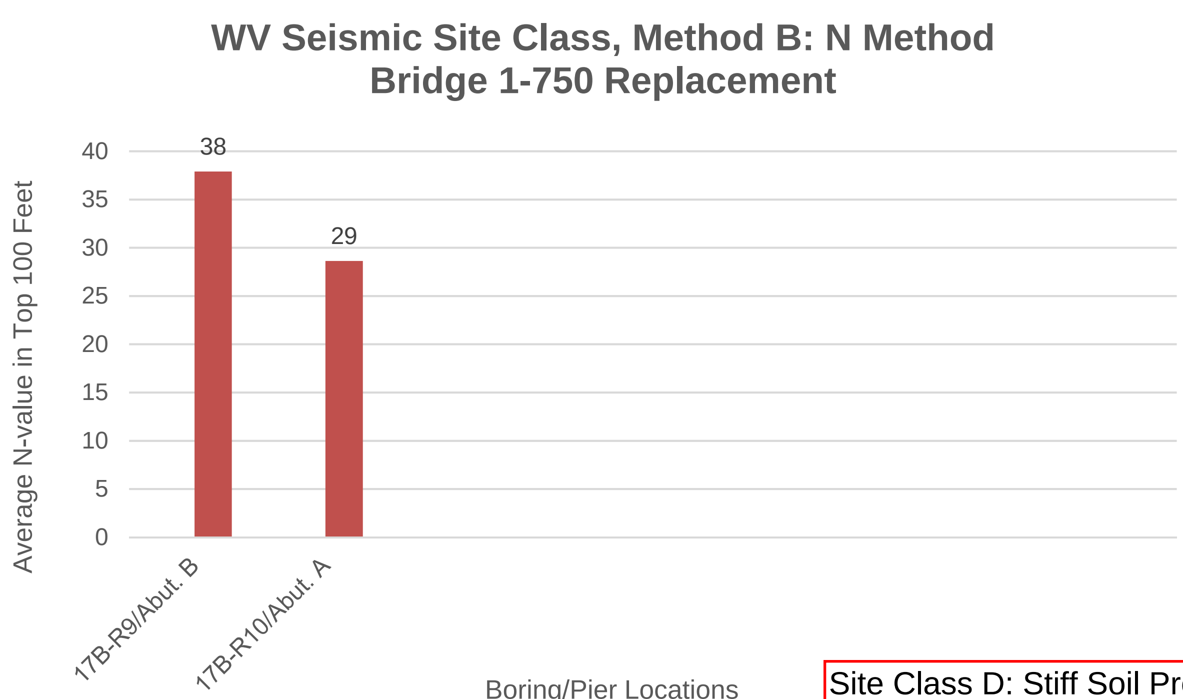
Remarks:

Delaware Department of Transportation
Materials and Research Laboratory

Figure

Appendix D



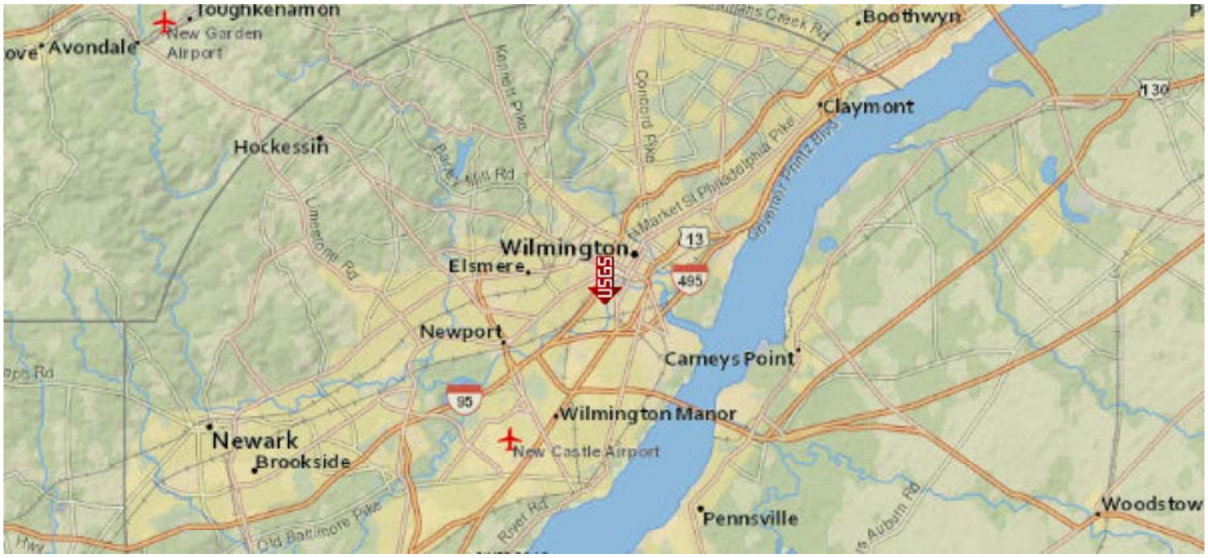


Site Class D: Stiff Soil Profile
15 bpf < N_average < 50 bpf

Design Maps Summary Report

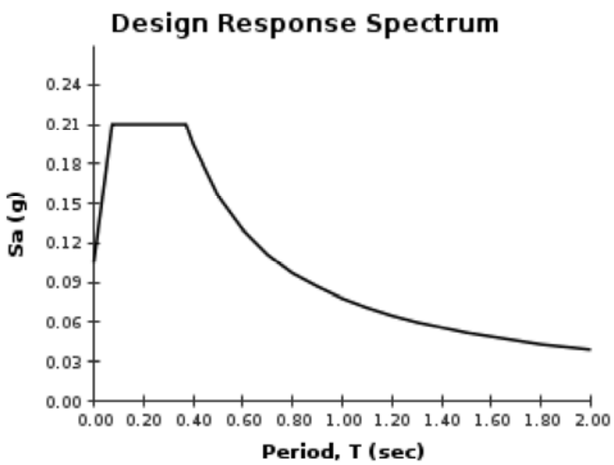
User–Specified Input

Report Title	Wilmington Viaduct Tue October 25, 2016 16:20:32 UTC
Building Code Reference Document	2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design (which utilizes USGS hazard data available in 2002)
Site Coordinates	39.737°N, 75.562°W
Site Soil Classification	Site Class D – "Stiff Soil"



USGS–Provided Output

PGA =	0.066 g	**A_s =**	0.105 g
S_S =	0.131 g	**S_{DS} =**	0.210 g
S_1 =	0.032 g	**S_{D1} =**	0.078 g



WHITMAN, REQUARDT & ASSOCIATES, LLP
801 South Caroline Street, Baltimore, Maryland 21231

SUBSURFACE DIAGRAM

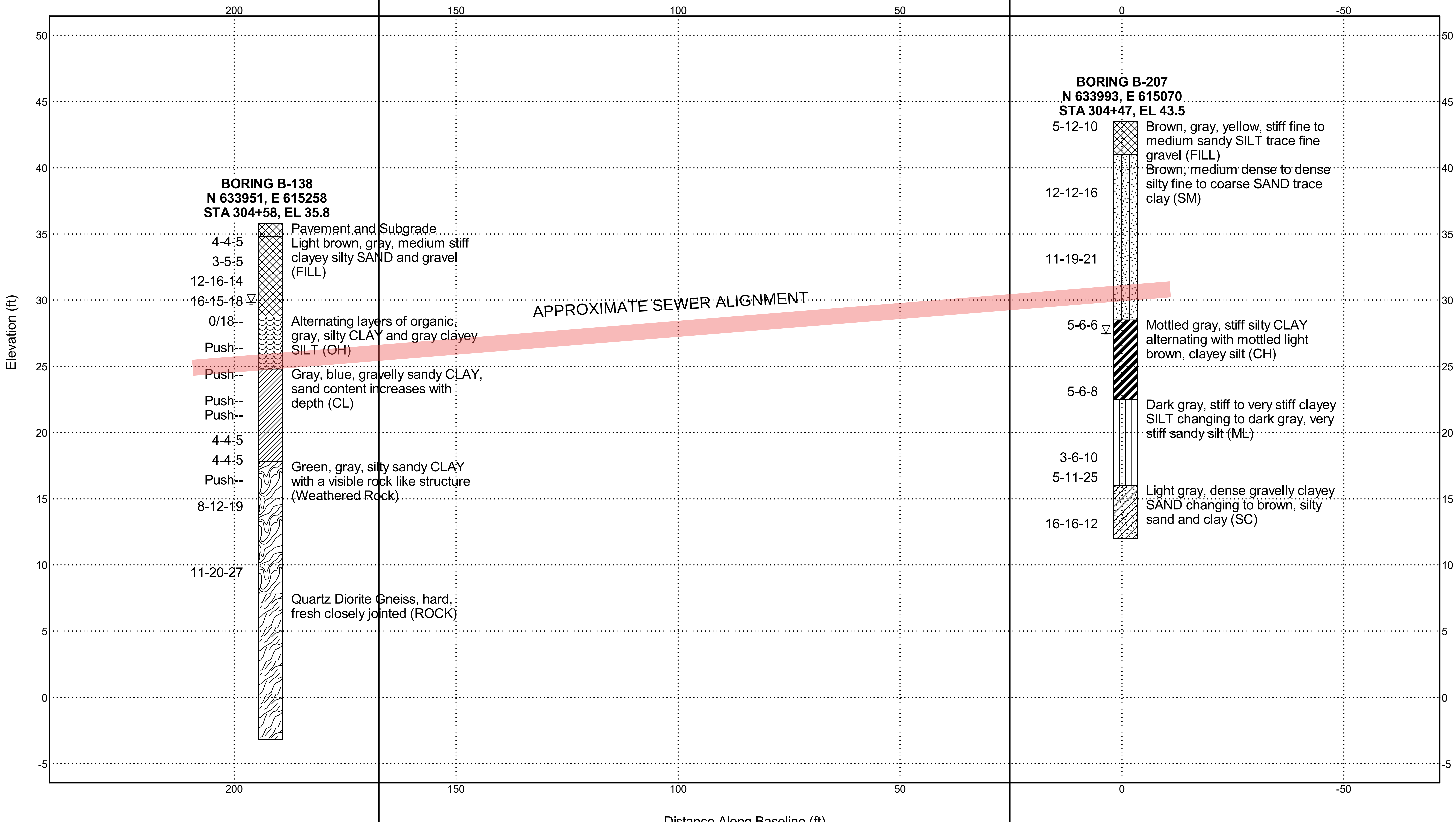
Fill (made ground)	USCS High Plasticity Organic Silt or Clay
USCS Low Plasticity Clay	Weathered Rock
Gneiss	USCS Silty Sand
USCS High Plasticity Clay	USCS Low Plasticity Silt
USCS Clayey Sand	

CLIENT Delaware Department of Transportation

Project Number 31987-004

PROJECT NAME Wilmington Viaduct Bridge - Del Dot#1711

PROJECT LOCATION Wilmington, Delaware



WRA CFT 2 WILMINGTON VIADUCT GINT - GDK COPY.GPJ JC WRA.GDT 2/18/19

Checked by: R.Fernós 12/4/17

```
TITLE PAGE
==============
PROJECT IDENTIFICATION:  Wilmington Viaduct Ramp D
Project Number:          31987-004
Client:                  DelDOT
Designer:                C. Troxel
Station Number:          S. Abutment
Description:             Check of MSE wall at Ramp D STA 304+00
```

See station cross-sections in Appendix A

```
Company's information:   Whitman, Requardt & Associates
                         801 S. Caroline St.
                         Baltimore, MD  21231

File path and name:  N:\31987-004\Engineering\Design\Geotechnical\Ramp
Relocation\Calculations\MSEW\304+00_LRFD.BEN

Original date and time of creating this file:  Wed Aug 02 14:33:04 2017
PROGRAM MODE:            ANALYSIS of a SIMPLE STRUCTURE using METAL STRIPS as reinforcing material.


SOIL DATA
REINFORCED SOIL
    Unit weight, gamma = 125.0 lb/ft ³
    Design value of internal angle of friction, phi = 34.0 °
RETAINED SOIL
    Unit weight, gamma = 120.0 lb/ft ³
    Design value of internal angle of friction, phi = 30.0 °
FOUNDATION SOIL (Considered as an equivalent uniform soil)
    Equivalent unit weight, gamma_equiv. = 120.0 lb/ft ³
    Equivalent internal angle of friction, phi_equiv. = 32.0 °
    Equivalent cohesion,  c_equiv. = 0.0 lb/ft ²
Water table does not affect bearing capacity


LATERAL EARTH PRESSURE COEFFICIENTS
Ka (internal stability) = 0.2827
      (if batter is less than 10°, Ka is calculated from eq. 15.  Otherwise, eq. 38 is utilized)
Ka (external stability) = 0.3333
      (if batter is less than 10°, Ka is calculated from eq. 16.  Otherwise, eq. 17 is utilized)
```

BEARING CAPACITY
Bearing capacity coefficients (calculated by MSEW): Nc = 0.00 N_gamma = 14.99

SEISMICITY
Maximum ground acceleration coefficient, Alpha_o = 0.11
Kae (Alpha_o > 0) = 0.4263 Kae (Alpha_o = 0) = 0.3333 d Kae = 0.0930 (see eq. 37 in DEMO 82)
Seismic soil-metal strip friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Metal strips (Analysis)

D A T A	Metal strip type #1	Metal strip type #2	Metal strip type #3	Metal strip type #4	Metal strip type #5
Yield strength of steel, Fy [ksi]	65.0	N/A	N/A	N/A	N/A
Gross width of strip, b [in]	2.0	N/A	N/A	N/A	N/A
Vertical spacing, Sv [ft]	Varies	N/A	N/A	N/A	N/A
Design cross section area, Ac [in²]	0.30	N/A	N/A	N/A	N/A
Ribbed steel strips.					
Uniformity Coefficient of reinforced soil, Cu = D60/D10 = 4.0					
Friction angle along reinforcement-soil interface, ro					
@ the top	60.97	N/A	N/A	N/A	N/A
@ 19.7 ft or below	32.00	N/A	N/A	N/A	N/A
Pullout resistance factor, F*					
@ the top	1.80	N/A	N/A	N/A	N/A
@ 19.7 ft or below	0.62	N/A	N/A	N/A	N/A
Scale-effect correct. factor, alpha	1.00	N/A	N/A	N/A	N/A

Variation of Lateral Earth Pressure Coefficient With Depth

Z	K / Ka
0 ft	1.70
3.3 ft	1.60
6.6 ft	1.55
9.8 ft	1.45
13.1 ft	1.35
16.4 ft	1.30

```
            19.7 ft      1.20
INPUT DATA: Facia and Connection  (Analysis)
=============================================
FACIA type:   Segmental precast concrete panels.
Depth of panel is 0.50 ft.  Horizontal distance to Center of Gravity of panel is 0.25 ft.
Average unit weight of panel is gamma_f = 152.00 lb/ft ³

          Z / Hd          To-static / Tmax
                     or   To-seismic / Tmd
        -----------------------------------
          0.00                1.00
          0.25                1.00
          0.50                1.00
          0.75                1.00
          1.00                1.00
```

D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Strength reduction at the connection, CRu = Fyc / Fy	0.90	N/A	N/A	N/A	N/A

```
INPUT DATA:  Geometry and Surcharge loads  (of a SIMPLE STRUCTURE)
=====================================================================
Design height, Hd      9.00 [ft]
{Embedded depth is E = 2.00 ft, and height above top of finished bottom grade is H = 7.00 ft }
Batter, omega           0.0 [deg]
Backslope,  beta        0.0 [deg]
Backslope rise          0.0 [ft]  Broken back equiv. angle, I = 0.00°  (see Fig. 25 in DEMO 82)
```

Exposed wall height = 7'
Embedment = 2'

```
U N I F O R M   S U R C H A R G E
Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 250.0 [lb/ft ²]
```

```
ANALYSIS: CALCULATED FACTORS (Static conditions)
=====================================================
Bearing capacity,  CDR = 1.49,
Foundation Interface: Direct sliding, CDR = 1.503, Eccentricity,   e/L = 0.1833.


ANALYSIS: CALCULATED FACTORS (Seismic conditions)
=====================================================
Bearing capacity,  CDR = 1.90,
Foundation Interface: Direct sliding, CDR = 1.201, Eccentricity,   e/L = 0.2449.


BEARING CAPACITY for GIVEN LAYOUT
=============================================
```

	STATIC	SEISMIC	UNITS
Ultimate bearing capacity, q-ult	3691	5169	[lb/ft ²]
Meyerhof stress, sigma_v	2478.8	2723.4	[lb/ft ²]
Eccentricity, e	0.84	1.13	[ft]
Eccentricity, e/L	0.105	0.141	
CDR calculated	1.49	1.90	
Base length	8.00	8.00	[ft]

Base length = Strap length = 8'

TITLE PAGE
==============

Checked by: R.Fernós 12/4/17

PROJECT IDENTIFICATION: Wilmington Viaduct Ramp D
Project Number: 31987-004
Client: DelDOT
Designer: C. Troxel
Station Number: S. Abutment
Description: Check of MSE wall at Ramp D STA 306+50

See station cross-sections in Appendix A

Company's information: Whitman, Requardt & Associates
801 S. Caroline St.
Baltimore, MD 21231

File path and name: N:\31987-004\Engineering\Design\Geotechnical\Ramp Relocation\Calculations\MSEW\306+50_LRFD.BEN

Original date and time of creating this file: Wed Aug 02 14:33:04 2017
PROGRAM MODE: ANALYSIS of a SIMPLE STRUCTURE using METAL STRIPS as reinforcing material.

SOIL DATA
REINFORCED SOIL
Unit weight, gamma = 125.0 lb/ft^3
Design value of internal angle of friction, phi = 34.0 °
RETAINED SOIL
Unit weight, gamma = 120.0 lb/ft^3
Design value of internal angle of friction, phi = 30.0 °
FOUNDATION SOIL (Considered as an equivalent uniform soil)
Equivalent unit weight, gamma_equiv. = 120.0 lb/ft^3
Equivalent internal angle of friction, phi_equiv. = 32.0 °
Equivalent cohesion, c_equiv. = 0.0 lb/ft^2
Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS
Ka (internal stability) = 0.2827
(if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Ka (external stability) = 0.3333
(if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 0.00 N_gamma = 14.99

SEISMICITY

Maximum ground acceleration coefficient, Alpha_o = 0.11

Kae (Alpha_o > 0) = 0.4263 Kae (Alpha_o = 0) = 0.3333 d Kae = 0.0930 (see eq. 37 in DEMO 82)

Seismic soil-metal strip friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Metal strips (Analysis)

D A T A	Metal strip type #1	Metal strip type #2	Metal strip type #3	Metal strip type #4	Metal strip type #5
Yield strength of steel, Fy [ksi]	65.0	N/A	N/A	N/A	N/A
Gross width of strip, b [in]	2.0	N/A	N/A	N/A	N/A
Vertical spacing, Sv [ft]	Varies	N/A	N/A	N/A	N/A
Design cross section area, Ac [in²]	0.30	N/A	N/A	N/A	N/A
Ribbed steel strips.					
Uniformity Coefficient of reinforced soil, Cu = D60/D10 = 4.0					
Friction angle along reinforcement-soil interface, ro					
@ the top	60.97	N/A	N/A	N/A	N/A
@ 19.7 ft or below	32.00	N/A	N/A	N/A	N/A
Pullout resistance factor, F*					
@ the top	1.80	N/A	N/A	N/A	N/A
@ 19.7 ft or below	0.62	N/A	N/A	N/A	N/A
Scale-effect correct. factor, alpha	1.00	N/A	N/A	N/A	N/A

Variation of Lateral Earth Pressure Coefficient With Depth

Z	K / Ka
0 ft	1.70
3.3 ft	1.60
6.6 ft	1.55
9.8 ft	1.45
13.1 ft	1.35
16.4 ft	1.30
19.7 ft	1.20

INPUT DATA: Facia and Connection (Analysis)
===
FACIA type: Segmental precast concrete panels.
Depth of panel is 0.50 ft. Horizontal distance to Center of Gravity of panel is 0.25 ft.
Average unit weight of panel is gamma_f = 152.00 lb/ft ³

Z / Hd	To-static / Tmax or To-seismic / Tmd
0.00	1.00
0.25	1.00
0.50	1.00
0.75	1.00
1.00	1.00

D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Strength reduction at the connection, CRu = Fyc / Fy	0.90	N/A	N/A	N/A	N/A

Exposed wall height = 10'
Embedment = 3'

INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)
===
Design height, Hd 13.00 [ft] {Embedded depth is E = 3.00 ft, and height above top of finished bottom grade is H = 10.00 ft }
Batter, omega 0.0 [deg]
Backslope, beta 0.0 [deg]
Backslope rise 0.0 [ft] Broken back equiv. angle, I = 0.00° (see Fig. 25 in DEMO 82)

U N I F O R M S U R C H A R G E
Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 250.0 [lb/ft ²]

```
ANALYSIS: CALCULATED FACTORS (Static conditions)
=====================================================
Bearing capacity,  CDR = 1.79,
Foundation Interface: Direct sliding, CDR = 1.749, Eccentricity,   e/L = 0.1466.


ANALYSIS: CALCULATED FACTORS (Seismic conditions)
=====================================================
Bearing capacity,  CDR = 2.30,
Foundation Interface: Direct sliding, CDR = 1.365, Eccentricity,   e/L = 0.2037.


BEARING CAPACITY for GIVEN LAYOUT
=============================================
                                          STATIC   SEISMIC    UNITS
Ultimate bearing capacity, q-ult            5746      8078   [lb/ft ²]
Meyerhof stress, sigma_v                  3212.8    3515.8   [lb/ft ²]
Eccentricity,  e                            1.09      1.51   [ft]
Eccentricity,  e/L                         0.091     0.126
CDR calculated                              1.79      2.30
Base length                                12.00     12.00   [ft]
```

Base length = Strap length = 12'

TITLE PAGE
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PROJECT IDENTIFICATION: Wilmington Viaduct Ramp D
Project Number: 31987-004
Client: DelDOT
Designer: C. Troxel
Station Number: S. Abutment
Description: Check of MSE wall at Ramp D STA 306+50 RT

See station cross-sections in Appendix A

Company's information: Whitman, Requardt & Assiciates
801 S. Caroline St.
Baltimore, MD 21231

File path and name: N:\31987-004\Engineering\Design\Geotechnical\Ramp Relocation\Calculations\MSEW\306+50 RT_LRFD.BEN

Original date and time of creating this file: Wed Aug 02 14:33:04 2017
PROGRAM MODE: ANALYSIS of a SIMPLE STRUCTURE using METAL STRIPS as reinforcing material.

SOIL DATA
REINFORCED SOIL
Unit weight, gamma = 125.0 lb/ft ^{3}
Design value of internal angle of friction, phi = 34.0 °
RETAINED SOIL
Unit weight, gamma = 120.0 lb/ft ^{3}
Design value of internal angle of friction, phi = 30.0 °
FOUNDATION SOIL (Considered as an equivalent uniform soil)
Equivalent unit weight, gamma_equiv. = 120.0 lb/ft ^{3}
Equivalent internal angle of friction, phi_equiv. = 32.0 °
Equivalent cohesion, c_equiv. = 0.0 lb/ft ^{2}
Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS
Ka (internal stability) = 0.2827
(if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Ka (external stability) = 0.3333
(if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY
Bearing capacity coefficients (calculated by MSEW): Nc = 0.00 N_gamma = 14.99

SEISMICITY
Maximum ground acceleration coefficient, Alpha_o = 0.11
Kae (Alpha_o > 0) = 0.4263 Kae (Alpha_o = 0) = 0.3333 d Kae = 0.0930 (see eq. 37 in DEMO 82)
Seismic soil-metal strip friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Metal strips (Analysis)

D A T A	Metal strip type #1	Metal strip type #2	Metal strip type #3	Metal strip type #4	Metal strip type #5
Yield strength of steel, Fy [ksi]	65.0	N/A	N/A	N/A	N/A
Gross width of strip, b [in]	2.0	N/A	N/A	N/A	N/A
Vertical spacing, Sv [ft]	Varies	N/A	N/A	N/A	N/A
Design cross section area, Ac [in²]	0.30	N/A	N/A	N/A	N/A
Ribbed steel strips.					
Uniformity Coefficient of reinforced soil, Cu = D60/D10 = 4.0					
Friction angle along reinforcement-soil interface, ro					
@ the top	60.97	N/A	N/A	N/A	N/A
@ 19.7 ft or below	32.00	N/A	N/A	N/A	N/A
Pullout resistance factor, F*					
@ the top	1.80	N/A	N/A	N/A	N/A
@ 19.7 ft or below	0.62	N/A	N/A	N/A	N/A
Scale-effect correct. factor, alpha	1.00	N/A	N/A	N/A	N/A

Variation of Lateral Earth Pressure Coefficient With Depth

Z	K / Ka
0 ft	1.70
3.3 ft	1.60
6.6 ft	1.55
9.8 ft	1.45
13.1 ft	1.35
16.4 ft	1.30
19.7 ft	1.20

```
INPUT DATA: Facia and Connection  (Analysis)
=============================================
FACIA type:   Segmental precast concrete panels.
Depth of panel is 0.50 ft.  Horizontal distance to Center of Gravity of panel is 0.25 ft.
Average unit weight of panel is gamma_f = 152.00 lb/ft ³

          Z / Hd          To-static / Tmax
                     or   To-seismic / Tmd
        -----------------------------------
          0.00                1.00
          0.25                1.00
          0.50                1.00
          0.75                1.00
          1.00                1.00


---------------------------------------------------------------------------------------------
  D  A  T  A  (for connection only)        Type #1   Type #2   Type #3   Type #4   Type #5
---------------------------------------------------------------------------------------------
Strength reduction at the connection,
                    CRu = Fyc / Fy            0.90      N/A       N/A       N/A       N/A
```

Exposed wall height = 14'
Embedment = 3'

```
INPUT DATA:  Geometry and Surcharge loads  (of a SIMPLE STRUCTURE)
=====================================================================
Design height, Hd     17.00 [ft] {Embedded depth is E = 3.00 ft, and height above top of finished
                                  bottom grade is H = 14.00 ft }
Batter, omega           0.0 [deg]
Backslope,  beta        0.0 [deg]
Backslope rise          0.0 [ft]  Broken back equiv. angle, I = 0.00°  (see Fig. 25 in DEMO 82)


U N I F O R M   S U R C H A R G E
Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 250.0 [lb/ft ²]
```

```
ANALYSIS: CALCULATED FACTORS (Static conditions)
=====================================================
Bearing capacity,  CDR = 1.75,
Foundation Interface: Direct sliding, CDR = 1.786, Eccentricity,   e/L = 0.1468.


ANALYSIS: CALCULATED FACTORS (Seismic conditions)
=====================================================
Bearing capacity,  CDR = 2.18,
Foundation Interface: Direct sliding, CDR = 1.373, Eccentricity,   e/L = 0.2094.


BEARING CAPACITY for GIVEN LAYOUT
=============================================
                                          STATIC   SEISMIC    UNITS
Ultimate bearing capacity, q-ult            7114      9861   [lb/ft ²]
Meyerhof stress, sigma_v                  4075.5    4523.7   [lb/ft ²]
Eccentricity,  e                            1.42      2.02   [ft]
Eccentricity,  e/L                         0.094     0.135
CDR calculated                              1.75      2.18
Base length                                15.00     15.00   [ft]
```

Base length = Strap length = 15'

TITLE PAGE
==============

Checked by: R.Fernós 12/4/17

```
PROJECT IDENTIFICATION:  Wilmington Viaduct Ramp D
Project Number:          31987-004
Client:                  DelDOT
Designer:                C. Troxel
Station Number:          S. Abutment
Description:             Check of MSE wall at Ramp D STA 307+50
```

See station cross-sections in Appendix A

```
Company's information:   Whitman, Requardt & Associates
                         801 S. Caroline St.
                         Baltimore, MD  21231


File path and name:  N:\31987-004\Engineering\Design\Geotechnical\Ramp
Relocation\Calculations\MSEW\307+50_LRFD.BEN

Original date and time of creating this file:  Wed Aug 02 14:33:04 2017
PROGRAM MODE:            ANALYSIS of a SIMPLE STRUCTURE using METAL STRIPS as reinforcing material.
```

SOIL DATA
REINFORCED SOIL
    Unit weight, gamma = 125.0 lb/ft ^{3}
    Design value of internal angle of friction, phi = 34.0 °
RETAINED SOIL
    Unit weight, gamma = 120.0 lb/ft ^{3}
    Design value of internal angle of friction, phi = 30.0 °
FOUNDATION SOIL (Considered as an equivalent uniform soil)
    Equivalent unit weight, gamma_equiv. = 120.0 lb/ft ^{3}
    Equivalent internal angle of friction, phi_equiv. = 32.0 °
    Equivalent cohesion, c_equiv. = 0.0 lb/ft ^{2}
Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS
Ka (internal stability) = 0.2827
            (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Ka (external stability) = 0.3333
            (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 0.00 N_gamma = 14.99

SEISMICITY

Maximum ground acceleration coefficient, Alpha_o = 0.11

Kae (Alpha_o > 0) = 0.4263 Kae (Alpha_o = 0) = 0.3333 d Kae = 0.0930 (see eq. 37 in DEMO 82)

Seismic soil-metal strip friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Metal strips (Analysis)

D A T A	Metal strip type #1	Metal strip type #2	Metal strip type #3	Metal strip type #4	Metal strip type #5
Yield strength of steel, Fy [ksi]	65.0	N/A	N/A	N/A	N/A
Gross width of strip, b [in]	2.0	N/A	N/A	N/A	N/A
Vertical spacing, Sv [ft]	Varies	N/A	N/A	N/A	N/A
Design cross section area, Ac [in²]	0.30	N/A	N/A	N/A	N/A

Ribbed steel strips.

Uniformity Coefficient of reinforced soil, Cu = D60/D10 = 4.0

Friction angle along reinforcement-soil interface, ro					
@ the top	60.97	N/A	N/A	N/A	N/A
@ 19.7 ft or below	32.00	N/A	N/A	N/A	N/A
Pullout resistance factor, F*					
@ the top	1.80	N/A	N/A	N/A	N/A
@ 19.7 ft or below	0.62	N/A	N/A	N/A	N/A
Scale-effect correct. factor, alpha	1.00	N/A	N/A	N/A	N/A

Variation of Lateral Earth Pressure Coefficient With Depth

Z	K / Ka
0 ft	1.70
3.3 ft	1.60
6.6 ft	1.55
9.8 ft	1.45
13.1 ft	1.35
16.4 ft	1.30
19.7 ft	1.20

INPUT DATA: Facia and Connection (Analysis)

FACIA type: Segmental precast concrete panels.
Depth of panel is 0.50 ft. Horizontal distance to Center of Gravity of panel is 0.25 ft.
Average unit weight of panel is gamma_f = 152.00 lb/ft ³

Z / Hd	To-static / Tmax or To-seismic / Tmd
0.00	1.00
0.25	1.00
0.50	1.00
0.75	1.00
1.00	1.00

D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Strength reduction at the connection, CRu = Fyc / Fy	0.90	N/A	N/A	N/A	N/A

Exposed wall height = 17'
Embedment = 3'

INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd 20.00 [ft] {Embedded depth is E = 3.00 ft, and height above top of finished bottom grade is H = 17.00 ft }
Batter, omega 0.0 [deg]
Backslope, beta 0.0 [deg]
Backslope rise 0.0 [ft] Broken back equiv. angle, I = 0.00° (see Fig. 25 in DEMO 82)

U N I F O R M S U R C H A R G E
Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 250.0 [lb/ft ²]

ANALYSIS: CALCULATED FACTORS (Static conditions)
===
Bearing capacity, CDR = 1.48,
Foundation Interface: Direct sliding, CDR = 1.676, Eccentricity,   e/L = 0.1706.

ANALYSIS: CALCULATED FACTORS (Seismic conditions)
===
Bearing capacity, CDR = 1.73,
Foundation Interface: Direct sliding, CDR = 1.278, Eccentricity,   e/L = 0.2467.

BEARING CAPACITY for GIVEN LAYOUT
===

	STATIC	SEISMIC	UNITS
Ultimate bearing capacity, q-ult	7262	9736	[lb/ft ^{2}]
Meyerhof stress, sigma_v	4911.1	5635.5	[lb/ft ^{2}]
Eccentricity, e	1.79	2.59	[ft]
Eccentricity, e/L	0.112	0.162	
CDR calculated	1.48	1.73	
Base length	16.00	16.00	[ft]

Base length = Strap length = 16'

TITLE PAGE
==============

Checked by: R.Fernós 12/4/17

```
PROJECT IDENTIFICATION:  Wilmington Viaduct Ramp D
Project Number:          31987-004
Client:                  DelDOT
Designer:                C. Troxel
Station Number:          S. Abutment
Description:             Check of MSE wall at Ramp D STA 308+50

Company's information:   Whitman, Requardt & Associates
                         801 S. Caroline St.
                         Baltimore, MD  21231


File path and name:  N:\31987-004\Engineering\Design\Geotechnical\Ramp
Relocation\Calculations\MSEW\308+50_LRFD.BEN

Original date and time of creating this file:  Wed Aug 02 14:33:04 2017
PROGRAM MODE:            ANALYSIS of a SIMPLE STRUCTURE using METAL STRIPS as reinforcing material.
```

See station cross-sections in Appendix A

SOIL DATA
REINFORCED SOIL
    Unit weight, gamma = 125.0 lb/ft ^{3}
    Design value of internal angle of friction, phi = 34.0 °
RETAINED SOIL
    Unit weight, gamma = 120.0 lb/ft ^{3}
    Design value of internal angle of friction, phi = 30.0 °
FOUNDATION SOIL (Considered as an equivalent uniform soil)
    Equivalent unit weight, gamma_equiv. = 120.0 lb/ft ^{3}
    Equivalent internal angle of friction, phi_equiv. = 32.0 °
    Equivalent cohesion, c_equiv. = 0.0 lb/ft ^{2}
Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS
Ka (internal stability) = 0.2827
            (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Ka (external stability) = 0.3333
            (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY
Bearing capacity coefficients (calculated by MSEW): Nc = 0.00 N_gamma = 14.99

SEISMICITY
Maximum ground acceleration coefficient, Alpha_o = 0.11
Kae (Alpha_o > 0) = 0.4263 Kae (Alpha_o = 0) = 0.3333 d Kae = 0.0930 (see eq. 37 in DEMO 82)
Seismic soil-metal strip friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Metal strips (Analysis)

D A T A	Metal strip type #1	Metal strip type #2	Metal strip type #3	Metal strip type #4	Metal strip type #5
Yield strength of steel, Fy [ksi]	65.0	N/A	N/A	N/A	N/A
Gross width of strip, b [in]	2.0	N/A	N/A	N/A	N/A
Vertical spacing, Sv [ft]	Varies	N/A	N/A	N/A	N/A
Design cross section area, Ac [in²]	0.30	N/A	N/A	N/A	N/A
Ribbed steel strips.					
Uniformity Coefficient of reinforced soil, Cu = D60/D10 = 4.0					
Friction angle along reinforcement-soil interface, ro					
@ the top	60.97	N/A	N/A	N/A	N/A
@ 19.7 ft or below	32.00	N/A	N/A	N/A	N/A
Pullout resistance factor, F*					
@ the top	1.80	N/A	N/A	N/A	N/A
@ 19.7 ft or below	0.62	N/A	N/A	N/A	N/A
Scale-effect correct. factor, alpha	1.00	N/A	N/A	N/A	N/A

Variation of Lateral Earth Pressure Coefficient With Depth

Z	K / Ka
0 ft	1.70
3.3 ft	1.60
6.6 ft	1.55
9.8 ft	1.45
13.1 ft	1.35
16.4 ft	1.30
19.7 ft	1.20

```
INPUT DATA: Facia and Connection  (Analysis)
============================================
FACIA type:   Segmental precast concrete panels.
Depth of panel is 0.50 ft.  Horizontal distance to Center of Gravity of panel is 0.25 ft.
Average unit weight of panel is gamma_f = 152.00 lb/ft ³

          Z / Hd          To-static / Tmax
                     or   To-seismic / Tmd
        ----------------------------------
          0.00                1.00
          0.25                1.00
          0.50                1.00
          0.75                1.00
          1.00                1.00



-------------------------------------------------------------------------------------------
  D  A  T  A  (for connection only)       Type #1   Type #2   Type #3   Type #4   Type #5
-------------------------------------------------------------------------------------------
Strength reduction at the connection,
                   CRu = Fyc / Fy            0.90      N/A       N/A       N/A       N/A
```

Exposed wall height = 23'
Embedment = 3'

```
INPUT DATA:  Geometry and Surcharge loads  (of a SIMPLE STRUCTURE)
=====================================================================
Design height, Hd     26.00 [ft] {Embedded depth is E = 3.00 ft, and height above top of finished
                                  bottom grade is H = 23.00 ft }
Batter, omega           0.0 [deg]
Backslope,  beta        0.0 [deg]
Backslope rise          0.0 [ft]  Broken back equiv. angle, I = 0.00°  (see Fig. 25 in DEMO 82)


U N I F O R M   S U R C H A R G E
Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 250.0 [lb/ft ²]
```

ANALYSIS: CALCULATED FACTORS (Static conditions)
===
Bearing capacity, CDR = 1.27,
Foundation Interface: Direct sliding, CDR = 1.603, Eccentricity,   e/L = 0.1918.

ANALYSIS: CALCULATED FACTORS (Seismic conditions)
===
Bearing capacity, CDR = 1.36,
Foundation Interface: Direct sliding, CDR = 1.209, Eccentricity,   e/L = 0.2830.

BEARING CAPACITY for GIVEN LAYOUT
===

	STATIC	SEISMIC	UNITS
Ultimate bearing capacity, q-ult	8238	10574	[lb/ft ²]
Meyerhof stress, sigma_v	6506.3	7797.9	[lb/ft ²]
Eccentricity, e	2.45	3.62	[ft]
Eccentricity, e/L	0.129	0.191	
CDR calculated	1.27	1.36	
Base length	19.00	19.00	[ft]

Base length = Strap length = 19'

Checked by: R.Fernós 12/4/17

TITLE PAGE
==============

```
PROJECT IDENTIFICATION:  Wilmington Viaduct Ramp D
Project Number:          31987-004
Client:                  DelDOT
Designer:                C. Troxel
Station Number:          S. Abutment
Description:             Check of MSE wall at Ramp D STA 308+60 (Abutment 40D)

Company's information:   Whitman, Requardt & Associates
                         801 S. Caroline St.
                         Baltimore, MD  21231

File path and name:  N:\31987-004\Engineering\Design\Geotechnical\Ramp
Relocation\Calculations\MSEW\308+60_LRFD.BEN

Original date and time of creating this file:  Wed Aug 02 14:33:04 2017
PROGRAM MODE:            ANALYSIS of a SIMPLE STRUCTURE using METAL STRIPS as reinforcing material.
```

See station cross-sections in Appendix A

SOIL DATA
REINFORCED SOIL
    Unit weight, gamma = 125.0 lb/ft ^{3}
    Design value of internal angle of friction, phi = 34.0 °
RETAINED SOIL
    Unit weight, gamma = 120.0 lb/ft ^{3}
    Design value of internal angle of friction, phi = 30.0 °
FOUNDATION SOIL (Considered as an equivalent uniform soil)
    Equivalent unit weight, gamma_equiv. = 120.0 lb/ft ^{3}
    Equivalent internal angle of friction, phi_equiv. = 32.0 °
    Equivalent cohesion, c_equiv. = 0.0 lb/ft ^{2}
Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS
Ka (internal stability) = 0.2827
      (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Ka (external stability) = 0.3333
      (if batter is less than 10°, Ka is calculated from eq. 16. Otherwise, eq. 17 is utilized)

BEARING CAPACITY
Bearing capacity coefficients (calculated by MSEW): Nc = 0.00 N_gamma = 14.99

SEISMICITY
Maximum ground acceleration coefficient, Alpha_o = 0.11
Kae (Alpha_o > 0) = 0.4263 Kae (Alpha_o = 0) = 0.3333 d Kae = 0.0930 (see eq. 37 in DEMO 82)
Seismic soil-metal strip friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Metal strips (Analysis)

D A T A	Metal strip type #1	Metal strip type #2	Metal strip type #3	Metal strip type #4	Metal strip type #5
Yield strength of steel, Fy [ksi]	65.0	N/A	N/A	N/A	N/A
Gross width of strip, b [in]	2.0	N/A	N/A	N/A	N/A
Vertical spacing, Sv [ft]	Varies	N/A	N/A	N/A	N/A
Design cross section area, Ac [in²]	0.30	N/A	N/A	N/A	N/A
Ribbed steel strips.					
Uniformity Coefficient of reinforced soil, Cu = D60/D10 = 4.0					
Friction angle along reinforcement-soil interface, ro					
@ the top	60.97	N/A	N/A	N/A	N/A
@ 19.7 ft or below	32.00	N/A	N/A	N/A	N/A
Pullout resistance factor, F*					
@ the top	1.80	N/A	N/A	N/A	N/A
@ 19.7 ft or below	0.62	N/A	N/A	N/A	N/A
Scale-effect correct. factor, alpha	1.00	N/A	N/A	N/A	N/A

Variation of Lateral Earth Pressure Coefficient With Depth

Z	K / Ka
0 ft	1.70
3.3 ft	1.60
6.6 ft	1.55
9.8 ft	1.45
13.1 ft	1.35
16.4 ft	1.30
19.7 ft	1.20

```
INPUT DATA: Facia and Connection  (Analysis)
=============================================
FACIA type:   Segmental precast concrete panels.
Depth of panel is 0.50 ft.  Horizontal distance to Center of Gravity of panel is 0.25 ft.
Average unit weight of panel is gamma_f = 152.00 lb/ft ³
```

Z / Hd	To-static / Tmax or To-seismic / Tmd
0.00	1.00
0.25	1.00
0.50	1.00
0.75	1.00
1.00	1.00

D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Strength reduction at the connection, CRu = Fyc / Fy	0.90	N/A	N/A	N/A	N/A

Exposed wall height = 26'
Embedment = 3'

```
INPUT DATA:  Geometry and Surcharge loads  (of a SIMPLE STRUCTURE)
=====================================================================
Design height, Hd     29.00 [ft] {Embedded depth is E = 3.00 ft, and height above top of finished
                                  bottom grade is H = 26.00 ft }
Batter, omega           0.0 [deg]
Backslope,  beta        0.0 [deg]
Backslope rise          0.0 [ft]  Broken back equiv. angle, I = 0.00°  (see Fig. 25 in DEMO 82)


U N I F O R M   S U R C H A R G E
Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 250.0 [lb/ft ²]
```

ANALYSIS: CALCULATED FACTORS (Static conditions)
===
Bearing capacity, CDR = 1.55,
Foundation Interface: Direct sliding, CDR = 1.768, Eccentricity, e/L = 0.1592.

ANALYSIS: CALCULATED FACTORS (Seismic conditions)
===
Bearing capacity, CDR = 1.79,
Foundation Interface: Direct sliding, CDR = 1.328, Eccentricity, e/L = 0.2366.

BEARING CAPACITY for GIVEN LAYOUT

	STATIC	SEISMIC	UNITS
Ultimate bearing capacity, q-ult	10536	14032	[lb/ft ²]
Meyerhof stress, sigma_v	6803.9	7859.9	[lb/ft ²]
Eccentricity, e	2.49	3.70	[ft]
Eccentricity, e/L	0.108	0.161	
CDR calculated	1.55	1.79	
Base length	23.00	23.00	[ft]

Base length = Strap length = 23'

Checked by: R.Fernós 2/1/18

```
TITLE PAGE
==============
PROJECT IDENTIFICATION:  Wilmington Viaduct Ramp D
Project Number:          31987
                         004
Client:                  DelDOT
Designer:                C. Troxel
Station Number:          S. Abutment
Description:             Check of MSE wall at Ramp D STA 304+00 with #57 stone wall fill

Company's information:   Whitman, Requardt & Associates
                         801 S. Caroline St.
                         Baltimore, MD  21231

File path and name:  N:\31987-004\Engineering\Design\Geotechnical\Ramp
Relocation\Calculations\MSEW\304+00_LRFD_57 stone.BEN

Original date and time of creating this file:  Wed Aug 02 14:33:04 2017
PROGRAM MODE:            ANALYSIS of a SIMPLE STRUCTURE using METAL STRIPS as reinforcing material.
```

Check of MSE wall with #57 stone back fill. Performed on section with lowest CDR for Direct Sliding.

Unit weight of #57 stone

```
SOIL DATA
REINFORCED SOIL
    Unit weight, gamma = 90.0 lb/ft ³
    Design value of internal angle of friction, phi = 34.0 °
RETAINED SOIL
    Unit weight, gamma = 90.0 lb/ft ³
    Design value of internal angle of friction, phi = 30.0 °
FOUNDATION SOIL (Considered as an equivalent uniform soil)
    Equivalent unit weight, gamma_equiv. = 120.0 lb/ft ³
    Equivalent internal angle of friction, phi_equiv. = 32.0 °
    Equivalent cohesion,  c_equiv. = 0.0 lb/ft ²
Water table does not affect bearing capacity

  LATERAL EARTH PRESSURE COEFFICIENTS
Ka (internal stability) = 0.2827
      (if batter is less than 10°, Ka is calculated from eq. 15.  Otherwise, eq. 38 is utilized)
Ka (external stability) = 0.3333
      (if batter is less than 10°, Ka is calculated from eq. 16.  Otherwise, eq. 17 is utilized)
```

BEARING CAPACITY
Bearing capacity coefficients (calculated by MSEW): Nc = 0.00 N_gamma = 14.99

SEISMICITY
Maximum ground acceleration coefficient, Alpha_o = 0.11
Kae (Alpha_o > 0) = 0.4263 Kae (Alpha_o = 0) = 0.3333 d Kae = 0.0930 (see eq. 37 in DEMO 82)
Seismic soil-metal strip friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Metal strips (Analysis)

D A T A	Metal strip type #1	Metal strip type #2	Metal strip type #3	Metal strip type #4	Metal strip type #5
Yield strength of steel, Fy [ksi]	65.0	N/A	N/A	N/A	N/A
Gross width of strip, b [in]	2.0	N/A	N/A	N/A	N/A
Vertical spacing, Sv [ft]	Varies	N/A	N/A	N/A	N/A
Design cross section area, Ac [in²]	0.30	N/A	N/A	N/A	N/A
Ribbed steel strips.					
Uniformity Coefficient of reinforced soil, Cu = D60/D10 = 4.0					
Friction angle along reinforcement-soil interface, ro					
@ the top	60.97	N/A	N/A	N/A	N/A
@ 19.7 ft or below	32.00	N/A	N/A	N/A	N/A
Pullout resistance factor, F*					
@ the top	1.80	N/A	N/A	N/A	N/A
@ 19.7 ft or below	0.62	N/A	N/A	N/A	N/A
Scale-effect correct. factor, alpha	1.00	N/A	N/A	N/A	N/A

Variation of Lateral Earth Pressure Coefficient With Depth

Z	K / Ka
0 ft	1.70
3.3 ft	1.60
6.6 ft	1.55
9.8 ft	1.45
13.1 ft	1.35
16.4 ft	1.30
19.7 ft	1.20

```
INPUT DATA: Facia and Connection (Analysis)
=============================================
FACIA type:   Segmental precast concrete panels.
Depth of panel is 0.50 ft.  Horizontal distance to Center of Gravity of panel is 0.25 ft.
Average unit weight of panel is gamma_f = 152.00 lb/ft ³

          Z / Hd          To-static / Tmax
                     or   To-seismic / Tmd
        -----------------------------------
          0.00                1.00
          0.25                1.00
          0.50                1.00
          0.75                1.00
          1.00                1.00



---------------------------------------------------------------------------------------------
  D  A  T  A  (for connection only)        Type #1   Type #2   Type #3   Type #4   Type #5
---------------------------------------------------------------------------------------------
Strength reduction at the connection,
                    CRu = Fyc / Fy            0.90      N/A       N/A       N/A       N/A



INPUT DATA:  Geometry and Surcharge loads  (of a SIMPLE STRUCTURE)
=====================================================================
Design height, Hd      9.00 [ft] {Embedded depth is E = 2.00 ft, and height above top of finished
                                  bottom grade is H = 7.00 ft }
Batter, omega           0.0 [deg]
Backslope,  beta        0.0 [deg]
Backslope rise          0.0 [ft]  Broken back equiv. angle, I = 0.00°  (see Fig. 25 in DEMO 82)


U N I F O R M   S U R C H A R G E
Uniformly distributed dead load is 0.0 [lb/ft ²], and live load is 250.0 [lb/ft ²]
```

Checked by: R.Fernós 2/1/18

```
ANALYSIS: CALCULATED FACTORS (Static conditions)
=====================================================
Bearing capacity,  CDR = 1.80,
Foundation Interface: Direct sliding, CDR = 1.292, Eccentricity,   e/L = 0.2194.
```

Sliding okay

```
ANALYSIS: CALCULATED FACTORS (Seismic conditions)
=====================================================
Bearing capacity,  CDR = 2.32,
Foundation Interface: Direct sliding, CDR = 1.060, Eccentricity,   e/L = 0.2817.
```

```
BEARING CAPACITY for GIVEN LAYOUT
=============================================
```

	STATIC	SEISMIC	UNITS
Ultimate bearing capacity, q-ult	3591	5050	[lb/ft ^{2}]
Meyerhof stress, sigma_v	1993.9	2181.3	[lb/ft ^{2}]
Eccentricity, e	0.93	1.19	[ft]
Eccentricity, e/L	0.116	0.149	
CDR calculated	1.80	2.32	
Base length	8.00	8.00	[ft]

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.83

Critical Circle: Xc = -3.10[ft], Yc = 15.25[ft], R = 16.80[ft]. (Number of slices used = 60)

STA 304+00 RT
Static

Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

Checked by: R.Fernós 12/4/17

Minimum Factor of Safety = 1.68

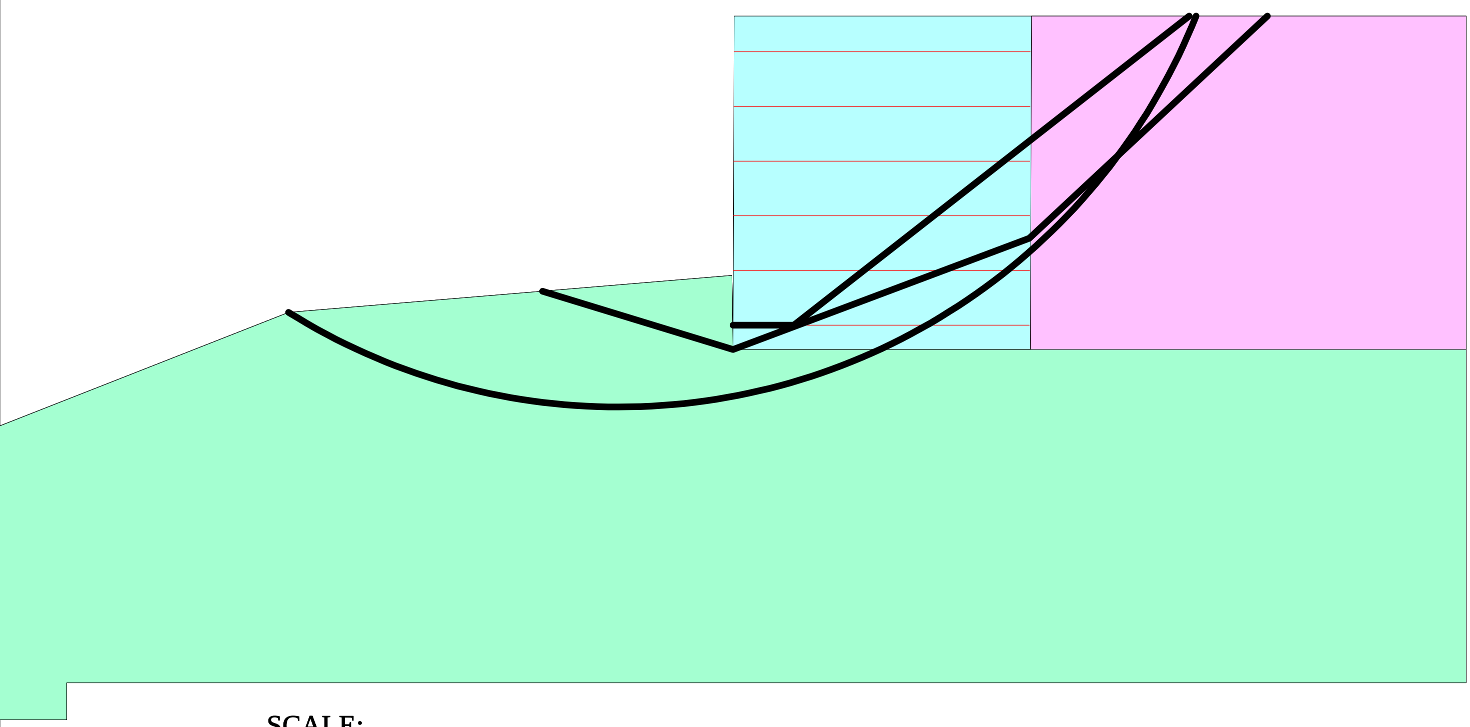
Critical Two-Part Wedge: (Xa = 0.00, Ya = 0.66) [ft]
(Xb = 1.64, Yb = 0.66) [ft]
(Xc = 12.32, Yc = 9.00) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 34.86 [degrees]

Three-Part Wedge Stability Analysis

Minimum Factor of Safety = 1.74

Critical Three-Part Wedge: (X2 = -5.14, Y2 = 1.57) [ft]
(X-left = 0.00, Y-left = 0.00) [ft]
(X-right = 8.00, Y-right = 3.00) [ft]
(X1 = 14.43, Y1 = 9.00) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 24.95 [degrees]

REINFORCEMENT LAYOUT: DRAWING



SCALE:

0 2 4 6 8 10 [ft]

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.63

Critical Circle: Xc = -10.26[ft], Yc = 27.82[ft], R = 31.51[ft]. (Number of slices used = 60)

STA 304+00 RT
Seismic

Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

Checked by: R.Fernós 12/4/17

Minimum Factor of Safety = 1.56

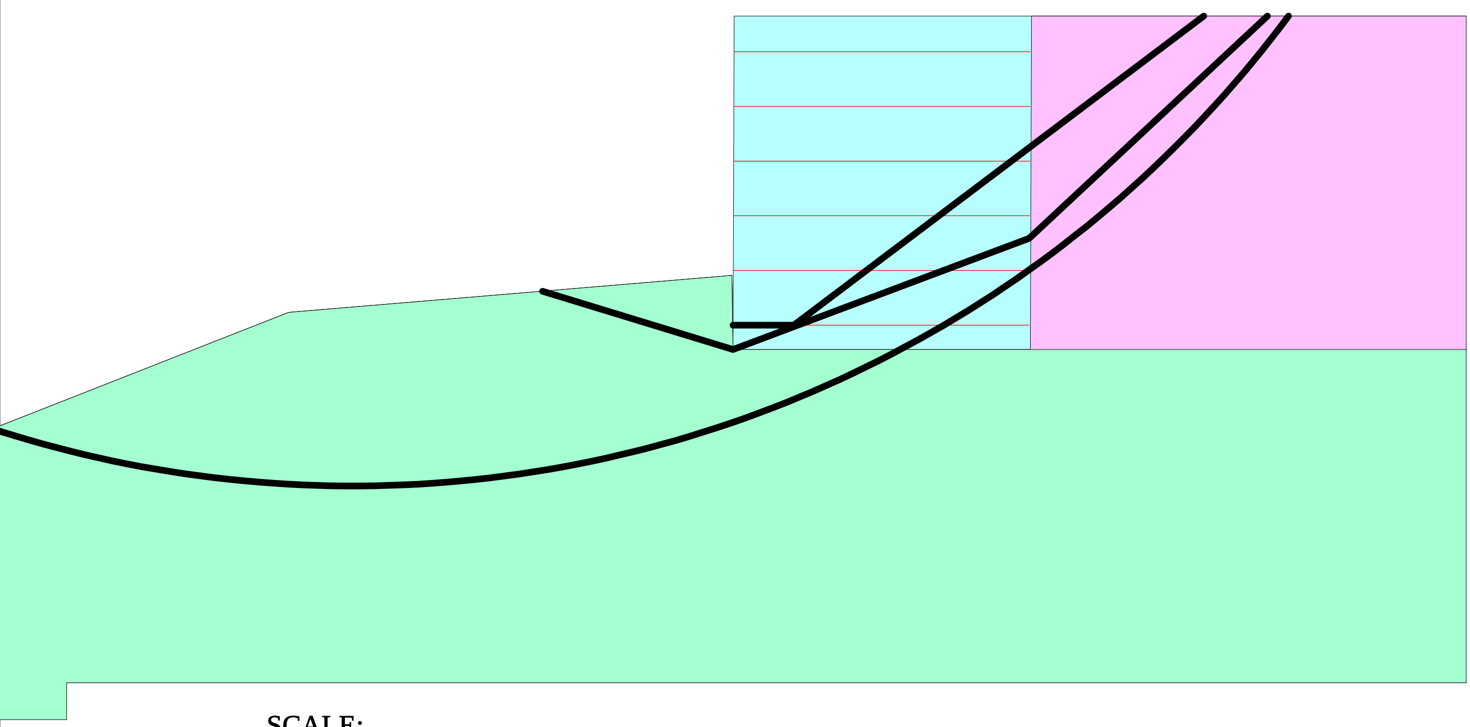
Critical Two-Part Wedge: (Xa = 0.00, Ya = 0.66) [ft]
(Xb = 1.64, Yb = 0.66) [ft]
(Xc = 12.71, Yc = 9.00) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 34.05 [degrees]

Three-Part Wedge Stability Analysis

Minimum Factor of Safety = 1.59

Critical Three-Part Wedge: (X2 = -5.14, Y2 = 1.57) [ft]
(X-left = 0.00, Y-left = 0.00) [ft]
(X-right = 8.00, Y-right = 3.00) [ft]
(X1 = 14.43, Y1 = 9.00) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 24.80 [degrees]

REINFORCEMENT LAYOUT: DRAWING



SCALE:

0 2 4 6 8 10 [ft]

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.64

Critical Circle: Xc = -9.14[ft], Yc = 29.19[ft], R = 33.34[ft]. (Number of slices used = 59)

STA 306+50 LT
Static

Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

Checked by: R.Fernós 12/4/17

Minimum Factor of Safety = 1.68

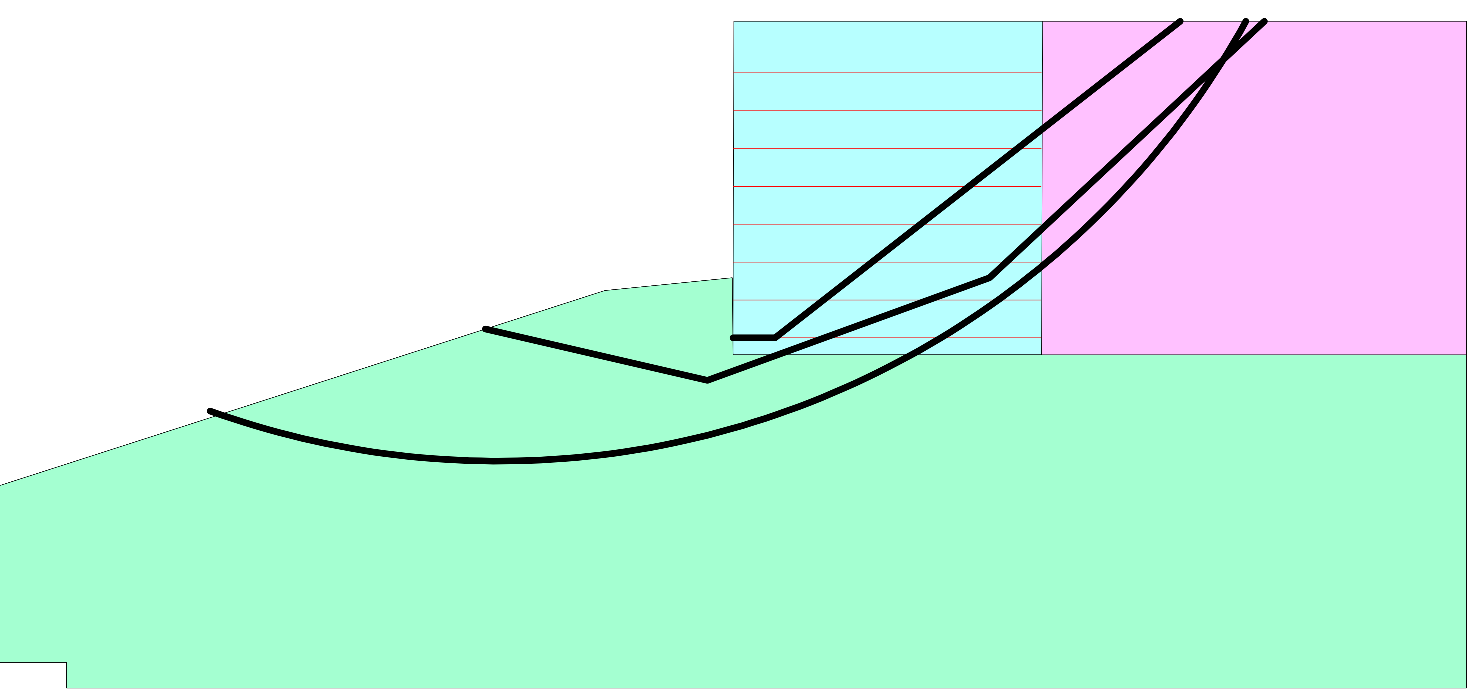
Critical Two-Part Wedge: (Xa = 0.00, Ya = 0.66) [ft]
(Xb = 1.64, Yb = 0.66) [ft]
(Xc = 17.44, Yc = 13.00) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 36.09 [degrees]

Three-Part Wedge Stability Analysis

Minimum Factor of Safety = 1.77

Critical Three-Part Wedge: (X2 = -9.66, Y2 = 1.00) [ft]
(X-left = -1.00, Y-left = -1.00) [ft]
(X-right = 10.00, Y-right = 3.00) [ft]
(X1 = 20.72, Y1 = 13.00) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 24.43 [degrees]

REINFORCEMENT LAYOUT: DRAWING



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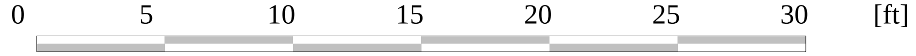


CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

STA 306+50 LT
Static

Minimum Factor of Safety = 1.48
Critical Circle: Xc = -9.48[ft], Yc = 30.06[ft], R = 34.06[ft]. (Number of slices used = 59)

Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

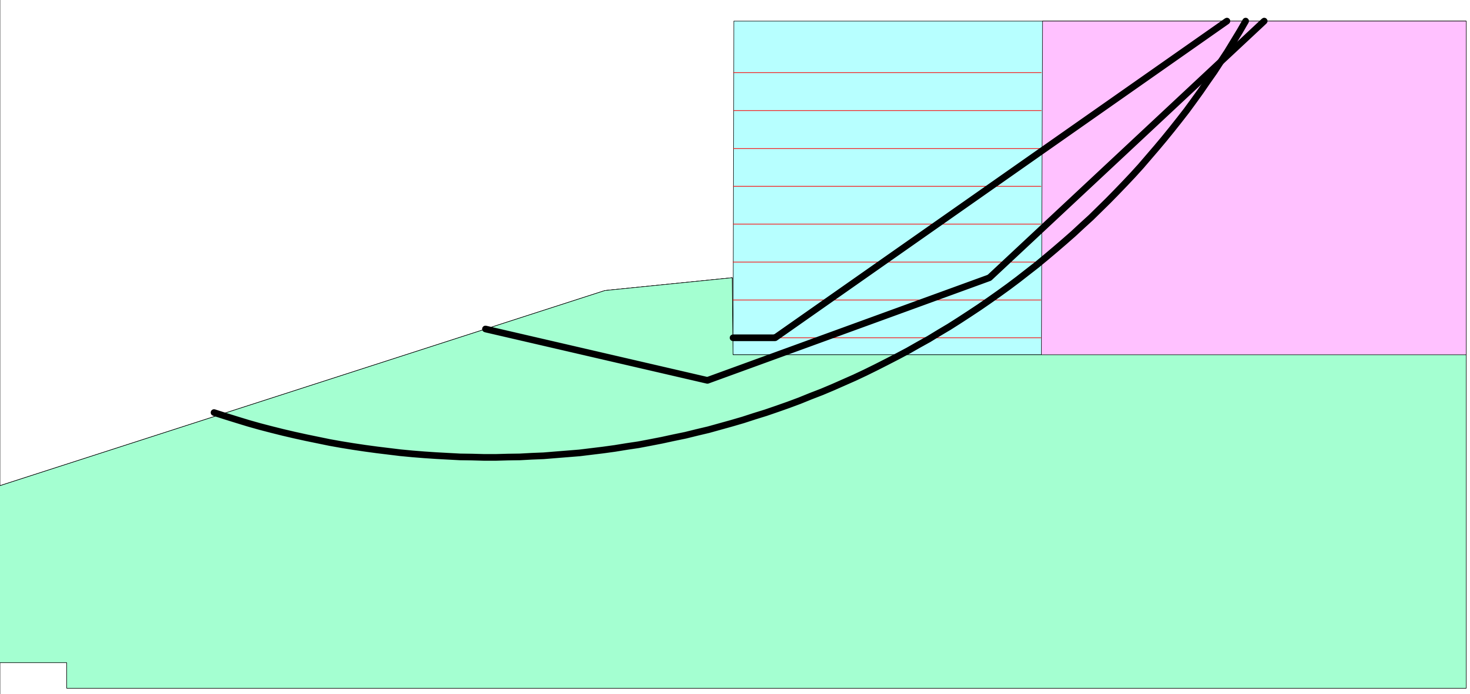
Checked by: R.Fernós 12/4/17

Minimum Factor of Safety = 1.56
Critical Two-Part Wedge: (Xa = 0.00, Ya = 0.66) [ft]
(Xb = 1.64, Yb = 0.66) [ft]
(Xc = 19.27, Yc = 13.00) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 33.47 [degrees]

Three-Part Wedge Stability Analysis

Minimum Factor of Safety = 1.60
Critical Three-Part Wedge: (X2 = -9.66, Y2 = 1.00) [ft]
(X-left = -1.00, Y-left = -1.00) [ft]
(X-right = 10.00, Y-right = 3.00) [ft]
(X1 = 20.72, Y1 = 13.00) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 24.32 [degrees]

REINFORCEMENT LAYOUT: DRAWING



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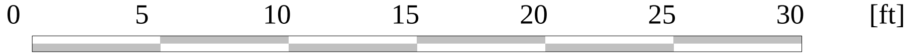


CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.45

Critical Circle: Xc = -14.18[ft], Yc = 35.01[ft], R = 43.12[ft]. (Number of slices used = 59)

STA 306+50 RT
Static

Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

Minimum Factor of Safety = 1.78

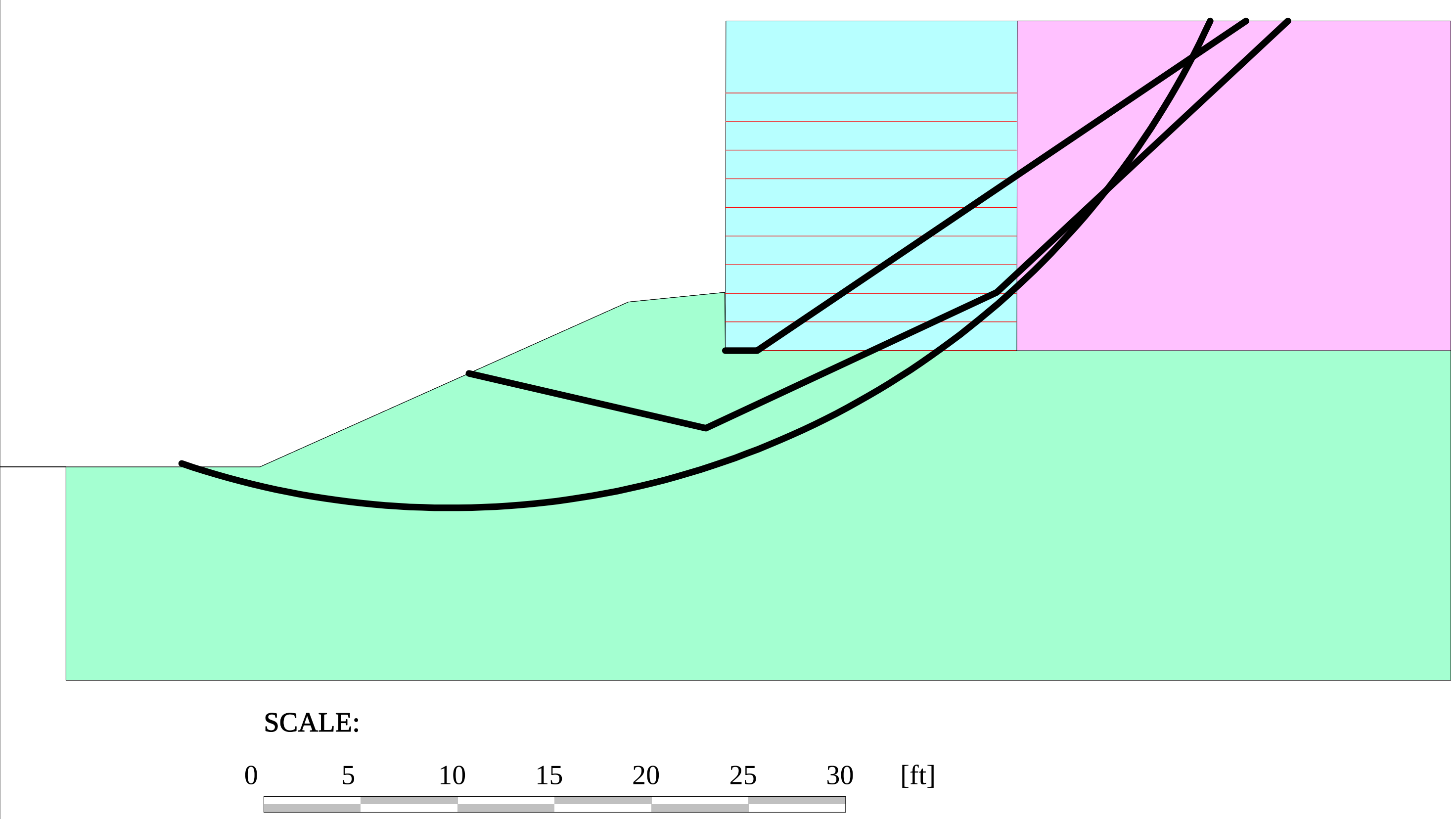
Critical Two-Part Wedge: (Xa = 0.00, Ya = 0.00) [ft]
(Xb = 1.64, Yb = 0.00) [ft]
(Xc = 26.84, Yc = 17.00) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 33.30 [degrees]

Three-Part Wedge Stability Analysis

Minimum Factor of Safety = 1.60

Critical Three-Part Wedge: (X2 = -13.22, Y2 = -1.18) [ft]
(X-left = -1.00, Y-left = -4.00) [ft]
(X-right = 14.00, Y-right = 3.00) [ft]
(X1 = 29.01, Y1 = 17.00) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 24.48 [degrees]

REINFORCEMENT LAYOUT: DRAWING



CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.31

Critical Circle: Xc = -14.64[ft], Yc = 36.06[ft], R = 43.99[ft]. (Number of slices used = 58)

STA 306+50 RT
Seismic

Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

Minimum Factor of Safety = 1.64

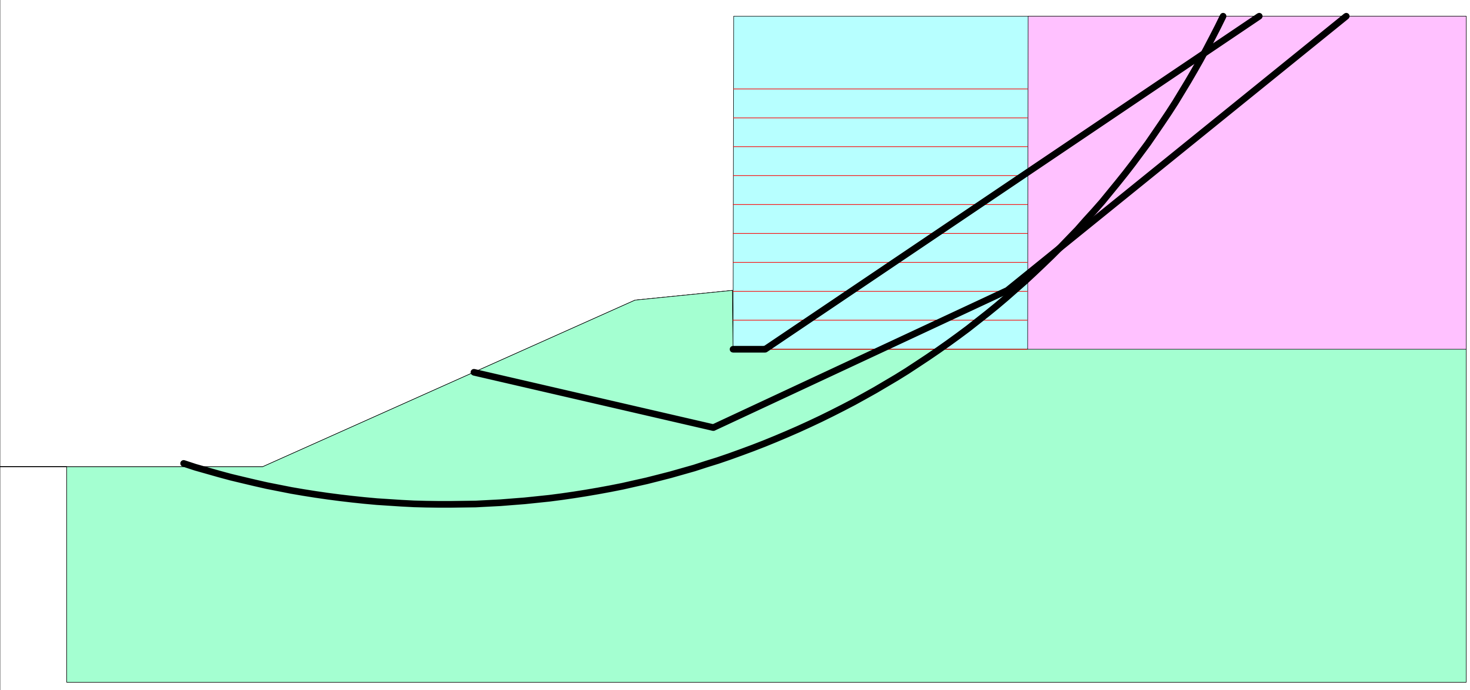
Critical Two-Part Wedge: (Xa = 0.00, Ya = 0.00) [ft]
(Xb = 1.64, Yb = 0.00) [ft]
(Xc = 26.84, Yc = 17.00) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 33.29 [degrees]

Three-Part Wedge Stability Analysis

Minimum Factor of Safety = 1.44

Critical Three-Part Wedge: (X2 = -13.22, Y2 = -1.18) [ft]
(X-left = -1.00, Y-left = -4.00) [ft]
(X-right = 14.00, Y-right = 3.00) [ft]
(X1 = 31.29, Y1 = 17.00) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 23.52 [degrees]

REINFORCEMENT LAYOUT: DRAWING



SCALE:

0 5 10 15 20 25 30 [ft]

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.52

STA 307+50 LT
Static

Critical Circle: Xc = -15.75[ft], Yc = 39.64[ft], R = 47.50[ft]. (Number of slices used = 60)

Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

Checked by: R.Fernós 12/4/17

Minimum Factor of Safety = 1.58

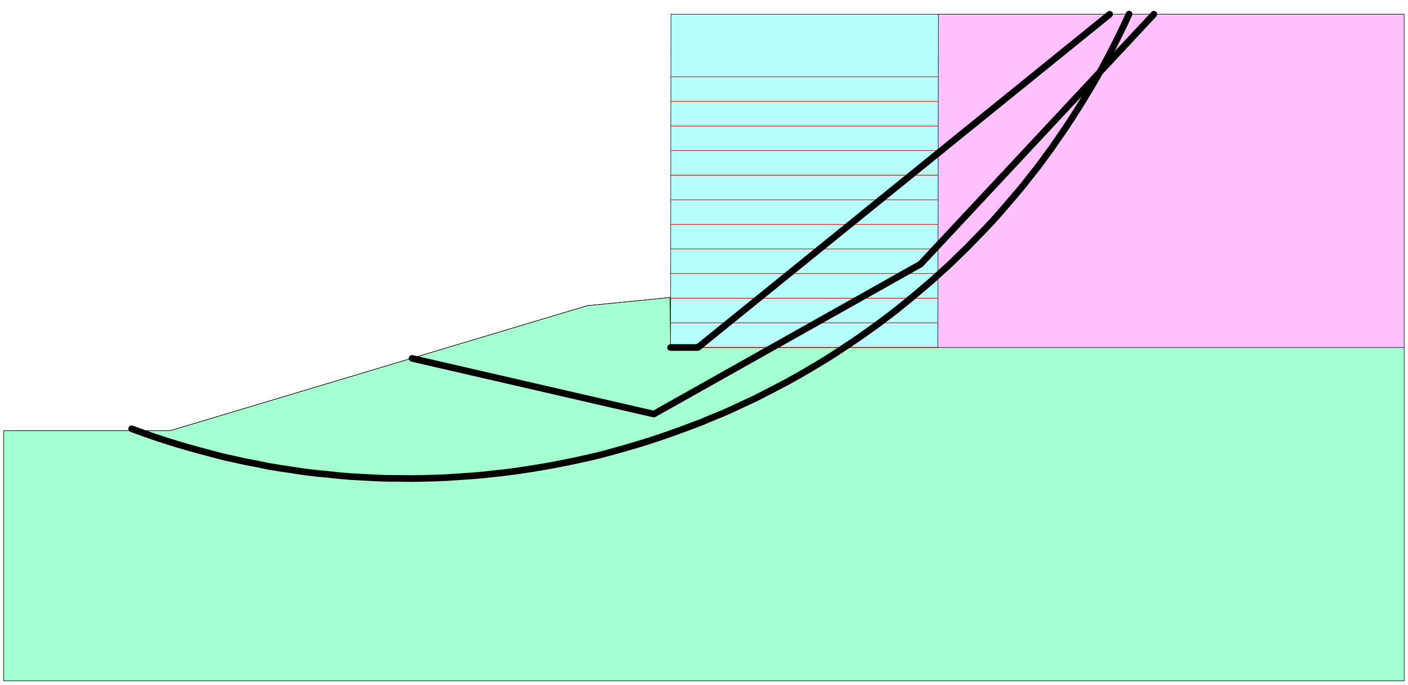
Critical Two-Part Wedge: (Xa = 0.00, Ya = 0.00) [ft]
(Xb = 1.64, Yb = 0.00) [ft]
(Xc = 26.34, Yc = 20.00) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 37.91 [degrees]

Three-Part Wedge Stability Analysis

Minimum Factor of Safety = 1.50

Critical Three-Part Wedge: (X2 = -15.50, Y2 = -0.65) [ft]
(X-left = -1.00, Y-left = -4.00) [ft]
(X-right = 15.00, Y-right = 5.00) [ft]
(X1 = 28.99, Y1 = 20.00) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 25.85 [degrees]

REINFORCEMENT LAYOUT: DRAWING



SCALE:

0 5 10 15 20 25 30 [ft]

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.37

Critical Circle: Xc = -16.23[ft], Yc = 40.79[ft], R = 48.42[ft]. (Number of slices used = 60)

STA 307+50 LT
Seismic

Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

Checked by: R.Fernós 12/4/17

Minimum Factor of Safety = 1.46

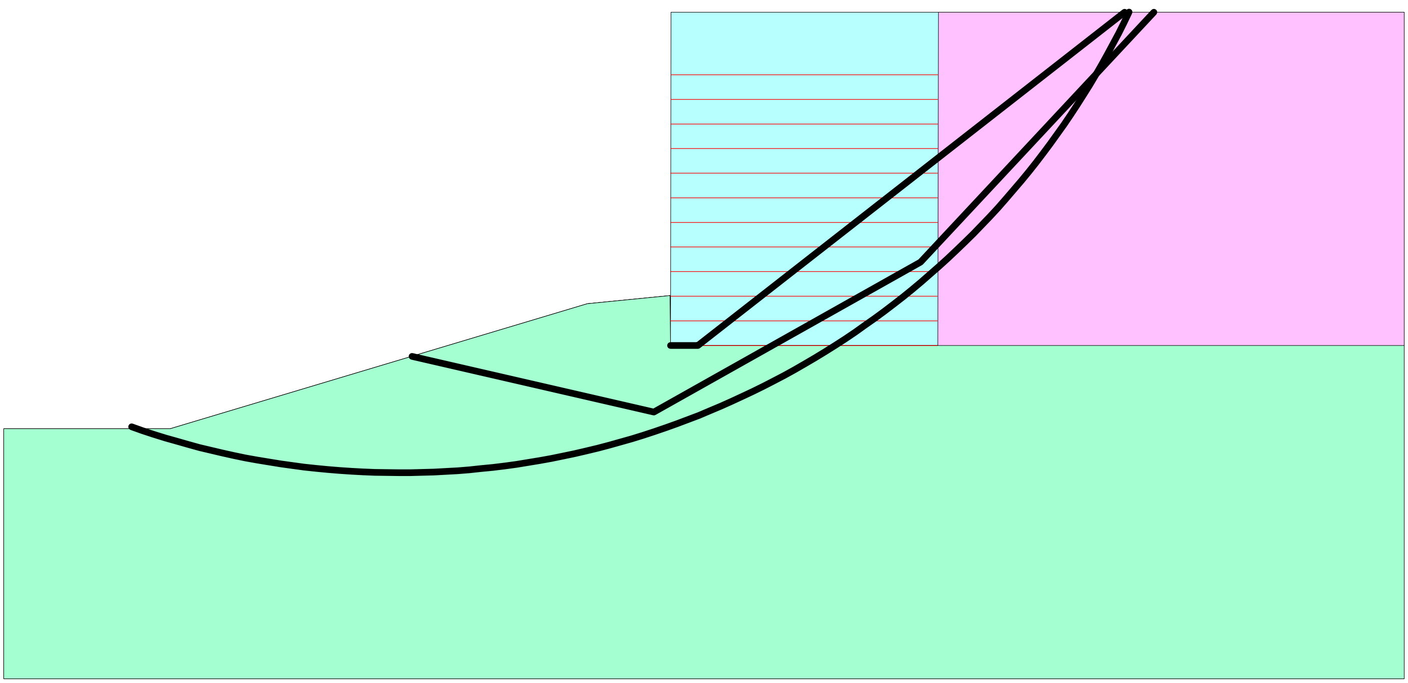
Critical Two-Part Wedge: (Xa = 0.00, Ya = 0.00) [ft]
(Xb = 1.64, Yb = 0.00) [ft]
(Xc = 27.24, Yc = 20.00) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 37.01 [degrees]

Three-Part Wedge Stability Analysis

Minimum Factor of Safety = 1.37

Critical Three-Part Wedge: (X2 = -15.50, Y2 = -0.65) [ft]
(X-left = -1.00, Y-left = -4.00) [ft]
(X-right = 15.00, Y-right = 5.00) [ft]
(X1 = 28.99, Y1 = 20.00) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 25.60 [degrees]

REINFORCEMENT LAYOUT: DRAWING



SCALE:

0 5 10 15 20 25 30 [ft]

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

STA 308+50 LT
Static

Minimum Factor of Safety = 1.56
Critical Circle: Xc = -7.28[ft], Yc = 32.43[ft], R = 37.44[ft]. (Number of slices used = 61)

Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

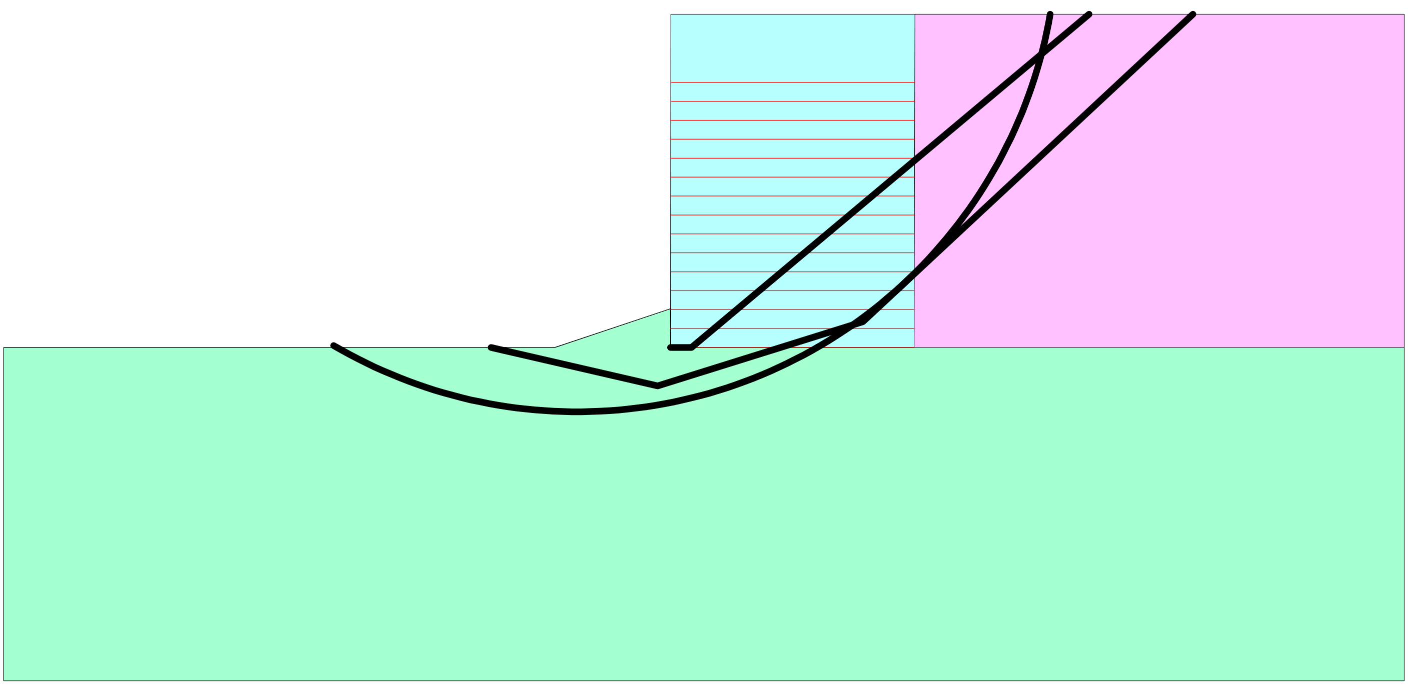
Checked by: R.Fernós 12/4/17

Minimum Factor of Safety = 1.53
Critical Two-Part Wedge: (Xa = 0.00, Ya = 0.00) [ft]
(Xb = 1.64, Yb = 0.00) [ft]
(Xc = 32.63, Yc = 26.00) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 39.25 [degrees]

Three-Part Wedge Stability Analysis

Minimum Factor of Safety = 1.52
Critical Three-Part Wedge: (X2 = -13.99, Y2 = 0.00) [ft]
(X-left = -1.00, Y-left = -3.00) [ft]
(X-right = 15.00, Y-right = 2.00) [ft]
(X1 = 40.74, Y1 = 26.00) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 29.50 [degrees]

REINFORCEMENT LAYOUT: DRAWING



SCALE:

0 5 10 15 20 25 30 [ft]

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.44

Critical Circle: Xc = -7.28[ft], Yc = 32.43[ft], R = 37.44[ft]. (Number of slices used = 61)

STA 308+50 LT
Seismic

Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

Checked by: R.Fernós 12/4/17

Minimum Factor of Safety = 1.42

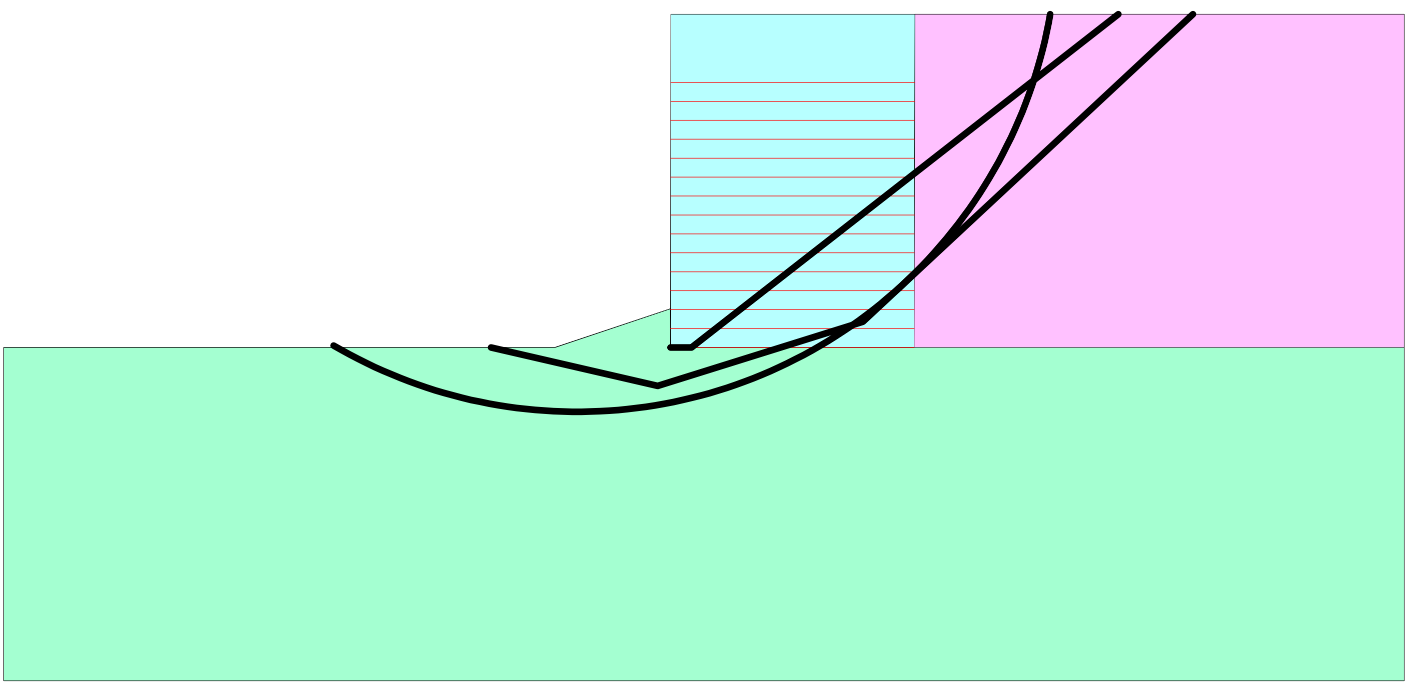
Critical Two-Part Wedge: (Xa = 0.00, Ya = 0.00) [ft]
(Xb = 1.64, Yb = 0.00) [ft]
(Xc = 34.92, Yc = 26.00) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 37.38 [degrees]

Three-Part Wedge Stability Analysis

Minimum Factor of Safety = 1.37

Critical Three-Part Wedge: (X2 = -13.99, Y2 = 0.00) [ft]
(X-left = -1.00, Y-left = -3.00) [ft]
(X-right = 15.00, Y-right = 2.00) [ft]
(X1 = 40.74, Y1 = 26.00) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 29.36 [degrees]

REINFORCEMENT LAYOUT: DRAWING



SCALE:

0 5 10 15 20 25 30 [ft]

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.63

Critical Circle: Xc = -6.12[ft], Yc = 37.90[ft], R = 43.76[ft]. (Number of slices used = 59)

STA 308+60 LT
Static

Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

Checked by: R.Fernós 12/4/17

Minimum Factor of Safety = 1.65

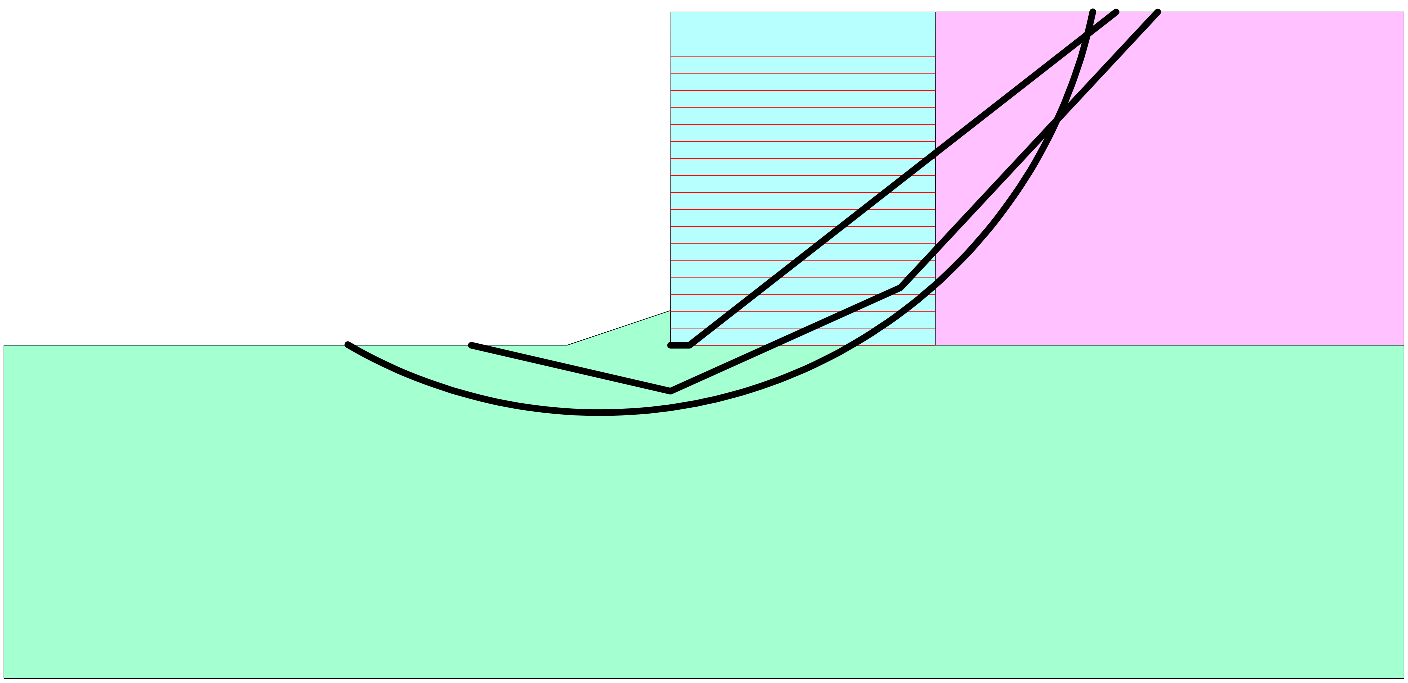
Critical Two-Part Wedge: (Xa = 0.00, Ya = 0.00) [ft]
(Xb = 1.64, Yb = 0.00) [ft]
(Xc = 38.76, Yc = 29.00) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 37.52 [degrees]

Three-Part Wedge Stability Analysis

Minimum Factor of Safety = 1.47

Critical Three-Part Wedge: (X2 = -17.33, Y2 = 0.00) [ft]
(X-left = 0.00, Y-left = -4.00) [ft]
(X-right = 20.00, Y-right = 5.00) [ft]
(X1 = 42.38, Y1 = 29.00) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 29.95 [degrees]

REINFORCEMENT LAYOUT: DRAWING



SCALE:

0 5 10 15 20 25 30 [ft]

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.50

Critical Circle: Xc = -8.61[ft], Yc = 40.51[ft], R = 46.78[ft]. (Number of slices used = 58)

STA 308+60 LT
Seismic

Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

Checked by: R.Fernós 12/4/17

Minimum Factor of Safety = 1.53

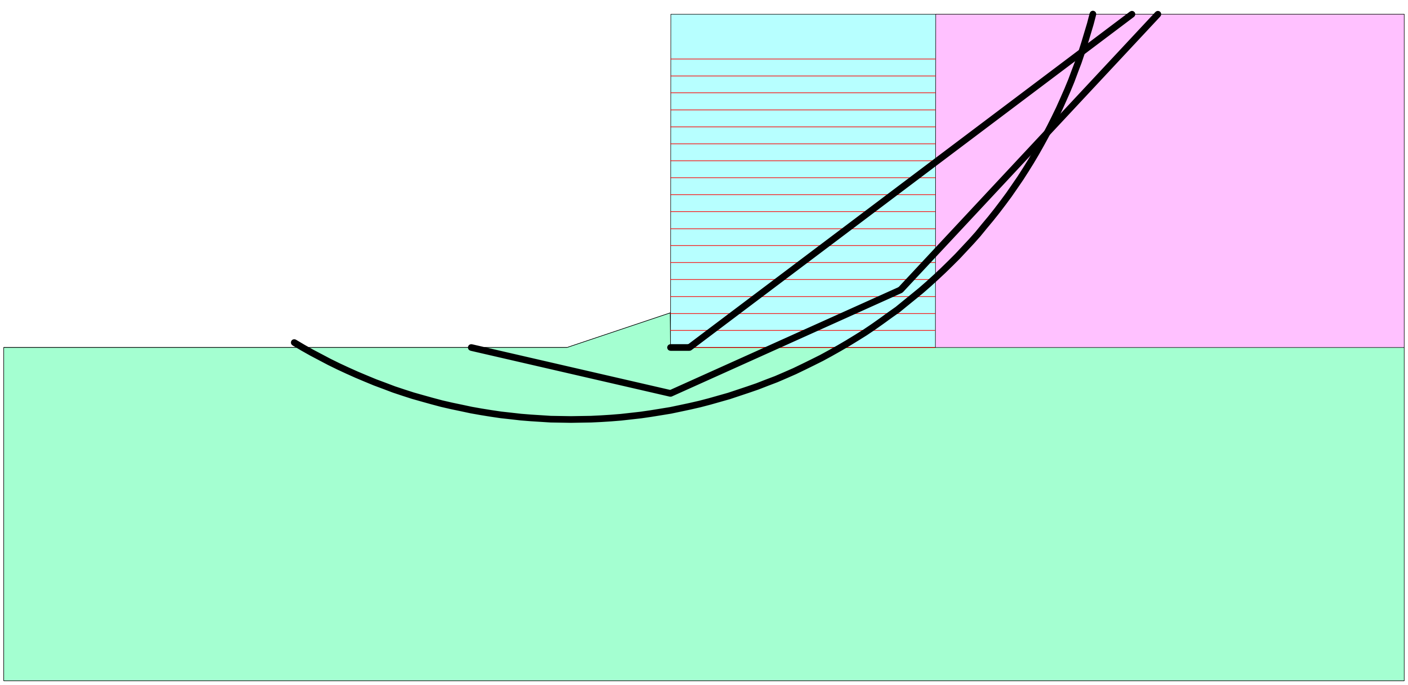
Critical Two-Part Wedge: (Xa = 0.00, Ya = 0.00) [ft]
(Xb = 1.64, Yb = 0.00) [ft]
(Xc = 40.12, Yc = 29.00) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 34.61 [degrees]

Three-Part Wedge Stability Analysis

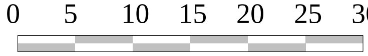
Minimum Factor of Safety = 1.34

Critical Three-Part Wedge: (X2 = -17.33, Y2 = 0.00) [ft]
(X-left = 0.00, Y-left = -4.00) [ft]
(X-right = 20.00, Y-right = 5.00) [ft]
(X1 = 42.38, Y1 = 29.00) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 29.77 [degrees]

REINFORCEMENT LAYOUT: DRAWING



SCALE:



ReSSA analysis at Ramp D STA 308+60 (Abutment 40D)
with undrained strength

Wilmington Viaduct Ramp D

Report created by ReSSA(3.0): Copyright (c) 2001-2010, ADAMA Engineering, Inc.

PROJECT IDENTIFICATION

Title:	Wilmington Viaduct Ramp D
Project Number:	31987 - 004
Client:	DelDOT
Designer:	C. Troxel
Station Number:	S. Abutment

Description:
Check of MSE wall at Ramp D STA 308+60 (Abutment 40D)

Company's information:

Name:
Street:
,
Telephone #:
Fax #:
E-Mail:

Original file path and name: N:\31987-0 ..... on\Calculations\ReSSA\308+60 clay ReSSA1 final.MSE
Original date and time of creating this file: Wed Oct 04 11:37:34 2017

PROGRAM MODE: Analysis of a General Slope using METALLIC as reinforcing material.

INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

=========== Soil Layer #: ===========	Unit weight, γ [lb/ft ³]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft ²]
....1....... Reinforced Soil................................	105.0	34.0	0.0
....2.......Retained/Reinforced Soil......................	105.0	34.0	0.0
....3.......Foundation Soil 1................................	120.0	32.0	0.0
....4.......Foundation Soil 2 - Clay......................	115.0	0.0	1000.0
....5.......Residual...	125.0	32.0	0.0

REINFORCEMENT

Reinforcement Type #	Metal Mat Designated Name	Yield Strength of Steel, Fy [kips/in.²]	Design Cross-Section Area per Mat, Ac [inch²]	Gross Width of Mat, b [inch]	Yield Strength Reduction Factor, RFy	Additional Reduction Factor, RFa	Coverage Ratio, Rc Rc = b / Sh
1	---	65.00	0.30	2.00	1.49	1.00	0.08

Interaction Parameters Type #	Metal Mat Designated Name	== Direct Sliding == Cds-phi	Cds-c	====== Pullout ====== F* top	F* @19.7ft.	Alpha	Thickness of Transverse Bars, t [in.]	Distance Between Transverse Bars, St [in.]
1	---	0.93	0.00	1.80	0.68	1.00	0.39	11.81

Relative Orientation of Reinforcement Force, ROR = 0.50. Assigned Factor of Safety to resist pullout, Fs-po = 1.50
Design method for Global Stability: Comprehensive Bishop.

WATER

Unit weight of water = 62.45 [lb/ft ³]
Water ponding is defined by 'phreatic surface' in Total Stress Analysis.

SEISMICITY

Horizontal peak ground acceleration coefficient, Ao = 0.105
Design horizontal seismic coefficient, kh = Am = 0.50 x Ao = 0.052 & design vertical seismic coefficient, kv (down) = 0.000 x kh = 0.000

DRAWING OF SPECIFIED GEOMETRY - COMPLEX - Quick Input

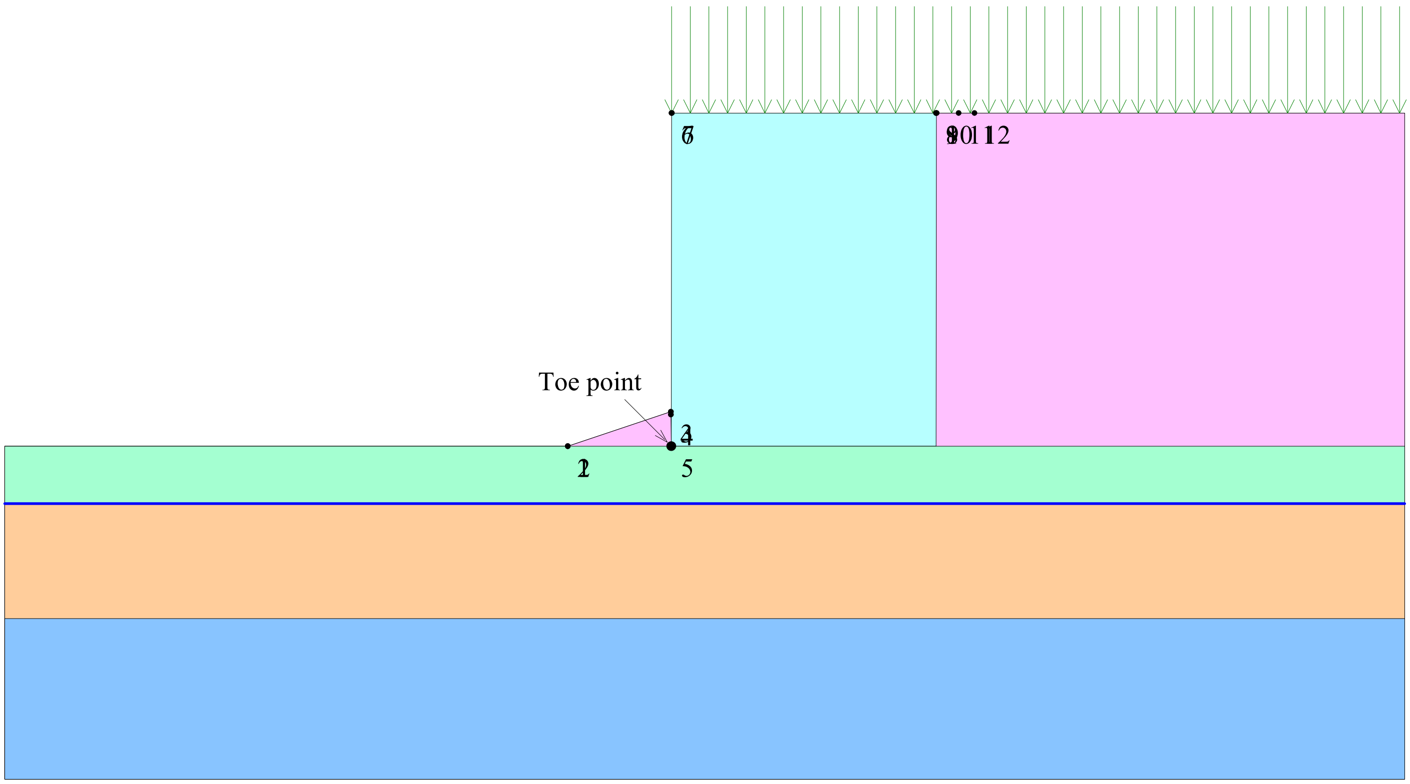
-- Problem geometry is defined along sections selected by user at x,y coordinates.
-- X1,Y1 represents the coordinates of soil surface. X2,Y2 represent the coordinates of the end of soil layer 1 and start of soil layer 2, and so on.
-- Xw,Yw represents the coordinates of phreatic surface.

GEOMETRY
Soil profile contains 5 layers (see details in next page)

WATER GEOMETRY
Phreatic line was specified.

UNIFORM SURCHARGE
Load Q1 = 250.00 [lb/ft²] inclined from verical at 0.00 degrees, starts at X1s = 0.03 and ends at X1e = 1000.03 [ft].
Surcharge load, Q2.................................... None
Surcharge load, Q3.................................... None

STRIP LOAD
............................None..................................



SCALE:

0 5 10 15 20 25 30 [ft]

TABULATED DETAILS OF QUICK SPECIFIED GEOMETRY

Soil profile contains 5 layers. Coordinates in [ft.]
Water was described by phreatic line.

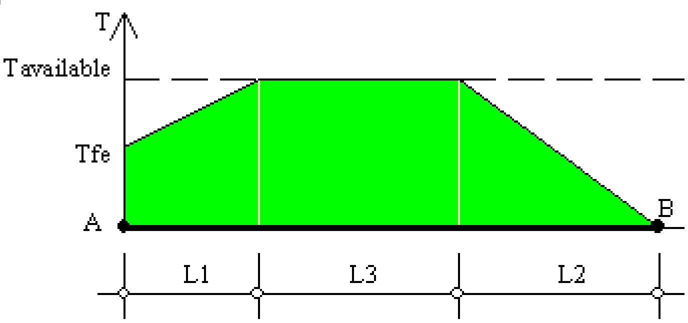
	#	Xi	Yi
Top of Layer 1	1	-9.00	0.00
	2	-0.03	3.00
	3	0.00	0.00
	4	0.03	29.00
	5	0.07	29.00
Top of Layer 2	6	-9.00	0.00
	7	-0.03	3.00
	8	0.00	0.00
	9	23.03	0.00
	10	23.06	29.00
	11	23.10	29.00
	12	26.38	29.00
Top of Layer 3	13	-9.00	0.00
	14	-0.03	0.00
	15	0.00	0.00
Top of Layer 4	16	-9.00	-5.00
	17	-0.03	-5.00
	18	0.00	-5.00
Top of Layer 5	19	-9.00	-15.00
	20	-0.03	-15.00
	21	0.00	-15.00
Top of Phreatic Line	23	0.00	-5.00
	24	25.00	-5.00

TABULATED DETAILS OF SPECIFIED GEOMETRY

Soil profile contains 5 layers. Coordinates in [ft.]
Water was described by phreatic line. Y values are tabulated in the right most column.

#	X	Y1	Y2	Y3	Y4	Y5	(phreatic) Yw
1	-9.00	0.00	0.00	0.00	-5.00	-15.00	-5.00
2	-9.00	0.00	0.00	0.00	-5.00	-15.00	-5.00
3	-0.03	3.00	3.00	0.00	-5.00	-15.00	-5.00
4	-0.03	2.74	2.74	0.00	-5.00	-15.00	-5.00
5	0.00	0.00	0.00	0.00	-5.00	-15.00	-5.00
6	0.03	29.00	0.00	0.00	-5.00	-15.00	-5.00
7	0.07	29.00	0.00	0.00	-5.00	-15.00	-5.00
8	23.03	29.00	0.00	0.00	-5.00	-15.00	-5.00
9	23.06	29.00	29.00	0.00	-5.00	-15.00	-5.00
10	23.10	29.00	29.00	0.00	-5.00	-15.00	-5.00
11	25.00	29.00	29.00	0.00	-5.00	-15.00	-5.00
12	26.38	29.00	29.00	0.00	-5.00	-15.00	-5.00

DISTRIBUTION OF AVAILABLE STRENGTH ALONG EACH REINFORCEMENT LAYER



A = Front-end of reinforcement (at face of slope)
B = Rear-end of reinforcement
AB = L1 + L2 + L3 = Embedded length of reinforcement

Tavailable = Long-term strength of reinforcement
Tfe = Available front-end strength (e.g., connection to facing)

L1 = Front-end 'pullout' length
L2 = Rear-end pullout length
Tavailable prevails along L3

Factor of safety on resistance to pullout on either end of reinforcement, Fs-po = 1.50

Reinforcement Layer #	Designated Name	Height Relative to Toe [ft]	L [ft]	L1 [ft]	L2 [ft]	L3 [ft]	Tfe [lb/ft]	Tavailable [lb/ft]
1	---	0.00	23.00	0.00	23.00	0.00	5074.69	5074.69 (*)
2	---	1.48	23.00	0.00	23.00	0.00	4815.52	4815.52 (*)
3	---	2.95	23.00	0.00	23.00	0.00	4556.65	4556.65 (*)
4	---	4.43	23.00	0.00	23.00	0.00	4299.11	4299.11 (*)
5	---	5.91	23.00	0.00	23.00	0.00	4041.63	4041.63 (*)
6	---	7.38	23.00	0.00	23.00	0.00	3784.22	3784.22 (*)
7	---	8.86	23.00	0.00	23.00	0.00	3526.88	3526.88 (*)
8	---	10.33	23.00	0.00	23.00	0.00	3548.31	3548.31 (*)
9	---	11.81	23.00	0.00	23.00	0.00	3640.43	3640.43 (*)
10	---	13.29	23.00	0.00	23.00	0.00	3668.81	3668.81 (*)
11	---	14.76	23.00	0.00	23.00	0.00	3633.47	3633.47 (*)
12	---	16.24	23.00	0.00	23.00	0.00	3534.39	3534.39 (*)
13	---	17.72	23.00	0.00	23.00	0.00	3371.58	3371.58 (*)
14	---	19.19	23.00	0.00	23.00	0.00	3145.04	3145.04 (*)
15	---	20.67	23.00	0.00	23.00	0.00	2854.77	2854.77 (*)
16	---	22.15	23.00	0.00	23.00	0.00	2500.76	2500.76 (*)
17	---	23.62	23.00	0.00	23.00	0.00	2083.02	2083.02 (*)
18	---	25.10	23.00	0.00	23.00	0.00	1601.55	1601.55 (*)

(*) This Tavailable is dictated by the pullout resistance capacity, which is smaller than the long-term strength of the reinforcement that is related to its specified yield strength

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each entry point (considering all specified exit points)

Entry Point #	Entry Point (X, Y) [ft]		Exit Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	18.00	29.00	-40.53	0.40	-18.53	29.56	36.53	8.56	
2	21.12	29.00	-32.27	0.16	-13.94	30.07	35.08	3.17	
3	24.25	29.00	-28.57	0.35	-10.24	29.56	34.49	1.96	
4	27.37	29.00	-24.06	0.04	-6.54	29.06	33.91	1.66	
5	30.49	29.00	-32.22	0.18	-7.55	29.14	38.04	1.56	
6	33.61	29.00	-32.51	0.47	-5.70	29.21	39.31	1.44	
7	36.73	29.00	-28.67	0.59	-2.36	29.50	39.09	1.39	
8	39.85	29.00	-32.61	0.65	-1.94	29.03	41.79	1.38	
9	42.97	29.00	-32.45	0.49	-0.33	29.53	43.30	1.37	
10	46.09	29.00	-32.37	0.40	1.00	30.78	45.13	1.37 .	OK
11	49.21	29.00	-32.30	0.32	2.35	32.03	46.96	1.37	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-entry' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

****

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

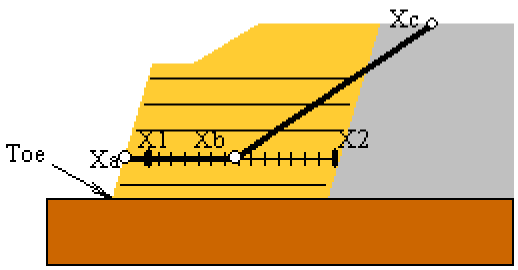
Critical circles for each exit point (considering all specified entry points)

Exit Point #	Exit Point (X, Y) [ft]		Entry Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	-40.33	0.33	49.21	29.00	-3.10	38.21	53.12	1.38	
2	-36.35	0.36	49.21	29.00	-0.53	35.47	50.16	1.37	
. 3	-32.37	0.40	46.09	29.00	1.00	30.78	45.13	1.37 .	OK
4	-28.02	0.02	42.97	29.00	1.40	29.40	41.58	1.37	
5	-24.02	0.02	39.85	29.00	1.29	29.10	38.56	1.38	
6	-20.01	0.01	39.85	29.00	2.81	29.19	37.04	1.41	
7	-16.11	0.07	39.85	29.00	4.36	29.07	35.49	1.45	
8	-12.20	0.13	42.97	29.00	7.71	29.23	35.26	1.56	
9	-8.58	0.52	39.85	29.00	2.98	36.27	37.58	1.88	
10	-4.17	1.75	42.97	29.00	10.67	30.48	32.33	2.53	
11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	N/A	#10 - Overhanging Cliff

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-exit' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

temporary condition - okay

RESULTS OF TRANSLATIONAL ANALYSIS



Results in the table below represent critical two-part wedges identified between specified starting (X1) and ending (X2) search points. Wedges along all reinforcement layers and at elevation zero are reported. The critical two-part wedge, one for each predetermined elevation, is defined by Xa, Xb and Xc where Xa is the front end of the passive wedge (slope face), Xb is where the passive wedge ends and the active one starts, and Xc is the X-ordinate at which the active wedge starts.

Critical two-part wedge along each interface:

Interface	Height Relative to Toe [ft]	(Xa, Ya) [ft]		(Xb, Yb) [ft]		(Xc, Yc) [ft]		Fs	STATUS
At toe elevation	0.00	0.00	0.00	198.49	0.00	246.75	29.00	12.94	Minimum on Edge
Reinf. Layer #1	0.00	0.00	0.00	1.64	0.00	40.12	29.00	1.52	Minimum on Edge
Reinf. Layer #2	1.48	0.00	1.48	3.61	1.48	41.49	29.00	1.61	Minimum on Edge
Reinf. Layer #3	2.95	0.00	2.95	5.58	2.95	42.78	29.00	1.70	Minimum on Edge
Reinf. Layer #4	4.43	0.01	4.43	7.55	4.43	42.64	29.00	1.78	Minimum on Edge
Reinf. Layer #5	5.91	0.01	5.91	9.51	5.91	42.50	29.00	1.88	Minimum on Edge
Reinf. Layer #6	7.38	0.01	7.38	11.48	7.38	42.36	29.00	1.98	Minimum on Edge
Reinf. Layer #7	8.86	0.01	8.86	13.45	8.86	42.22	29.00	2.09	Minimum on Edge
Reinf. Layer #8	10.33	0.01	10.33	15.42	10.33	41.11	29.00	2.23	Minimum on Edge
Reinf. Layer #9	11.81	0.01	11.81	17.39	11.81	41.05	29.00	2.38	Minimum on Edge
Reinf. Layer #10	13.29	0.02	13.29	19.36	13.29	40.98	29.00	2.55	Minimum on Edge
Reinf. Layer #11	14.76	0.02	14.76	21.33	14.76	40.92	29.00	2.76	Minimum on Edge
Reinf. Layer #12	16.24	0.02	16.24	23.29	16.24	40.86	29.00	3.02	Minimum on Edge
Reinf. Layer #13	17.72	0.02	17.72	25.26	17.72	40.79	29.00	3.34	Minimum on Edge
Reinf. Layer #14	19.19	0.02	19.19	27.23	19.19	41.24	29.00	3.72	Minimum on Edge
Reinf. Layer #15	20.67	0.02	20.67	29.20	20.67	41.10	29.00	4.21	Minimum on Edge
Reinf. Layer #16	22.15	0.00	0.00	0.00	0.00	0.00		(1000033.30	No convergence
Reinf. Layer #17	23.62	0.00	0.00	0.00	0.00	0.00		(1000033.30	No convergence
Reinf. Layer #18	25.10	0.00	0.00	0.00	0.00	0.00		(1000033.30	No convergence

Note: In the 'Status' column, OK means the critical two part-wedge was identified within the specified search domain. 'Minimum on Edge' means the critical result corresponds to a minimum on the edge of the search domain; i.e., either on X1 or X2 or the internally preset limits on Xc.

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.37

Critical Circle: Xc = 1.00[ft], Yc = 30.78[ft], R = 45.13[ft]. (Number of slices used = 63)

Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

Minimum Factor of Safety = 1.52

Critical Two-Part Wedge: (Xa = 0.00, Ya = 0.00) [ft]
(Xb = 1.64, Yb = 0.00) [ft]
(Xc = 40.12, Yc = 29.00) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 34.54 [degrees]

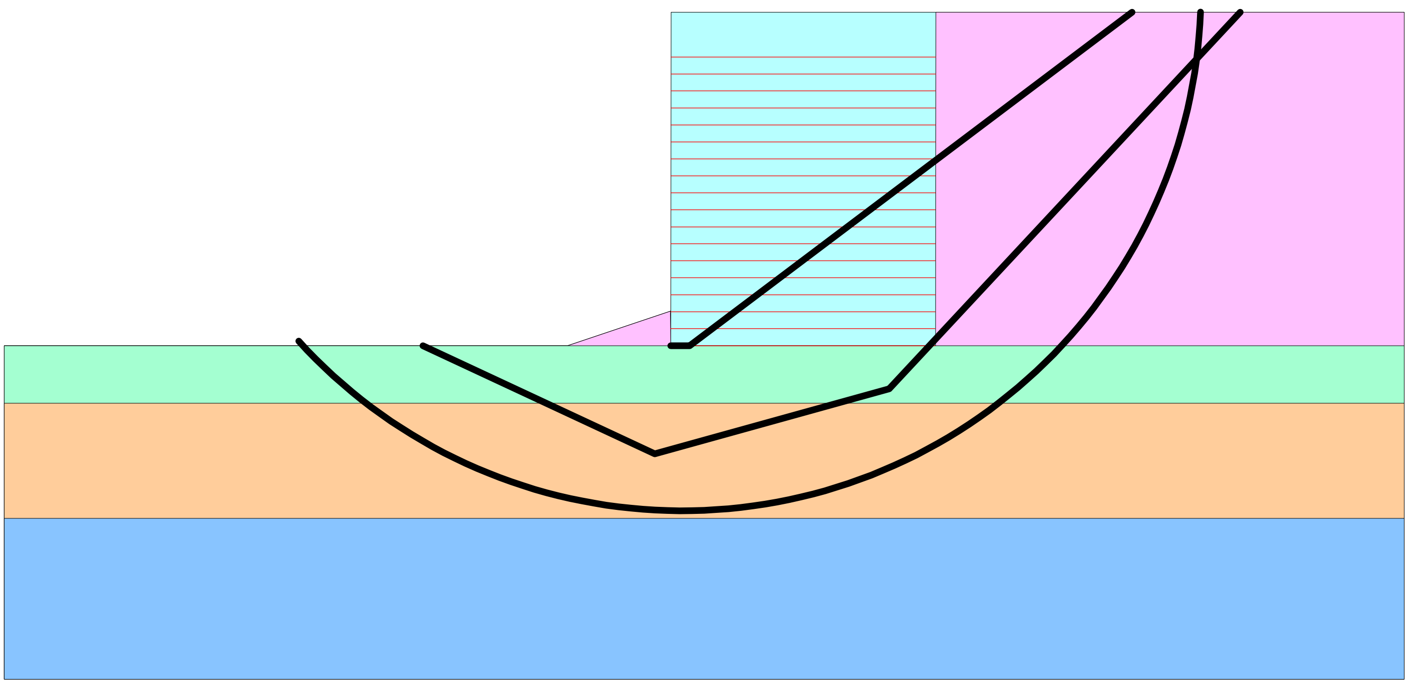
temporary condition - okay

Three-Part Wedge Stability Analysis

Minimum Factor of Safety = 1.22

Critical Three-Part Wedge: (X2 = -21.56, Y2 = 0.00) [ft]
(X-left = -1.40, Y-left = -9.40) [ft]
(X-right = 19.00, Y-right = -3.75) [ft]
(X1 = 49.54, Y1 = 29.00) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 18.92 [degrees]

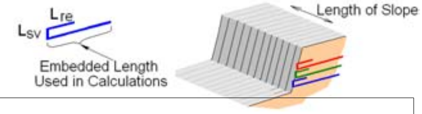
REINFORCEMENT LAYOUT: DRAWING



SCALE:

0 5 10 15 20 25 30 [ft]

REINFORCEMENT LAYOUT: TABULATED DATA & QUANTITIES



Layer #	Reinf. Type #	Metallic Mat Designated Name	Height Relative to Toe [ft]	Embedded Length [ft]	Covergae Ratio, Rc	(X, Y) front [ft]		(X, Y) rear [ft]		Lsv * [ft]	Lre [ft]
1	1	---	0.00	23.00	0.08	0.00	0.00	23.00	0.00	0.00	0.00
2	1	---	1.48	23.00	0.08	0.00	1.48	23.00	1.48	0.00	0.00
3	1	---	2.95	23.00	0.08	0.00	2.95	23.00	2.95	0.00	0.00
4	1	---	4.43	23.00	0.08	0.01	4.43	23.01	4.43	0.00	0.00
5	1	---	5.91	23.00	0.08	0.01	5.91	23.01	5.91	0.00	0.00
6	1	---	7.38	23.00	0.08	0.01	7.38	23.01	7.38	0.00	0.00
7	1	---	8.86	23.00	0.08	0.01	8.86	23.01	8.86	0.00	0.00
8	1	---	10.33	23.00	0.08	0.01	10.33	23.01	10.33	0.00	0.00
9	1	---	11.81	23.00	0.08	0.01	11.81	23.01	11.81	0.00	0.00
10	1	---	13.29	23.00	0.08	0.02	13.29	23.02	13.29	0.00	0.00
11	1	---	14.76	23.00	0.08	0.02	14.76	23.02	14.76	0.00	0.00
12	1	---	16.24	23.00	0.08	0.02	16.24	23.02	16.24	0.00	0.00
13	1	---	17.72	23.00	0.08	0.02	17.72	23.02	17.72	0.00	0.00
14	1	---	19.19	23.00	0.08	0.02	19.19	23.02	19.19	0.00	0.00
15	1	---	20.67	23.00	0.08	0.02	20.67	23.02	20.67	0.00	0.00
16	1	---	22.15	23.00	0.08	0.03	22.15	23.03	22.15	0.00	0.00
17	1	---	23.62	23.00	0.08	0.03	23.62	23.03	23.62	0.00	0.00
18	1	---	25.10	23.00	0.08	0.03	25.10	23.03	25.10	0.00	0.00

* Vertical distance between layers.

QUANTITIES

Reinf. Type #	Designated Name	Coverage Ratio	Area of reinforcemnt [ft²] / length of slope [ft]
1	---	0.08	33.12

ReSSA analysis at Ramp D STA 308+60 (Abutment 40D)
with undrained strength and double sided wall model

Wilmington Viaduct Ramp D

PROJECT IDENTIFICATION

Title:	Wilmington Viaduct Ramp D
Project Number:	31987 - 004
Client:	DelDOT
Designer:	C. Troxel
Station Number:	S. Abutment

Description:
Check of MSE wall at Ramp D STA 308+60 (Abutment 40D) at side of MSE wall - geometry is changed to include the width of the MSE wall.

Company's information:

Name:
Street:
,
Telephone #:
Fax #:
E-Mail:

Original file path and name: N:\31987-0 ..... lculations\ReSSA\308+60 clay ReSSA1 final side.MSE
Original date and time of creating this file: Wed Oct 04 11:37:34 2017

PROGRAM MODE: Analysis of a General Slope using METALLIC as reinforcing material.

INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

=========== Soil Layer #: ===========	Unit weight, γ [lb/ft ³]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft ²]
....1....... Reinforced Soil................................	105.0	34.0	0.0
....2.......Retained/Reinforced Soil......................	105.0	34.0	0.0
....3.......Foundation Soil 1................................	120.0	32.0	0.0
....4.......Foundation Soil 2 - Clay......................	115.0	0.0	1000.0
....5.......Residual..	125.0	32.0	0.0

REINFORCEMENT

R e i n f o r c e m e n t Type #	Metal Mat Designated Name	Yield Strength of Steel, Fy [kips/in.²]	Design Cross-Section Area per Mat, Ac [inch²]	Gross Width of Mat, b [inch]	Yield Strength Reduction Factor, RFy	Additional Reduction Factor, RFa	Coverage Ratio, Rc Rc = b / Sh
1	---	65.00	0.30	2.00	1.49	1.00	0.08

I n t e r a c t i o n P a r a m e t e r s Type #	Metal Mat Designated Name	== Direct Sliding == Cds-phi	Cds-c	====== Pullout ====== F* top	F* @19.7ft.	Alpha	Thickness of Transverse Bars, t [in.]	Distance Between Transverse Bars, St [in.]
1	---	0.93	0.00	1.80	0.68	1.00	0.39	11.81

Relative Orientation of Reinforcement Force, ROR = 0.50. Assigned Factor of Safety to resist pullout, Fs-po = 1.50
Design method for Global Stability: Comprehensive Bishop.

WATER

Unit weight of water = 62.45 [lb/ft ³]
Water ponding is defined by 'phreatic surface' in Total Stress Analysis.

SEISMICITY

Horizontal peak ground acceleration coefficient, Ao = 0.105
Design horizontal seismic coefficient, kh = Am = 0.50 x Ao = 0.052 & design vertical seismic coefficient, kv (down) = 0.000 x kh = 0.000

DRAWING OF SPECIFIED GEOMETRY - COMPLEX - Quick Input

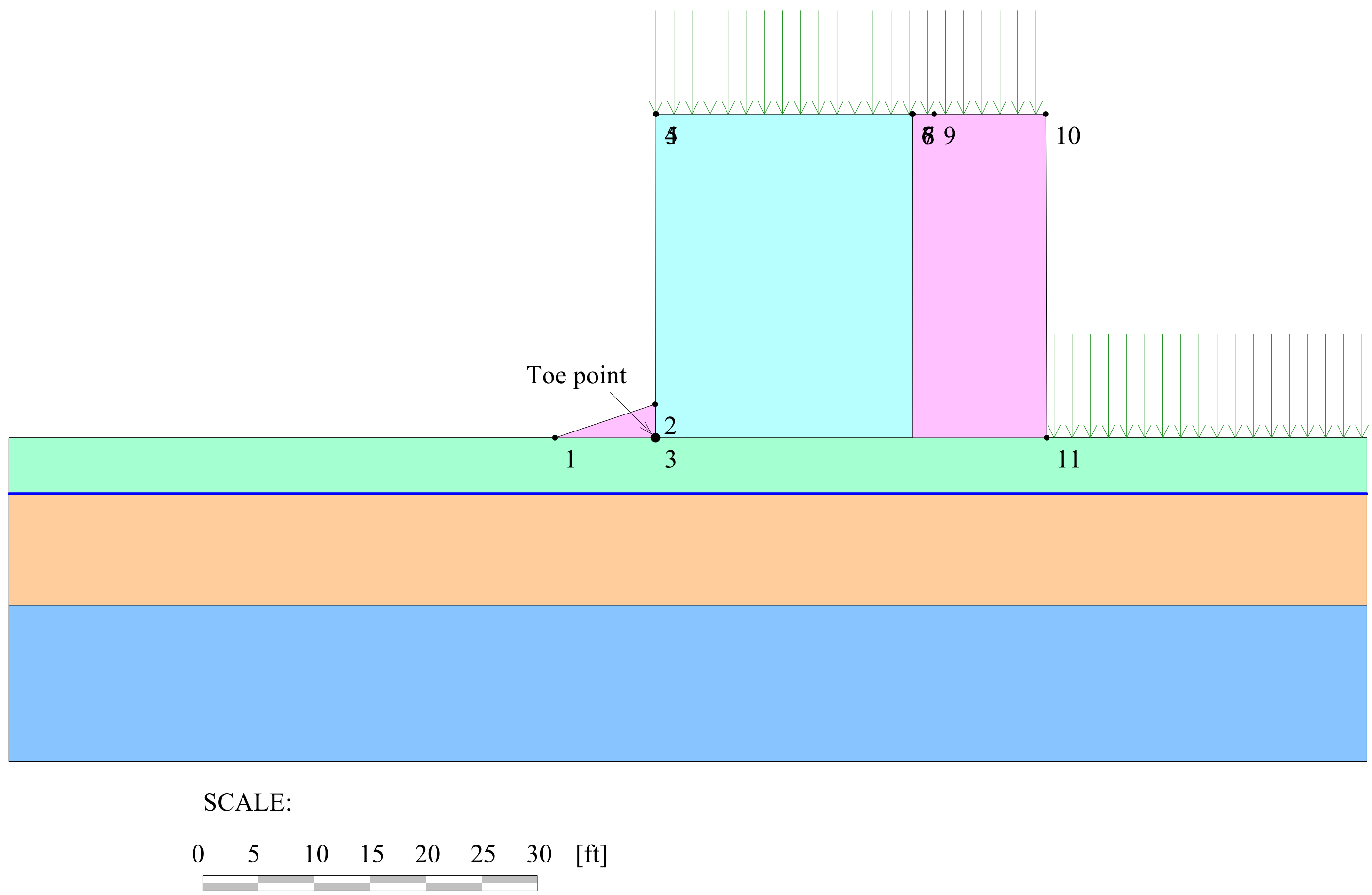
-- Problem geometry is defined along sections selected by user at x,y coordinates.
-- X1,Y1 represents the coordinates of soil surface. X2,Y2 represent the coordinates of the end of soil layer 1 and start of soil layer 2, and so on.
-- Xw,Yw represents the coordinates of phreatic surface.

GEOMETRY
Soil profile contains 5 layers (see details in next page)

WATER GEOMETRY
Phreatic line was specified.

UNIFORM SURCHARGE
Load Q1 = 250.00 [lb/ft²] inclined from verical at 0.00 degrees, starts at X1s = 0.03 and ends at X1e = 1000.03 [ft].
Surcharge load, Q2.................................... None
Surcharge load, Q3.................................... None

STRIP LOAD
............................None...................................



TABULATED DETAILS OF QUICK SPECIFIED GEOMETRY

Soil profile contains 5 layers. Coordinates in [ft.]
Water was described by phreatic line.

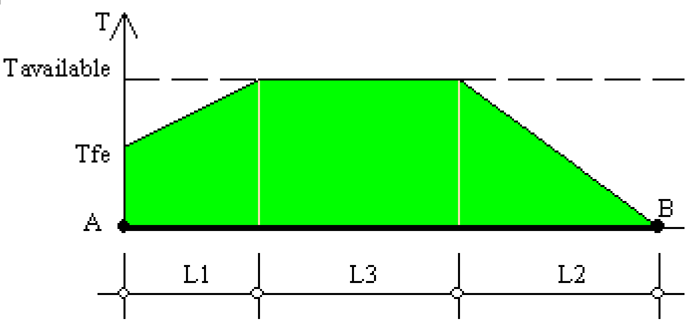
	#	Xi	Yi
Top of Layer 1	1	-9.00	0.00
	2	-0.03	3.00
	3	0.00	0.00
	4	0.03	29.00
	5	0.07	29.00
	6	35.00	29.00
	7	35.10	0.00
Top of Layer 2	8	-9.00	0.00
	9	-0.03	3.00
	10	0.00	0.00
	11	23.03	0.00
	12	23.06	29.00
	13	23.10	29.00
	14	35.00	29.00
	15	35.10	0.00
Top of Layer 3	16	-9.00	0.00
	17	-0.03	0.00
	18	0.00	0.00
Top of Layer 4	19	-9.00	-5.00
	20	-0.03	-5.00
	21	0.00	-5.00
Top of Layer 5	22	-9.00	-15.00
	23	-0.03	-15.00
	24	0.00	-15.00
Top of Phreatic Line	26	0.00	-5.00
	27	25.00	-5.00

TABULATED DETAILS OF SPECIFIED GEOMETRY

Soil profile contains 5 layers. Coordinates in [ft.]
Water was described by phreatic line. Y values are tabulated in the right most column.

#	X	Y1	Y2	Y3	Y4	Y5	(phreatic) Yw
1	-9.00	0.00	0.00	0.00	-5.00	-15.00	-5.00
2	-0.03	3.00	3.00	0.00	-5.00	-15.00	-5.00
3	0.00	0.00	0.00	0.00	-5.00	-15.00	-5.00
4	0.03	29.00	0.00	0.00	-5.00	-15.00	-5.00
5	0.07	29.00	0.00	0.00	-5.00	-15.00	-5.00
6	23.03	29.00	0.00	0.00	-5.00	-15.00	-5.00
7	23.06	29.00	29.00	0.00	-5.00	-15.00	-5.00
8	23.10	29.00	29.00	0.00	-5.00	-15.00	-5.00
9	25.00	29.00	29.00	0.00	-5.00	-15.00	-5.00
10	35.00	29.00	29.00	0.00	-5.00	-15.00	-5.00
11	35.10	0.00	0.00	0.00	-5.00	-15.00	-5.00

DISTRIBUTION OF AVAILABLE STRENGTH ALONG EACH REINFORCEMENT LAYER



A = Front-end of reinforcement (at face of slope)
B = Rear-end of reinforcement
AB = L1 + L2 + L3 = Embedded length of reinforcement

Tavailable = Long-term strength of reinforcement
Tfe = Available front-end strength (e.g., connection to facing)

L1 = Front-end 'pullout' length
L2 = Rear-end pullout length
Tavailable prevails along L3

Factor of safety on resistance to pullout on either end of reinforcement, Fs-po = 1.50

Reinforcement Layer #	Designated Name	Height Relative to Toe [ft]	L [ft]	L1 [ft]	L2 [ft]	L3 [ft]	Tfe [lb/ft]	Tavailable [lb/ft]
1	---	0.00	23.00	0.00	23.00	0.00	5074.93	5074.93 (*)
2	---	1.48	23.00	0.00	23.00	0.00	4815.86	4815.86 (*)
3	---	2.95	23.00	0.00	23.00	0.00	4556.72	4556.72 (*)
4	---	4.43	23.00	0.00	23.00	0.00	4299.17	4299.17 (*)
5	---	5.91	23.00	0.00	23.00	0.00	4041.68	4041.68 (*)
6	---	7.38	23.00	0.00	23.00	0.00	3784.26	3784.26 (*)
7	---	8.86	23.00	0.00	23.00	0.00	3526.91	3526.91 (*)
8	---	10.33	23.00	0.00	23.00	0.00	3548.16	3548.16 (*)
9	---	11.81	23.00	0.00	23.00	0.00	3640.29	3640.29 (*)
10	---	13.29	23.00	0.00	23.00	0.00	3668.70	3668.70 (*)
11	---	14.76	23.00	0.00	23.00	0.00	3633.37	3633.37 (*)
12	---	16.24	23.00	0.00	23.00	0.00	3534.31	3534.31 (*)
13	---	17.72	23.00	0.00	23.00	0.00	3371.52	3371.52 (*)
14	---	19.19	23.00	0.00	23.00	0.00	3144.99	3144.99 (*)
15	---	20.67	23.00	0.00	23.00	0.00	2854.74	2854.74 (*)
16	---	22.15	23.00	0.00	23.00	0.00	2500.75	2500.75 (*)
17	---	23.62	23.00	0.00	23.00	0.00	2083.03	2083.03 (*)
18	---	25.10	23.00	0.00	23.00	0.00	1601.58	1601.58 (*)

(*) This Tavailable is dictated by the pullout resistance capacity, which is smaller than the long-term strength of the reinforcement that is related to its specified yield strength

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each entry point (considering all specified exit points)

Entry Point #	Entry Point (X, Y) [ft]		Exit Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	18.00	29.00	-40.53	0.40	-18.53	29.56	36.53	8.55	
2	21.12	29.00	-32.27	0.16	-13.94	30.07	35.08	3.16	
3	24.25	29.00	-28.57	0.35	-10.24	29.56	34.49	1.96	
4	27.37	29.00	-24.06	0.04	-6.54	29.06	33.91	1.66	
5	30.49	29.00	-32.22	0.18	-7.55	29.14	38.04	1.56	
. 6	33.61	29.00	-32.51	0.47	-5.70	29.21	39.31	1.44 .	OK
7	36.73	0.00	-32.43	0.47	2.36	32.05	46.99	1.71	
8	39.85	0.00	-32.46	0.47	3.92	35.93	50.81	1.88	
9	42.97	0.00	-40.59	0.48	1.49	51.23	65.92	2.10	
10	46.09	0.00	-40.57	0.45	3.05	55.10	69.92	2.40	
11	49.21	0.00	-40.57	0.43	4.61	59.19	74.12	2.74	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-entry' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

****

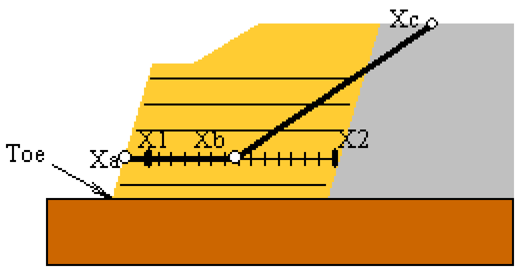
Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each exit point (considering all specified entry points)

Exit Point #	Exit Point (X, Y) [ft]		Entry Point (X, Y) [ft]		Critical Circle (Xc, Yc, R) [ft]			Fs	STATUS
1	-40.57	0.60	33.61	29.00	-9.20	29.74	42.82	1.47	
2	-36.15	0.15	33.61	29.00	-7.47	29.55	41.08	1.45	
. 3	-32.51	0.47	33.61	29.00	-5.70	29.21	39.31	1.44 .	OK
4	-28.41	0.34	33.61	29.00	-4.20	29.39	37.81	1.45	
5	-24.57	0.42	33.61	29.00	-2.67	29.35	36.28	1.47	
6	-20.05	0.03	33.61	29.00	-1.10	29.11	34.71	1.53	
7	-16.31	0.14	33.61	29.00	-1.23	31.67	34.94	1.64	
8	-12.31	0.13	33.61	29.00	0.44	30.80	33.22	1.74	
9	-8.52	0.53	33.61	29.00	2.60	29.47	31.01	1.93	
10	-4.14	1.84	36.73	0.00	17.12	19.40	27.58	15.51	
11	0.00	0.00	0.00	0.00	0.00	0.00	0.00	N/A	#10 - Overhanging Cliff

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-exit' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

RESULTS OF TRANSLATIONAL ANALYSIS



Results in the table below represent critical two-part wedges identified between specified starting (X1) and ending (X2) search points. Wedges along all reinforcement layers and at elevation zero are reported. The critical two-part wedge, one for each predetermined elevation, is defined by Xa, Xb and Xc where Xa is the front end of the passive wedge (slope face), Xb is where the passive wedge ends and the active one starts, and Xc is the X-ordinate at which the active wedge starts.

Critical two-part wedge along each interface:

Interface	Height Relative to Toe [ft]	(Xa, Ya) [ft]		(Xb, Yb) [ft]		(Xc, Yc) [ft]		Fs	STATUS
At toe elevation	0.00	0.00	0.00	0.00	0.00	N/A	N/A	N/A	No convergence
. Reinf. Layer #1	0.00	0.00	0.00	1.64	0.00	35.00	28.00	1.53	Minimum on Edge
Reinf. Layer #2	1.48	0.00	1.48	3.61	1.48	35.01	26.90	1.64	Minimum on Edge
Reinf. Layer #3	2.95	0.00	2.95	5.58	2.95	35.01	26.79	1.73	Minimum on Edge
Reinf. Layer #4	4.43	0.00	4.43	7.55	4.43	35.01	25.89	1.82	Minimum on Edge
Reinf. Layer #5	5.91	0.01	5.91	9.51	5.91	35.01	26.55	1.92	Minimum on Edge
Reinf. Layer #6	7.38	0.01	7.38	11.48	7.38	35.01	27.12	2.02	Minimum on Edge
Reinf. Layer #7	8.86	0.01	8.86	13.45	8.86	35.01	27.59	2.15	Minimum on Edge
Reinf. Layer #8	10.33	0.01	10.33	15.42	10.33	35.00	27.97	2.28	Minimum on Edge
Reinf. Layer #9	11.81	0.01	11.81	17.39	11.81	35.00	28.24	2.44	Minimum on Edge
Reinf. Layer #10	13.29	0.01	13.29	19.36	13.29	35.00	28.40	2.62	Minimum on Edge
Reinf. Layer #11	14.76	0.02	14.76	21.33	14.76	35.00	28.44	2.84	Minimum on Edge
Reinf. Layer #12	16.24	0.02	16.24	23.29	16.24	35.00	28.79	3.11	Minimum on Edge
Reinf. Layer #13	17.72	0.02	17.72	25.26	17.72	35.00	28.92	3.47	Minimum on Edge
Reinf. Layer #14	19.19	0.02	19.19	27.23	19.19	35.00	28.79	3.91	Minimum on Edge
Reinf. Layer #15	20.67	0.00	0.00	0.00	0.00	0.00	(1000033.30		No convergence
Reinf. Layer #16	22.15	0.00	0.00	0.00	0.00	0.00	(1000033.30		No convergence
Reinf. Layer #17	23.62	0.00	0.00	0.00	0.00	0.00	(1000033.30		No convergence
Reinf. Layer #18	25.10	0.00	0.00	0.00	0.00	0.00	(1000033.30		No convergence

Note: In the 'Status' column, OK means the critical two part-wedge was identified within the specified search domain. 'Minimum on Edge' means the critical result corresponds to a minimum on the edge of the search domain; i.e., either on X1 or X2 or the internally preset limits on Xc.

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 1.44
Critical Circle: Xc = -5.70[ft], Yc = 29.21[ft], R = 39.31[ft]. (Number of slices used = 59)

Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

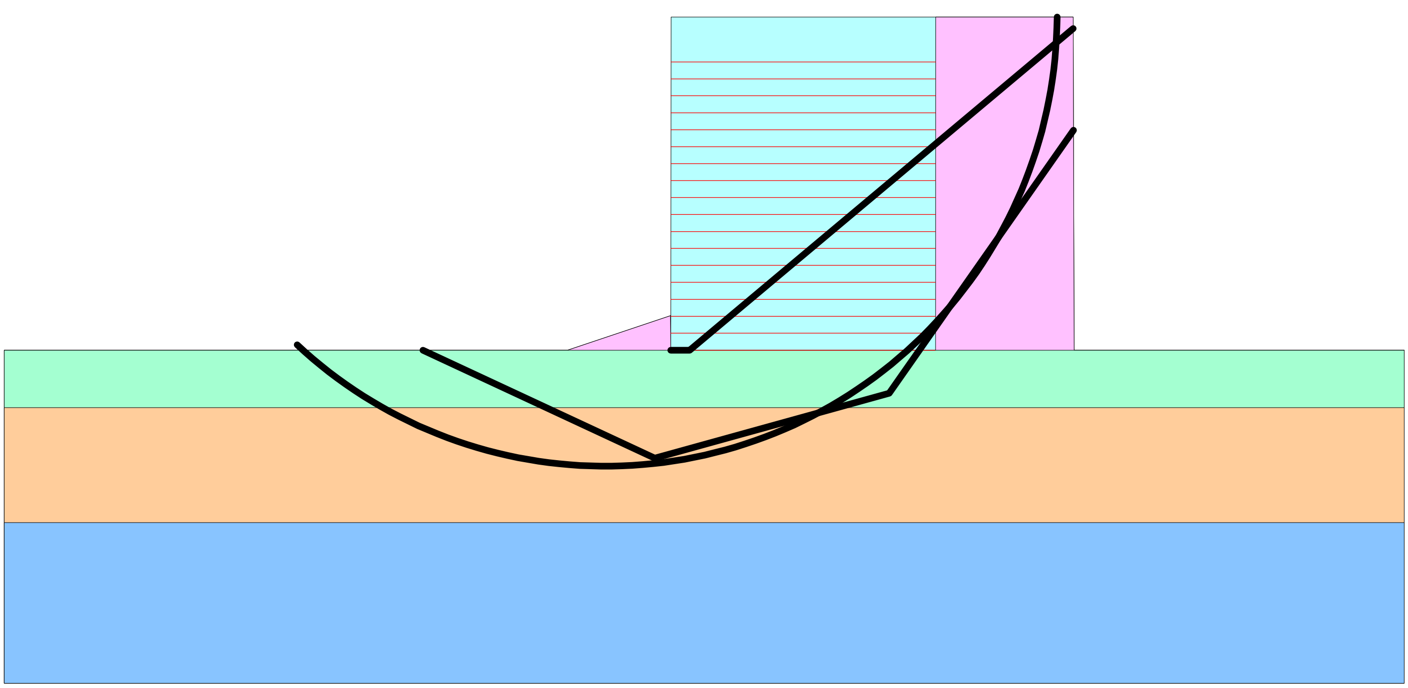
Minimum Factor of Safety = 1.53
Critical Two-Part Wedge: (Xa = 0.00, Ya = 0.00) [ft]
(Xb = 1.64, Yb = 0.00) [ft]
(Xc = 35.00, Yc = 28.00) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 39.47 [degrees]

temporary condition - okay

Three-Part Wedge Stability Analysis

Minimum Factor of Safety = 1.26
Critical Three-Part Wedge: (X2 = -21.56, Y2 = 0.00) [ft]
(X-left = -1.40, Y-left = -9.40) [ft]
(X-right = 19.00, Y-right = -3.75) [ft]
(X1 = 35.03, Y1 = 19.15) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 17.74 [degrees]

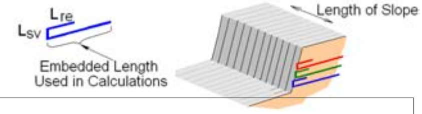
REINFORCEMENT LAYOUT: DRAWING



SCALE:

0 5 10 15 20 25 30 [ft]

REINFORCEMENT LAYOUT: TABULATED DATA & QUANTITIES



Layer #	Reinf. Type #	Metallic Mat Designated Name	Height Relative to Toe [ft]	Embedded Length [ft]	Covergae Ratio, Rc	(X, Y) front [ft]		(X, Y) rear [ft]		Lsv * [ft]	Lre [ft]
1	1	---	0.00	23.00	0.08	0.00	0.00	23.00	0.00	0.00	0.00
2	1	---	1.48	23.00	0.08	0.00	1.48	23.00	1.48	0.00	0.00
3	1	---	2.95	23.00	0.08	0.00	2.95	23.00	2.95	0.00	0.00
4	1	---	4.43	23.00	0.08	0.00	4.43	23.00	4.43	0.00	0.00
5	1	---	5.91	23.00	0.08	0.01	5.91	23.01	5.91	0.00	0.00
6	1	---	7.38	23.00	0.08	0.01	7.38	23.01	7.38	0.00	0.00
7	1	---	8.86	23.00	0.08	0.01	8.86	23.01	8.86	0.00	0.00
8	1	---	10.33	23.00	0.08	0.01	10.33	23.01	10.33	0.00	0.00
9	1	---	11.81	23.00	0.08	0.01	11.81	23.01	11.81	0.00	0.00
10	1	---	13.29	23.00	0.08	0.01	13.29	23.01	13.29	0.00	0.00
11	1	---	14.76	23.00	0.08	0.02	14.76	23.02	14.76	0.00	0.00
12	1	---	16.24	23.00	0.08	0.02	16.24	23.02	16.24	0.00	0.00
13	1	---	17.72	23.00	0.08	0.02	17.72	23.02	17.72	0.00	0.00
14	1	---	19.19	23.00	0.08	0.02	19.19	23.02	19.19	0.00	0.00
15	1	---	20.67	23.00	0.08	0.02	20.67	23.02	20.67	0.00	0.00
16	1	---	22.15	23.00	0.08	0.02	22.15	23.02	22.15	0.00	0.00
17	1	---	23.62	23.00	0.08	0.02	23.62	23.02	23.62	0.00	0.00
18	1	---	25.10	23.00	0.08	0.03	25.10	23.03	25.10	0.00	0.00

* Vertical distance between layers.

QUANTITIES

Reinf. Type #	Designated Name	Coverage Ratio	Area of reinforcemnt [ft²] / length of slope [ft]
1	---	0.08	33.12

Elastic Settlement Estimate - Schmertmann Modified Method

Schmertmann's Modified Method is an elastic solution for stress increase beneath surcharge loads. Schmertmann developed an influence factor diagram for square footings and strip surcharges based on displacements measured in laboratory models and finite element models of material with nonlinear stress-strain behavior. Thus, Schmertmann's method includes the effect of lateral strain. Schmertmann also includes a creep factor for estimating long term settlements. The Schmertmann Modified Method used herein is as described in FHWA (2006). Elastic modulus values are calculated from N60 correlations in FHWA (2006) Table 5-16.

FHWA (2006). Soils and Foundations: Reference Manual - Volume 1. Report No. FHWA-NHI-06-088. Federal Highway Administration (FHWA).

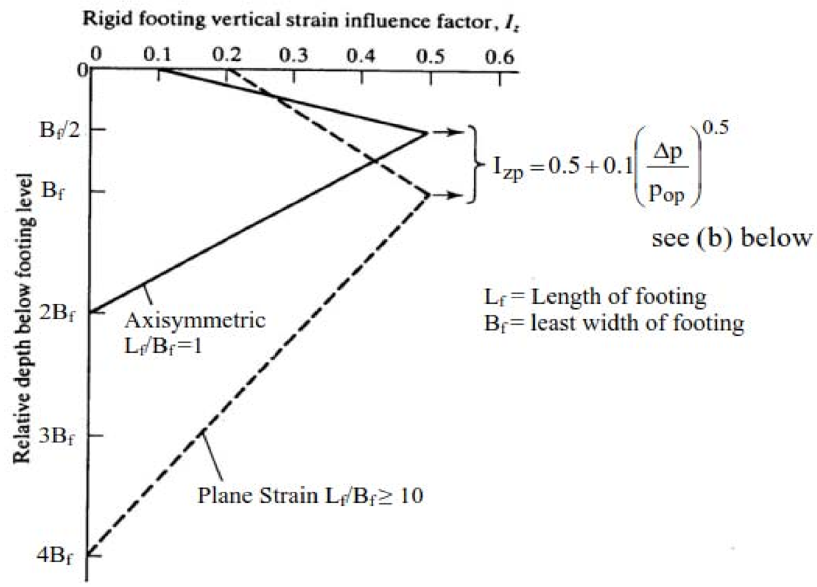
Table 5-16
Elastic constants of various soils (after AASHTO 2004 with 2006 Interims)

Soil Type	**Typical Range of Young's Modulus Values, E_s (tsf)**	**Poisson's Ratio, ν**
Clay:		
Soft sensitive	25-150	
Medium stiff to stiff	150-500	0.4-0.5 (undrained)
Very stiff	500-1,000	
Loess	150-600	0.1-0.3
Silt	20-200	0.3-0.35
Fine Sand:		
Loose	80-120	0.25
Medium dense	120-200	
Dense	200-300	
Sand:		
Loose	100-300	0.20-0.36
Medium dense	300-500	
Dense	500-800	0.30-0.40
Gravel:		
Loose	300-800	0.20-0.35
Medium dense	800-1,000	
Dense	1,000-2,000	0.30-0.40
Estimating E_s from SPT N-value		
Soil Type		E_s (tsf)
Silts, sandy silts, slightly cohesive mixtures		4 $N1_{60}$
Clean fine to medium sands and slightly silty sands		7 $N1_{60}$
Coarse sands and sands with little gravel		10 $N1_{60}$
Sandy gravel and gravels		12 $N1_{60}$
Estimating E_s (tsf) from q_c static cone resistance		
Sandy soils	$2q_c$ where (q_c is in tsf)	
Note: 1 tsf = 95.76 kPa		

An estimate of the immediate settlement, S_i, of spread footings can be made by using Equation 8-16 as proposed by Schmertmann, *et al.* (1978).

$$S_i = C_1 C_2 \, \Delta p \sum_{i=1}^{n} \Delta H_i \quad \text{where} \quad \Delta H_i = H_c \left(\frac{I_z}{XE}\right) \qquad \text{8-16}$$

(a)



(b)

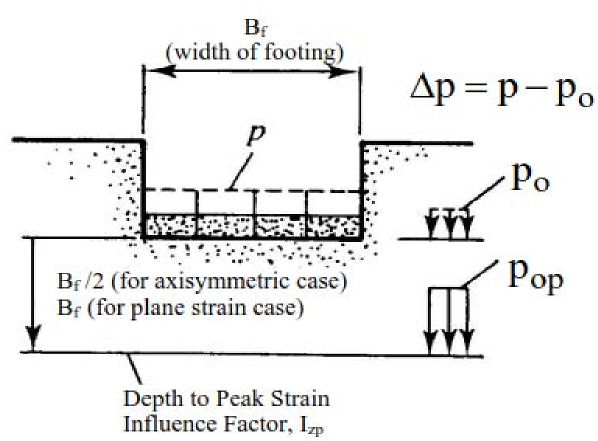


Figure 8-21. (a) Simplified vertical strain influence factor distributions, (b) Explanation of pressure terms in equation for I_{zp} (after Schmertmann, *et al.*, 1978).

Calc By: CFT (10/3/2017)
Check By: LCG (10/4/2017)

Elastic Settlement Estimate - Schmertmann Modified Method
Bridge 1-748S Abutment 40D (And MSE Wall)

Description: Assume 30' height and 120 pcf unit weight of fill. Load from MSE wall only, bridge foundation on piles.
Boring 17B-R6.

Variables

Width, B	35	ft
Length, L	350	ft
Depth of Footing	6	ft
Soil Unit Wt., γ	120	pcf
Stress, q = Q/A	3720	psf
Time of Creep	10	years

Calculation Factors

L/B	10	--
X Factor	1	--
Izp_square	0.62	--
Izp_strip	0.58	--
C1	0.88	--
C2	1.40	--
Δp	3000	psf

Vertical Stress Increase and Displacement from Schmertmann

			Interpolated From FHWA (2006), Fig. 8-21(a), L/B = 1	Interpolated From FHWA (2006), Fig. 8-21(a), L/B = 10	From N60 Correlations	= 12*C1*q*Δz*I / E	= q*Δz*I / E
Depth, z	Ratio z/B	Layer Thickness, ΔH	Influence Factor, Iz_square	Influence Factor, Iz_strip	E	Displ. Square, ρ_square	Displ. Strip, ρ_strip
ft	--	ft	--	--	ksf	in	in
5	0.14	10	0.25	0.30	1000	0.08	0.09
13.5	0.39	7	0.50	0.39	650	0.17	0.13
20	0.57	6	0.59	0.43	400	0.28	0.20
25.5	0.73	5	0.53	0.40	1350	0.06	0.05

Σρ At Center of Load =	**0.59**	**0.48**	**in**
Σρ At Edge of Load =	**0.30**	**0.24**	**in**
Estimated Creep =	**0.12**	**0.10**	**in**
Total Immediate + Creep =	**0.41**	**0.33**	**in**

Calc By: CFT (10/3/2017)
Check By: LCG (10/4/2017)

Elastic Settlement Estimate - Schmertmann Modified Method
Bridge 1-750 Abutment A/North (And MSE Wall)

Description: Assume 20' height and 120 pcf unit weight of fill. Load from MSE wall only, bridge foundation on piles. Boring 17B-R10.

Variables

Width, B	35	ft
Length, L	50	ft
Depth of Footing	3	ft
Soil Unit Wt., γ	120	pcf
Stress, q = Q/A	2500	psf
Time of Creep	10	years

Calculation Factors

L/B	1.43	--
X Factor	1	--
Izp_square	0.60	--
Izp_strip	0.57	--
C1	0.92	--
C2	1.40	--
Δp	2140	psf

Vertical Stress Increase and Displacement from Schmertmann

			Interpolated From FHWA (2006), Fig. 8-21(a), L/B = 1	Interpolated From FHWA (2006), Fig. 8-21(a), L/B = 10	From N60 Correlations	= 12*C1*q*Δz*I / E	= q*Δz*I / E
Depth, z	Ratio z/B	Layer Thickness, ΔH	Influence Factor, Iz_square	Influence Factor, Iz_strip	E	Displ. Square, ρ_square	Displ. Strip, ρ_strip
ft	--	ft	--	--	ksf	in	in
2.5	0.07	5	0.17	0.26	400	0.05	0.08
10	0.29	10	0.39	0.34	700	0.13	0.12

Σρ At Center of Load =	**0.18**	**0.19**	**in**
Σρ At Edge of Load =	**0.09**	**0.10**	**in**
Estimated Creep =	**0.04**	**0.04**	**in**
Total Immediate + Creep =	**0.13**	**0.14**	**in**

Boring 17B-R10 shows about 4 feet of loose/soft material under the bottom of the new MSE wall. This boring was performed outside the footprint of the existing ramp embankment. Recommend inspection of MSE wall excavations and undercutting to remove overly soft or wet material prior to MSE wall footing construction.

Calc By: CFT (10/3/2017)
Check By: LCG (10/4/2017)

Elastic Settlement Estimate - Schmertmann Modified Method
Bridge 1-750 Abutment B/South (And MSE Wall)

Description: Assume 27' height and 120 pcf unit weight of fill. Load from MSE wall only, bridge foundation on piles. Boring 17B-R9.

Variables

Width, B	35	ft
Length, L	350	ft
Depth of Footing	4	ft
Soil Unit Wt., γ	120	pcf
Stress, q = Q/A	3240	psf
Time of Creep	10	years

Calculation Factors

L/B	10	--
X Factor	1	--
Izp_square	0.61	--
Izp_strip	0.58	--
C1	0.91	--
C2	1.40	--
Δp	2760	psf

Vertical Stress Increase and Displacement from Schmertmann

			Interpolated From FHWA (2006), Fig. 8-21(a), L/B = 1	Interpolated From FHWA (2006), Fig. 8-21(a), L/B = 10	From N60 Correlations	= 12*C1*q*Δz*I / E	= q*Δz*I / E
Depth, z	Ratio z/B	Layer Thickness, ΔH	Influence Factor, Iz_square	Influence Factor, Iz_strip	E	Displ. Square, ρ_square	Displ. Strip, ρ_strip
ft	--	ft	--	--	ksf	in	in
2.5	0.07	5	0.17	0.27	400	0.07	0.10
12.5	0.36	15	0.47	0.38	700	0.30	0.25

Σρ At Center of Load =	**0.37**	**0.35**	**in**
Σρ At Edge of Load =	**0.18**	**0.17**	**in**
Estimated Creep =	**0.07**	**0.07**	**in**
Total Immediate + Creep =	**0.26**	**0.24**	**in**

Increase in stress calculated using influence factor by Harr.

Harr, M.E. (1977). *Mechanics of Particulate Materials,* McGraw-Hill.

$$I := \frac{1}{2\pi}\cdot\left[\frac{m\cdot n}{\sqrt{1+m^2+n^2}}\cdot\frac{1+m^2+2\cdot n^2}{\left(1+n^2\right)\cdot\left(m^2+n^2\right)}+\operatorname{asin}\left(\frac{m}{\sqrt{m^2+n^2}\cdot\sqrt{1+n^2}}\right)\right]$$

Variables

Length, c	175	ft
Width, d	17.5	ft
Stress, q = Q/A	3000	psf
Distortion, m=c/d	10.000	--

This is additional settlement estimate at Abutment 40 D (STA 308+60) using the Harr method. The center edge of wall is the front face of wall directly in front of Abutment 40D. The wall corners are the wall corners directly east and west of Abutment 40D.

3000 PSF is the net stress at the bottom of the MSE wall.

Settlement at Center Edge of Wall

	= z/d	See Equation Above		From N60 Correlations	= 2*q*Δz*I / E	
Depth, z	Normalized Depth, n	Influence Factor, I	Layer Thickness, Δz	E	Displacement, ρ	Cumulative Displacement, ρt
ft	--	--	ft	ksf	in	in
5	0.29	0.25	10	1000	0.18	0.60
13.5	0.77	0.22	7	650	0.17	0.43
20	1.14	0.19	6	400	0.21	0.25
25.5	1.46	0.17	5	1350	0.05	0.05

Settlement at Wall Corners

	= z/(2*d)	See Equation Above		From N60 Correlations	= q*Δz*I / E	
Depth, z	Normalized Depth, n	Influence Factor, I	Layer Thickness, Δz	E	Displacement, ρ	Cumulative Displacement, ρt
ft	--	--	ft	ksf	in	in
5	0.14	0.25	10	1000	0.09	0.34
13.5	0.39	0.24	7	650	0.09	0.25
20	0.57	0.24	6	400	0.13	0.16
25.5	0.73	0.23	5	1350	0.03	0.03

Increase in stress calculated using influence factor by Harr.

Harr, M.E. (1977). *Mechanics of Particulate Materials,* McGraw-Hill.

$$I := \frac{1}{2\pi}\cdot\left[\frac{m\cdot n}{\sqrt{1+m^2+n^2}}\cdot\frac{1+m^2+2\cdot n^2}{\left(1+n^2\right)\cdot\left(m^2+n^2\right)}+\operatorname{asin}\left(\frac{m}{\sqrt{m^2+n^2}\cdot\sqrt{1+n^2}}\right)\right]$$

Variables

Stress, q = Q/A	3000	psf
Length, c	350	ft
Width, d	40	ft
Distort., m=c/d	8.750	--

Length, c'	350	ft
Width, d'	5	ft
m'=c'/'d	70.000	--

Settlement 5' Outside of Wall Corner

	= z/d1	See Equation Above	= z/d2	See Equation Above		From N60 Correlations
Depth, z	Normalized Depth, n	Influence Factor, I1	Normalized Depth, n2	Influence Factor, I2	Layer Thickness, Δz	E
ft	--	--	--	--	ft	ksf
5	0.13	0.25	1.00	0.20	10	1000
13.5	0.34	0.25	2.70	0.11	7	650
20	0.50	0.24	4.00	0.08	6	400
25.5	0.64	0.23	5.10	0.06	5	1350

	= q*Δz*I / E	
Depth, z	Displacement, ρ	Cumulative Displacement, ρt
ft	in	in
5	0.02	0.18
13.5	0.05	0.16
20	0.09	0.11
25.5	0.02	0.02

This is additional settlement estimate at Abutment 40 D (STA 308+60) using the influence factor method by Harr. The point 5' outside the wall is 5' directly east of the east wall corner at Abutment 40D.

The point outside the wall was checked in order to estimate ground settlement at the existing Pier 40 piles.

B/MSE wall = EL 33
B/Pier 40 Ftg = EL 32
Est. B/Pile @ Pier 40 = EL 7

The existing pile tips are 26 feet below the B/SE wall, so they are at the bottom of this profile.

* UPDATED TO DELDOT LAB RESULTS & LIQUIDITY INDEX (LI) 6/22/18

WR&A WHITMAN, REQUARDT & ASSOCIATES, LLP
801 South Caroline Street, Baltimore, Maryland 21231

SUBSURFACE DIAGRAM

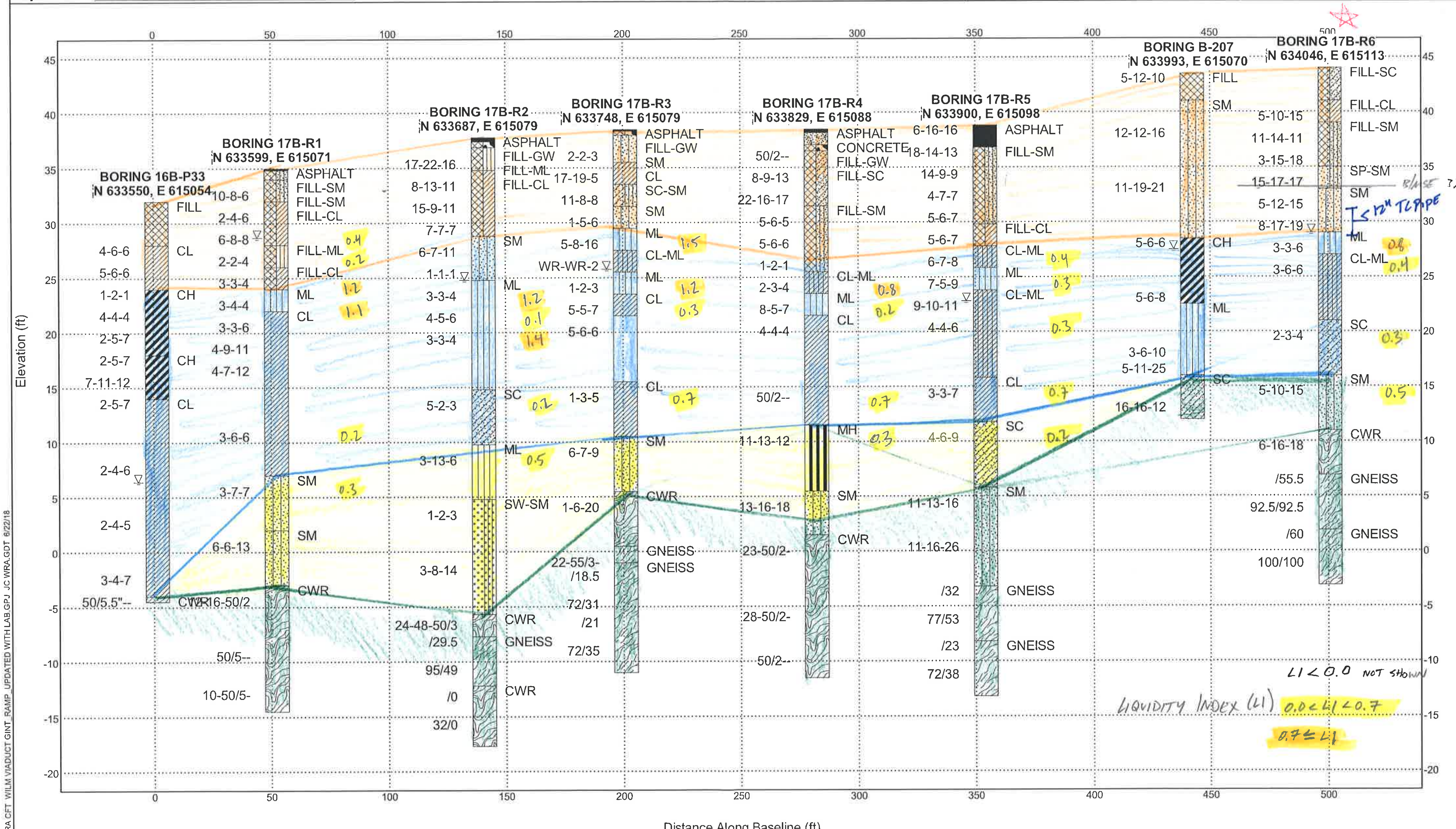
CLIENT Delaware Department of Transportation

Project Number

PROJECT NAME Wilmington Viaduct: Ramp D: 2nd St to Southbound I-95

PROJECT LOCATION Wilmington

Fill (made ground)	USCS Low Plasticity Clay	USCS High Plasticity Clay
Weathered Rock	Asphalt	FILL with USCS Classification of SM
FILL with USCS Classification of CL	FILL with USCS Classificaiton of ML	USCS Low Plasticity Silt
USCS Silty Sand	USCS Well-graded Gravel	USCS Clayey Sand



WRA CFT WILM VIADUCT GINT RAMP UPDATED WITH LAB.GPJ JC WRA.GDT 6/22/18

*UPDATED WITH DelDOT LAB RESULTS & HAS LIQUIDITY INDEX (LI) SHOWN 6/22/18

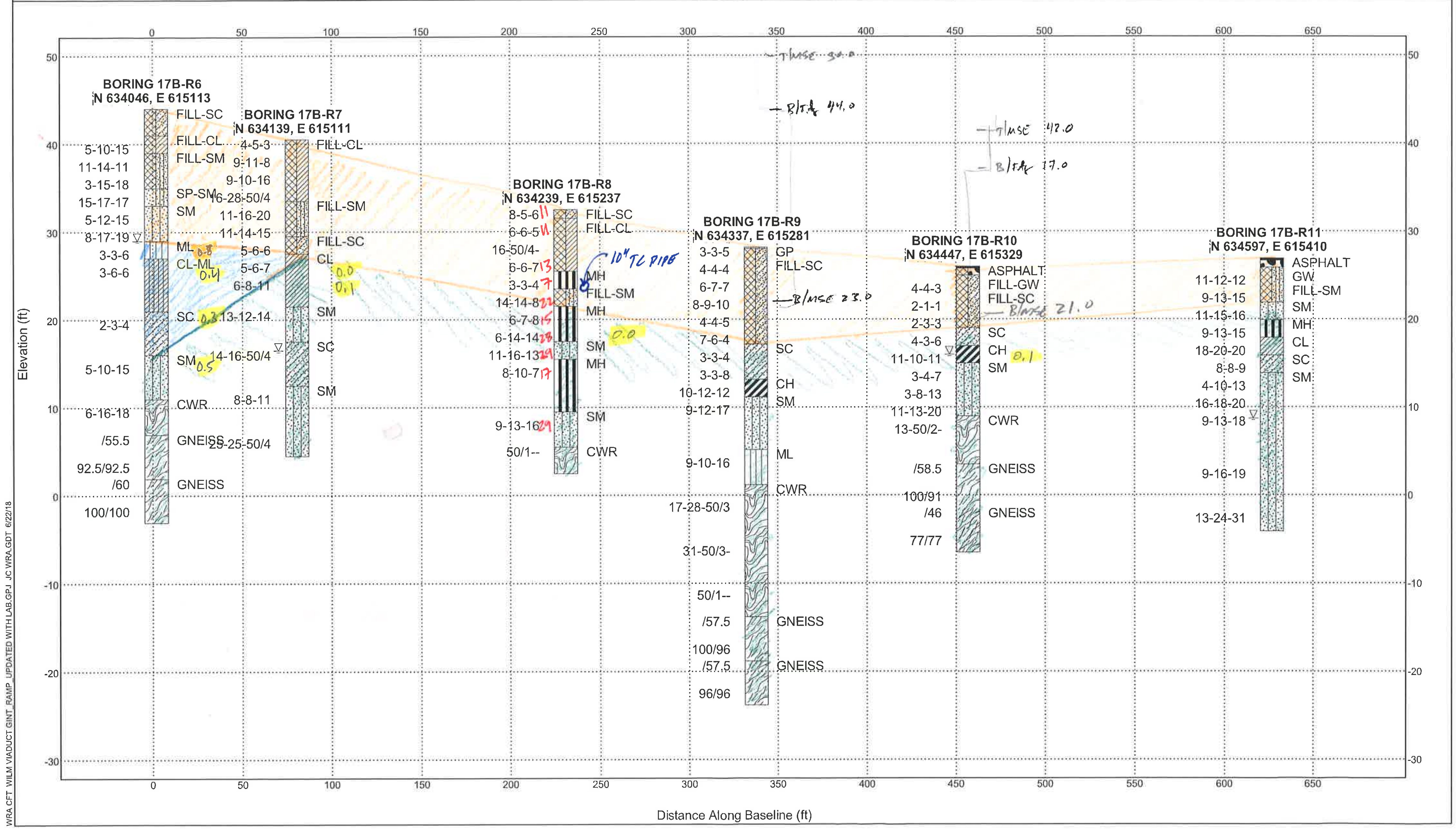
WR&A WHITMAN, REQUARDT & ASSOCIATES, LLP
801 South Caroline Street, Baltimore, Maryland 21231

SUBSURFACE DIAGRAM

CLIENT Delaware Department of Transportation
Project Number

PROJECT NAME Wilmington Viaduct: Ramp D: 2nd St to Southbound I-95
PROJECT LOCATION Wilmington

Asphalt	USCS Well-graded Gravel	FILL with USCS Classification of SC
USCS Clayey Sand	USCS High Plasticity Clay	USCS Silty Sand
Weathered Rock	Gneiss	USCS Well-graded Gravel
FILL with USCS Classification of SM	USCS High Plasticity Silt	USCS Low Plasticity Clay



WRA CFT WILM VIADUCT GINT_RAMP_UPDATED WITH LAB.GPJ JC WRA.GDT 6/22/18

CONSOLIDATION SETTLEMENT

Check of consolidation settlement at Abutment 40D corner. Consolidation index is estimated based on correlations with liquid limit (LL) and initial void ratio (e0) in Bowles (1979). *Physical and Geotechnical Properties of Soils.*

Consolidation Index Approximation

$LL := 38$ — Liquid limit at R6, sample 9, depth 16'

$e_0 := 0.679$ — Initial void ratio from R7 undisturbed sample and 1 tsf normal stress

$C_{c\_LL} := 0.009\ (LL - 10) = 0.252$ — Correlation for undisturbed clays, low to medium sensitivity

$C_{c\_e0} := 0.30\ (e_0 - 0.27) = 0.123$ — Correlation for inorganic cohesive silt and clay

$C_c := 0.19$ — Assumed consolidation index

Soil Parameters, Depths and Stresses

$\gamma_s := 120\ pcf$ $\quad \gamma_w := 62.4\ pcf$ — Unit weight, soil and water

$d := 17\ ft$ $\quad d_w := 15\ ft$ — Depth to center of clay layer, depth to water

$H_0 := 6\ ft$ — Thickness of clay layer

$\sigma_0{}' := d \cdot \gamma_s - (d - d_w) \cdot \gamma_w = 1915.2\ psf$ — Initial stress at center of clay layer

$H_{wall} := 24\ ft$ $\quad B := 35\ ft$ — Height of new wall fill, width of MSE embankment

$\frac{d}{B} = 0.486$ $\quad \eta := 0.4$ — Depth/width ratio and corresponding stress reduction from Westergaard stress distribution

$\sigma_f{}' := \sigma_0{}' + \eta\ H_{wall} \cdot \gamma_s = 3067.2\ psf$ — Final stress at clay layer

Consolidation Settlement

$$\rho_c := \frac{H_0}{1 + e_0}\ C_c \cdot \log\left(\frac{\sigma_f{}'}{\sigma_0{}'}\right) = 1.67\ in$$

Estimate of consolidation settlement along cornerof MSE embankment. Settlment along front face of MSE wall, at the end of the strip load, will be half this value.

$$\rho_{estimated} := \frac{\rho_c}{2} = 0.8\ in$$

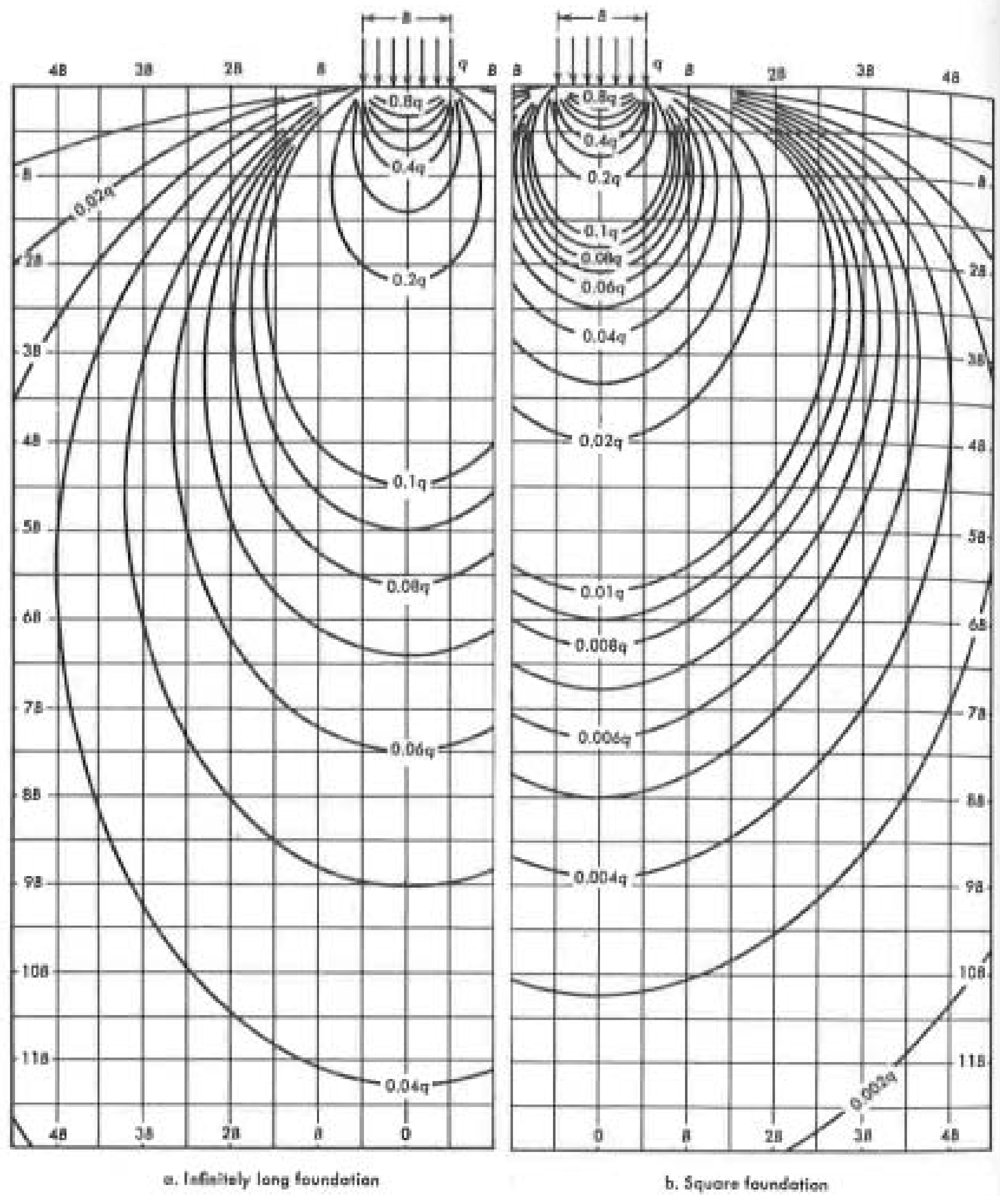


Figure 10.12 *Contours of equal vertical stress beneath a foundation on a semiinfinite, homogeneous, thinly stratified material—the Westergaard analysis. Stresses are given as proportions of the uniform surface pressure, q; distances and depths in terms of the foundation width, B.*

Westergaard pressure distribution from Sowers (1979). The 0.4 load factor is located at the edge of the footing, 0.5*B below the surface. This is for an infinite strip load, the MSE embankment is finite and the area being checked is at the end of the strip, the load will be half of the load estimated herein.

Additional Calculations

Lateral Squeeze

Calculate the FS for lateral squeeze at Abutment 40D. Initial check according to FHWA-NHI-16-009 *Design and Construction of Driven Pile Foundations.* Calculation according to FHWA-NHI-06-088 *Soils and Foundations Reference Manual,* section 7.6.2.

$s_u := 1000\ psf$ Undrained shear strength of clay layer

$D_s := 6\ ft$ Assumed thickness of clay layer

$\gamma := 105\ pcf$ Unit weight of MSE wall fill

$H := 24\ ft$ Additional height of wall fill

$\theta := 85\ deg$ Slope angle

$H \cdot \gamma = 2520\ psf$ Initial check of susceptibility to lateral squeeze.
For H*g < 3su, lateral squeeze not likely to occur.

$3 \cdot s_u = 3000\ psf$

$$FS_{squeeze} := \frac{2\ s_u}{\gamma \cdot D_s \cdot \tan(\theta)} + \frac{4.14\ s_u}{H \cdot \gamma} = 1.92$$

For 1.3 < FS < 2.0 additional checks are recommended but lateral squeeze is not expected for the stiff CL material.

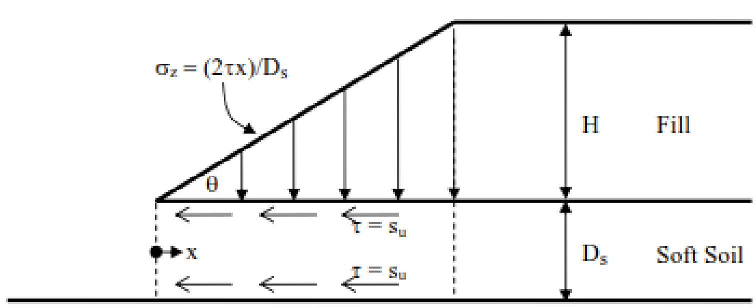


Figure 7-20. Definitions for calculating safety factor against lateral squeeze (after Silvestri, 1983).

Consolidation Waiting Period

Estimate time for consolidation to occur beneath MSE wall.

$t_{90\_lab} := 6\ min$ t_90 estimated from DS testing at 2.0 TSF stress

$H_{dr\_lab} := 1\ in$ Drainage layer thickness (thickness of direct shear test sample)

$H_{dr\_field} := 6\ ft$ Thickness of clay layer

$$t_{90\_field} := \frac{{H_{dr\_field}}^2}{{H_{dr\_lab}}^2} \cdot t_{90\_lab} = 21.6\ day$$ Approximate time for 90% of consolidation

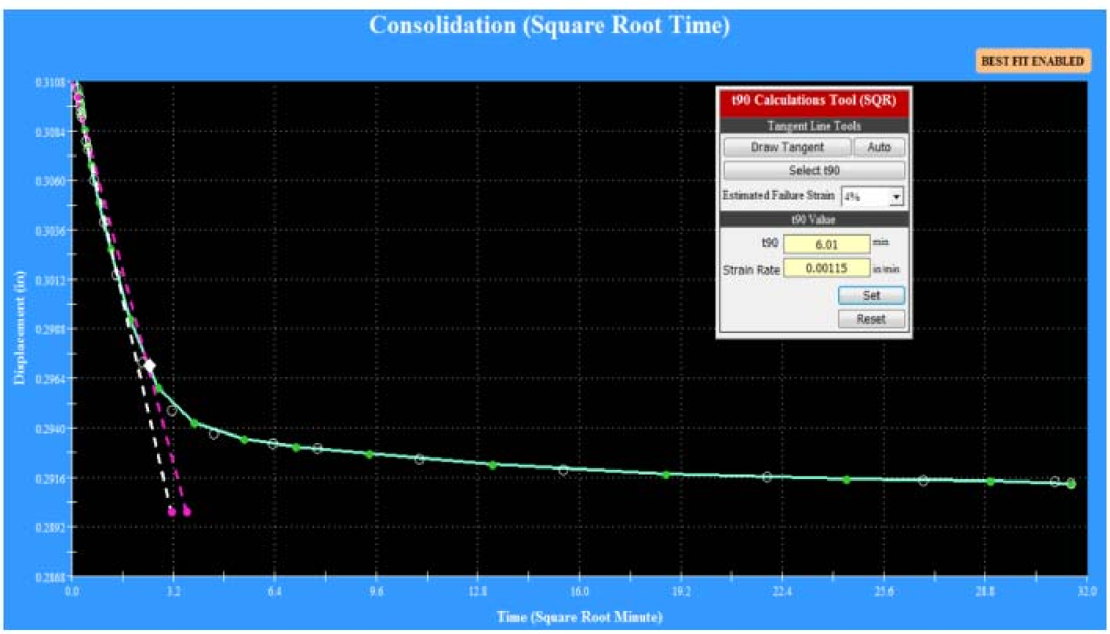


Figure 2 from ETS Report No. ETS-17T219-10:
Determination of Time Required to Achieve 90% Consolidation.

Soil parameters for H-piles

Revised Soil Parameters

Design: CFT (8/14/2017)

Table DP - 3: Design Profile for Pier 39D and Abutment 40D.

Review: R.Fernós 12/4/17

Layer Description	Depth From - To	Elevation From - To	Soil Type	Typical N60 Range	Average N60	Effective Unit Weight[1]	Total Unit Weight[1], γ
	(Feet)	(Feet)	--	(BPF)	(BPF)	(pcf)	(pcf)
Fill	0 to 10	40 to 30	FILL	25	25	53	115
Q1: Fine-grained	10 to 25	30 to 15	CL - CH	10 to 30	20	53	115
Residual/HWR	25 to 35	15 to 5	ML to CL	25 to 40	30	63	125
Rock	35 to 45	5 to -5	GNEISS	--	--	83	145

Layer Description	Elevation From - To	Soil Friction Angle[1]	Soil Cohesion[1], c	Subgrade Modulus[2], k	Strain at 50%[2], ε_{50}	Shear Modulus[2], G	Poisson's Ratio[2], v
	(Feet)	(Degrees)	(psf)	(pci)	--	(ksi)	--
Fill	40 to 30	32	--	75	--	1.40	0.25
Q1: Fine-grained	30 to 15	30	--	60	--	1.00	0.35
Residual/HWR	15 to 5	36	--	150	--	2.45	0.25
Rock	5 to -5	45	--	--	--	90	0.2

			STEEL PILES		
Layer Description	Elevation From - To	Uniaxial Comp. Strength, qu	Ult. Side Friction[2] MP, q_{u_side}	Ult. Bearing[2] H12X53 q_p	Unit End Bearing
	(Feet)	(ksf)	(psf)	(kips)	(ksi)
Fill	40 to 30	--	500		--
Q1: Fine-grained	30 to 15	--	1000		--
Residual/HWR	15 to 5	--	1500	250	1.75
Rock	5 to -5	1000	2000	1000	7.5

References

1) Bowles, Joseph E. (1979). *Physical and Geotechnical Properties of Soils.* McGraw Hill, Inc. New York, NY.

2) FB-MultiPier User Manual. Appendix: *FB-MultiPier Soil Parameter Table.* Software version 4.19.2.

Revised Soil Parameters

Design: CFT (9/26/2017)

Table DP - 4: Design Profile for 750 Abutments 1 and 2. Review: R.Fernós 12/4/17

Layer Description	Depth From - To	Elevation From - To	Soil Type	Typical N60 Range	Average N60	Effective Unit Weight[1]	Total Unit Weight[1], γ
	(Feet)	(Feet)	--	(BPF)	(BPF)	(pcf)	(pcf)
Fill	0 to 15	45 to 20	FILL	7 to 11	8	53	115
Residual/HWR	15 to 37	20 to 7	ML to CL	15 to 25	20	63	125
Rock	37 to 65	7 to -10	GNEISS	--	--	83	145

Layer Description	Elevation From - To	Soil Friction Angle[1]	Soil Cohesion[1], c	Subgrade Modulus[2], k	Strain at 50%[2], ε_{50}	Shear Modulus[2], G	Poisson's Ratio[2], ν
	(Feet)	(Degrees)	(psf)	(pci)	--	(ksi)	--
Fill	45 to 20	30	--	60	--	0.60	0.25
Residual/HWR	20 to 7	34	--	120	--	1.50	0.25
Rock	7 to -10	45	--	--	--	90	0.2

			STEEL PILES		
Layer Description	Elevation From - To	Uniaxial Comp. Strength, qu	Ult. Side Friction[2] MP, q_{u_side}	Ult. Bearing[2] H12X53 q_p	Unit End Bearing
	(Feet)	(ksf)	(psf)	(kips)	(ksi)
Fill	45 to 20	--	1000		--
Residual/HWR	20 to 7	--	1500	150	1
Rock	7 to -10	1000	2000	1000	7.5

References

1) Bowles, Joseph E. (1979). *Physical and Geotechnical Properties of Soils.* McGraw Hill, Inc. New York, NY.

2) FB-MultiPier User Manual. Appendix: *FB-MultiPier Soil Parameter Table.* Software version 4.19.2.

H-PILE SECTION CORROSION

H12x53 before section loss

$d := 11.8\ in$ $\quad b_f := 12.0\ in$ — Pile dimension: depth, flange width and flange and web thickness

$t_f := 0.435\ in$ $\quad t_w := 0.435\ in$

$A := 2 \cdot b_f \cdot t_f + d \cdot t_w = 15.6\ in^2$ — Approximate uncorroded area of pile

H12x53 after section loss

$x := 2.20\ mm$ — Assumed corrosion loss over 100 year design life for non-aggressive fill. From Skyline Piling Handbook, 8th edition (revised 2008)

$x = 0.087\ in$

$b_{fc} := b_f - 2\ x = 11.827\ in$ — Corroded pile dimensions

$t_{fc} := t_f - 2\ x = 0.262\ in$

$t_{wc} := t_w - 2\ x = 0.262\ in$

$A_c := 2 \cdot b_{fc} \cdot t_{fc} + d \cdot t_{wc} = 9.3\ in^2$ — Approximate corroded area of pile

$\phi_c := 0.70$ — Resistance factor, axial resistance of undamaged H-piles
2014 AASHTO LRFD, 6.5.4.2

$F_y := 50\ ksi$ — Steel strength

$P_n := F_y \cdot A_c = 464\ kip$ — Nominal compressive resistance

$P_r := \phi_c \cdot P_n = 325\ kip$ — Factored compressive resistance

PRELIMINARY STRUCTURAL RESISTANCE - STEEL H-PILE

Structural resistance calculated according to 2012 AASHTO LRFD Bride Design Specification sections 6.9.2.1 for axial compression only; 6.9.2.2 for axial compression with flexure; resistance factors according to section 6.5.4.2; nominal compressive resistance from section 6.9.4.1.

Unbraced length of pile (l) is taken as 0 ft, leading to simplifications for flexural resistance calculations.

Pile Dimensions - HP12X53

$A_g := 15.5\ in^2$ — Pile section area/steel area

$F_y := 50\ ksi$ $E := 29000\ ksi$ — Steel yield strength and elastic modulus

$Z_x := 74.0\ in^3$ $Z_y := 32.2\ in^3$ — Plastic section modulus, strong and weak axes

$b_f := 12.0\ in$ $t_f := 0.435\ in$ — Flange and web thickness

Axial Compression Only

$\phi_{c0} := 0.50$ — Resistance factor for H-piles in severe driving and axial compression

$P_n := F_y \cdot A_g = 775\ kip$ — Nominal structural resistance

$P_{r0} := \phi_{c0} \cdot P_n = 388\ kip$ — Factored resistance (compression only)

Factored Compressive & Moment Resistance

$\phi_c := 0.70$ — Axial resistance for H-piles in compression and flexure

$\phi_f := 1.0$ — Flexural resistance for H-piles

$P_n = 775\ kip$ — Nominal structural resistance

$P_r := \phi_c \cdot P_n = 543\ kip$ — Factored compressive resistance (compression and flexure)

Nominal Moment Resistance - Weak Axis Bending, Section 6.12.2.2

$\lambda_f := \dfrac{b_f}{2\ t_f} = 14$ — Slenderness ratio for flange

$F_{yf} := F_y = 50\ \boldsymbol{ksi}$ — Minimum yield strength of lower strength flange

$\lambda_{pf} := 0.38 \cdot \sqrt{\dfrac{E}{F_{yf}}} = 9$ — Limiting slenderness ratio for compact flange

$\lambda_{rf} := 0.83 \cdot \sqrt{\dfrac{E}{F_{yf}}} = 20$ — Limiting slenderness ratio for a noncompact flange

$S_y := 21.1\ \boldsymbol{in}^3 \qquad Z_y := 32.2\ \boldsymbol{in}^3$ — Section moduli across weak axis, elastic and plastic

$$M_{py} := \left(1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45 \cdot \sqrt{\dfrac{E}{F_{yf}}}}\right)\right) F_{yf} \cdot Z_y = 1372\ \boldsymbol{in} \cdot \boldsymbol{kip}$$

Nominal moment resistance (eq. 6.12.2.2.1-2)

$M_{px} := F_y \cdot Z_x$ — Plastic moment about strong axis

$M_{rx} := \phi_f \cdot M_{px} = 3700\ \boldsymbol{in} \cdot \boldsymbol{kip}$ — Factored flexural resistance about strong axis

$M_{ry} := \phi_f \cdot M_{py} = 1372\ \boldsymbol{in} \cdot \boldsymbol{kip}$ — Factored flexural resistance about weak axis

Ultimate Loadings

Moments calculated using P-y analysis (GROUP) for anticipated axial and lateral loads. Applied moment direction depends on orientation of piles relative to bridge (weak or strong axis bending); weak-axis bending is assumed in this analysis.

$P_u := 250\ \boldsymbol{kip}$ — Axial compressive load

$M_{ux} := 0\ \boldsymbol{in} \cdot \boldsymbol{kip}$ — Factored flexural moment about strong axis

$M_{uy} := 460\ \boldsymbol{in} \cdot \boldsymbol{kip}$ — Factored flexural moment about weak axis

Check of Structural Resistance (Value to remain less than 1.0)

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) = 0.76$$

6.9.2.2 - Values less than 1.0 are okay for axial load and bending

AXIALLY LOADING PILE ANALYSIS PROGRAM - APILE
VERSION 2013 - (C) COPYRIGHT ENSOFT,INC.,1987-2013.

Wilm. Viaduct, Ramp D, Pier 35D to 39D

DESIGNER : C. Troxel

DATE : 9/26/2017

Plugged Pile Tip

PILE PROPERTIES :

PERIMETER OF PILE WITH NONCIRCULAR SECTION= 47.60 IN.
TIP AREA OF PILE WITH NONCIRCULAR SECTION = 0.97 SQF
OUTSIDE DIAMETER OF CIRCULAR PILE = 0.00 IN.
INTERNAL DIAMETER OF CIRCULAR PILE = 0.00 IN.
PILE LENGTH = 45.00 FT.
MODULUS OF ELASTICITY = 0.290E+08 PSI

LENGTH OF SURFACE SECTION WITH ZERO SKIN FRICTION = 0.00 FT.
INCREMENT OF PILE LENGTH USED IN COMPUTATION = 1.00 FT.

SOIL INFORMATIONS :

	DEPTH FT.	SOIL TYPE	LATERAL EARTH PRESSURE	EFFECTIVE UNIT WEIGHT LB/CF	FRICTION ANGLE DEGREES	BEARING CAPACITY FACTOR
	0.00	SAND	0.00	120.00	32.00	0.00
	10.00	SAND	0.00	120.00	32.00	0.00
	10.00	CLAY	0.00	43.00	0.00	0.00
	30.00	CLAY	0.00	43.00	0.00	0.00
	30.00	SAND	0.00	53.00	32.00	0.00
	40.00	SAND	0.00	53.00	32.00	0.00
	40.00	SAND	0.00	63.00	40.00	0.00
HWR	45.00	SAND	0.00	63.00	40.00	0.00
	45.00	CLAY	0.00	78.00	0.00	0.00
ROCK	60.00	CLAY	0.00	78.00	0.00	0.00

MAXIMUM UNIT FRICTION KSF	MAXIMUM UNIT BEARING KSF	UNDISTURB SHEAR STRENGTH KSF	REMOLDED SHEAR STRENGTH KSF	BLOW COUNT	UNIT SKIN FRICTION KSF	UNIT END BEARING KSF
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	1.25	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	1.25	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	100.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	100.00	0.00	0.00	0.00	0.00

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*********************
* COMPUTATION RESULT *
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                  *  FED. HWY. METHOD  *
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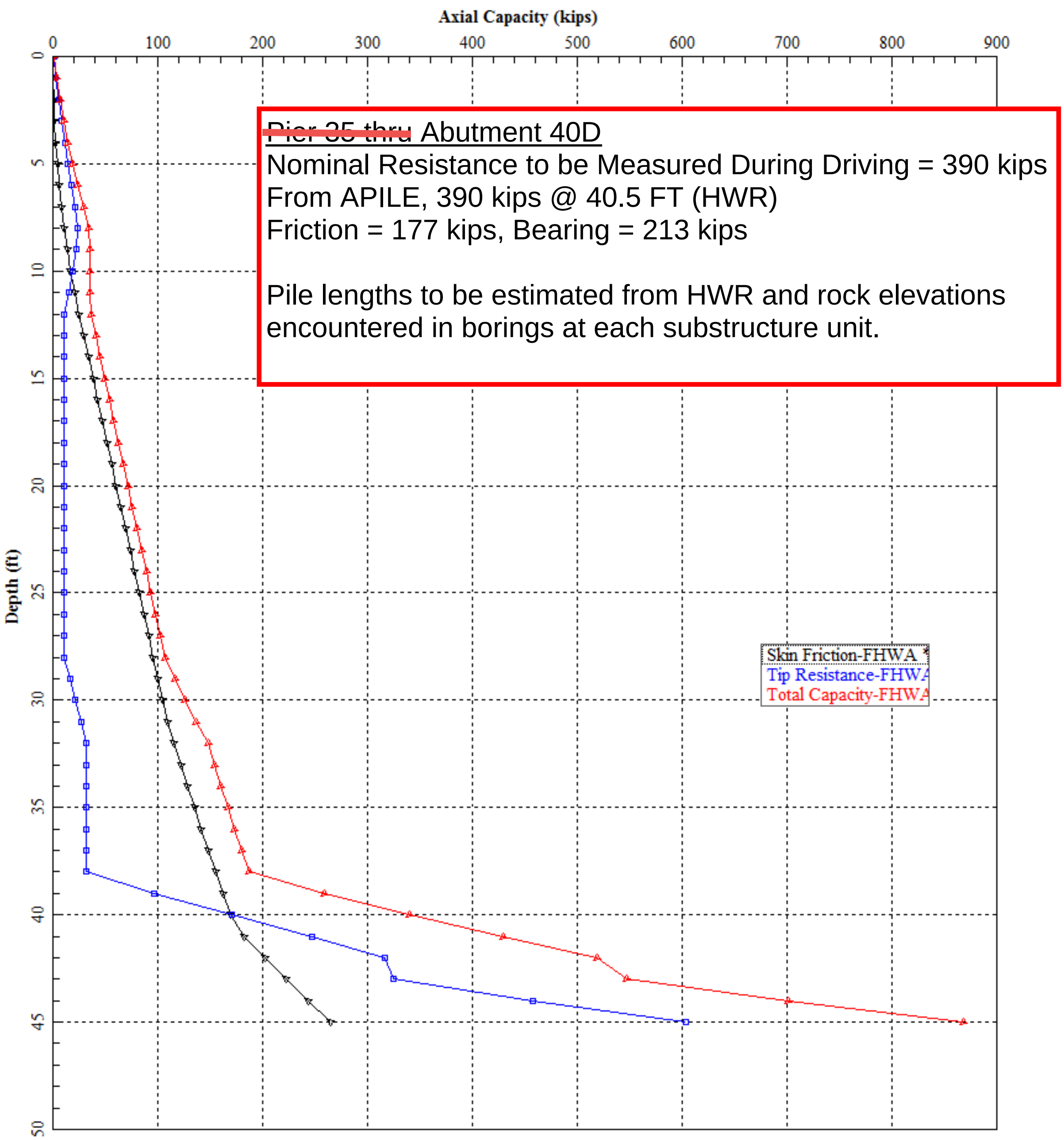
PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	1.4	1.4
1.00	0.2	2.9	3.0
2.00	0.7	5.9	6.6
3.00	1.5	8.8	10.4
4.00	2.7	11.8	14.5
5.00	4.2	14.7	19.0
6.00	6.1	17.7	23.8
7.00	8.3	20.6	28.9
8.00	10.8	23.6	34.4
9.00	13.7	21.8	35.6
10.00	16.9	18.8	35.8
11.00	20.8	15.0	35.8
12.00	25.2	10.9	36.2
13.00	29.6	10.9	40.6
14.00	34.1	10.9	45.0
15.00	38.5	10.9	49.4
16.00	42.9	10.9	53.8
17.00	47.3	10.9	58.2
18.00	51.7	10.9	62.6
19.00	56.1	10.9	67.0
20.00	60.5	10.9	71.4
21.00	64.9	10.9	75.8
22.00	69.3	10.9	80.2
23.00	73.7	10.9	84.6
24.00	78.1	10.9	89.0
25.00	82.5	10.9	93.5
26.00	86.9	10.9	97.9
27.00	91.3	10.9	102.3
28.00	95.7	10.9	106.7
29.00	100.1	15.9	116.1
30.00	104.5	21.5	126.1
31.00	109.7	27.1	136.8
32.00	115.8	32.1	147.9
33.00	122.0	32.1	154.0
34.00	128.3	32.1	160.4
35.00	134.8	32.1	166.9
36.00	141.4	32.1	173.5
37.00	148.2	32.1	180.3
38.00	155.1	32.1	187.2
39.00	162.2	96.6	258.8
40.00	169.5	170.6	340.1
41.00	182.9	246.6	429.4
42.00	202.5	316.9	519.4
43.00	222.7	324.2	546.9
44.00	243.3	457.3	700.6
45.00	264.4	603.5	867.9

PILES MOSTLY END BEARING IN HWR


Axial Capacity (kips)
0
100
200
300
400
500
600
700
800
900
Depth (ft)
0
5
10
15
20
25
30
35
40
45
50
~~Pier 35 thru~~ Abutment 40D
Nominal Resistance to be Measured During Driving = 390 kips
From APILE, 390 kips @ 40.5 FT (HWR)
Friction = 177 kips, Bearing = 213 kips
Pile lengths to be estimated from HWR and rock elevations encountered in borings at each substructure unit.
Skin Friction-FHWA
Tip Resistance-FHWA
Total Capacity-FHWA

Checked by: R.Fernós 12/4/17

AXIALLY LOADING PILE ANALYSIS PROGRAM - APILE
VERSION 2013 - (C) COPYRIGHT ENSOFT,INC.,1987-2013.

Wilm. Viaduct, Ramp D, Abutment 40D with MSE Embankment

DESIGNER : C. Troxel

DATE : 11/6/2017

Check downdrag load from settlement and possible consolidation of clay layer at 26 feet below top of pile. Assume open pile section.

PILE PROPERTIES :

PERIMETER OF PILE WITH NONCIRCULAR SECTION= 47.00 IN.
TIP AREA OF PILE WITH NONCIRCULAR SECTION = 0.11 SQF
OUTSIDE DIAMETER OF CIRCULAR PILE = 0.00 IN.
INTERNAL DIAMETER OF CIRCULAR PILE = 0.00 IN.
PILE LENGTH = 30.00 FT.
MODULUS OF ELASTICITY = 0.290E+08 PSI

LENGTH OF SURFACE SECTION WITH ZERO SKIN FRICTION = 0.00 FT.
INCREMENT OF PILE LENGTH USED IN COMPUTATION = 1.00 FT.

SOIL INFORMATIONS :

	DEPTH FT.	SOIL TYPE	LATERAL EARTH PRESSURE	EFFECTIVE UNIT WEIGHT LB/CF	FRICTION ANGLE DEGREES	BEARING CAPACITY FACTOR
MSE Fill	0.00	SAND	0.00	110.00	32.00	0.00
	16.00	SAND	0.00	110.00	32.00	0.00
Ex. Fill	16.00	SAND	0.00	115.00	32.00	0.00
	26.00	SAND	0.00	115.00	32.00	0.00
Silt/Clay	26.00	CLAY	0.00	58.00	0.00	0.00
	40.00	CLAY	0.00	58.00	0.00	0.00
	40.00	SAND	0.00	63.00	40.00	0.00
	45.00	SAND	0.00	63.00	40.00	0.00
	45.00	CLAY	0.00	78.00	0.00	0.00
	60.00	CLAY	0.00	78.00	0.00	0.00

MAXIMUM UNIT FRICTION KSF	MAXIMUM UNIT BEARING KSF	UNDISTURB SHEAR STRENGTH KSF	REMOLDED SHEAR STRENGTH KSF	BLOW COUNT	UNIT SKIN FRICTION KSF	UNIT END BEARING KSF
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	1.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	1.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	100.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	100.00	0.00	0.00	0.00	0.00

SET MAXIMUM UNIT FRICTION AND MAXIMUM UNIT BEARING
TO BE 0.10E+08 BECAUSE THE USER DOES NOT PLAN TO
LIMIT THE COMPUTED DATA.

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*********************
* COMPUTATION RESULT *
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*********************
*  FED. HWY. METHOD  *
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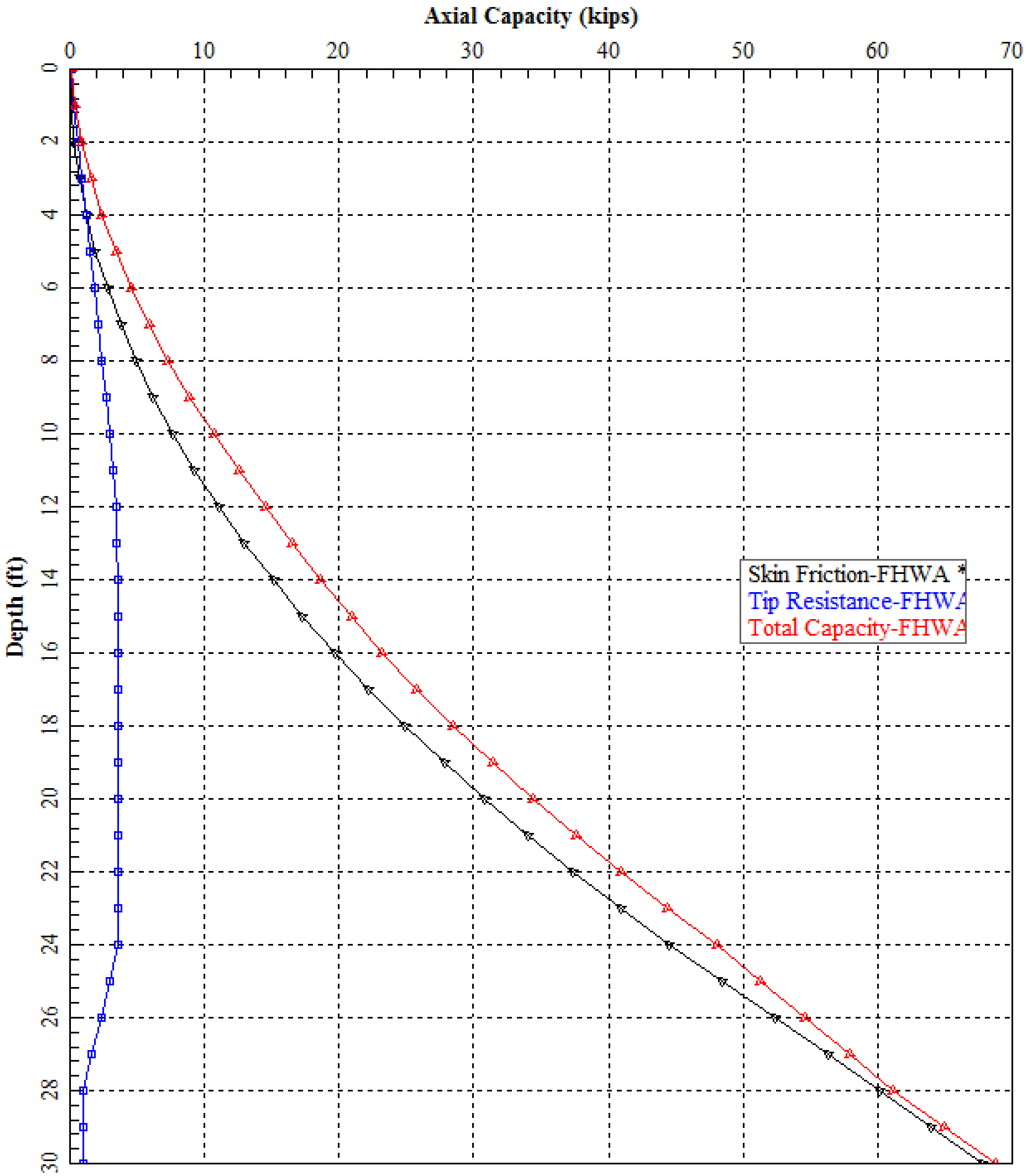
PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	0.1	0.1
1.00	0.1	0.3	0.4
2.00	0.3	0.6	0.9
3.00	0.7	0.9	1.6
4.00	1.2	1.2	2.4
5.00	1.9	1.5	3.4
6.00	2.8	1.8	4.6
7.00	3.8	2.1	5.9
8.00	4.9	2.4	7.3
9.00	6.2	2.7	8.9
10.00	7.7	3.0	10.7
11.00	9.3	3.2	12.6
12.00	11.1	3.4	14.5
13.00	13.0	3.5	16.5
14.00	15.1	3.6	18.6
15.00	17.3	3.6	20.9
16.00	19.7	3.6	23.2
17.00	22.2	3.6	25.8
18.00	24.9	3.6	28.5
19.00	27.8	3.6	31.4
20.00	30.8	3.6	34.4
21.00	34.0	3.6	37.6
22.00	37.4	3.6	40.9
23.00	40.9	3.6	44.4
24.00	44.5	3.6	48.1
25.00	48.4	3.0	51.3
26.00	52.4	2.3	54.6
27.00	56.3	1.6	57.9
28.00	60.1	1.0	61.1
29.00	64.0	1.0	65.0
30.00	67.8	1.0	68.8

AN ASTERISK WILL BE PLACED IN THE END-BEARING COLUMN
IF THE TIP RESISTANCE IS CONTROLLED BY THE FRICTION
OF SOIL PLUG INSIDE AN OPEN-ENDED PIPE PILE.

<u>Abutment 40D Downdrag Estimate</u>
Skin Friction at 26', top of clay layer = 52.4 kips
Load factor for downdrag (Nordlund Method) = 1.05
Factored downdrag load = 1.05*52.4 kips = 55 kips

This is less than anticipated live load at the abutment. Per DelDOT BDM 210.7.1.6.2-Downdrag, downdrag load not considered.


Axial Capacity (kips)
0
10
20
30
40
50
60
70
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
Skin Friction-FHWA *
Tip Resistance-FHWA
Total Capacity-FHWA

Checked by: R.Fernós 12/4/17

AXIALLY LOADING PILE ANALYSIS PROGRAM - APILE
VERSION 2013 - (C) COPYRIGHT ENSOFT,INC.,1987-2013.

Wilm. Viaduct, Bridge 750 Abutment A

DESIGNER : C. Troxel

DATE : 9/28/2017

Plugged Pile Tip

PILE PROPERTIES :

PERIMETER OF PILE WITH NONCIRCULAR SECTION	=	47.60 IN.
TIP AREA OF PILE WITH NONCIRCULAR SECTION	=	0.97 SQF
OUTSIDE DIAMETER OF CIRCULAR PILE	=	0.00 IN.
INTERNAL DIAMETER OF CIRCULAR PILE	=	0.00 IN.
PILE LENGTH	=	25.00 FT.
MODULUS OF ELASTICITY	=	0.290E+08 PSI

LENGTH OF SURFACE SECTION WITH ZERO SKIN FRICTION	=	0.00 FT.
INCREMENT OF PILE LENGTH USED IN COMPUTATION	=	1.00 FT.

SOIL INFORMATIONS :

DEPTH FT.	SOIL TYPE	LATERAL EARTH PRESSURE	EFFECTIVE UNIT WEIGHT LB/CF	FRICTION ANGLE DEGREES	BEARING CAPACITY FACTOR
0.00	SAND	0.00	120.00	30.00	0.00
10.00	SAND	0.00	120.00	30.00	0.00
10.00	SAND	0.00	58.00	32.00	0.00
20.00	SAND	0.00	58.00	32.00	0.00
20.00	SAND	0.00	63.00	40.00	0.00
25.00	SAND	0.00	63.00	40.00	0.00
25.00	CLAY	0.00	78.00	0.00	0.00
40.00	CLAY	0.00	78.00	0.00	0.00

MAXIMUM UNIT FRICTION KSF	MAXIMUM UNIT BEARING KSF	UNDISTURB SHEAR STRENGTH KSF	REMOLDED SHEAR STRENGTH KSF	BLOW COUNT	UNIT SKIN FRICTION KSF	UNIT END BEARING KSF
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	100.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	100.00	0.00	0.00	0.00	0.00

SET MAXIMUM UNIT FRICTION AND MAXIMUM UNIT BEARING
TO BE 0.10E+08 BECAUSE THE USER DOES NOT PLAN TO
LIMIT THE COMPUTED DATA.

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*********************
* COMPUTATION RESULT *
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                    *  FED. HWY. METHOD  *
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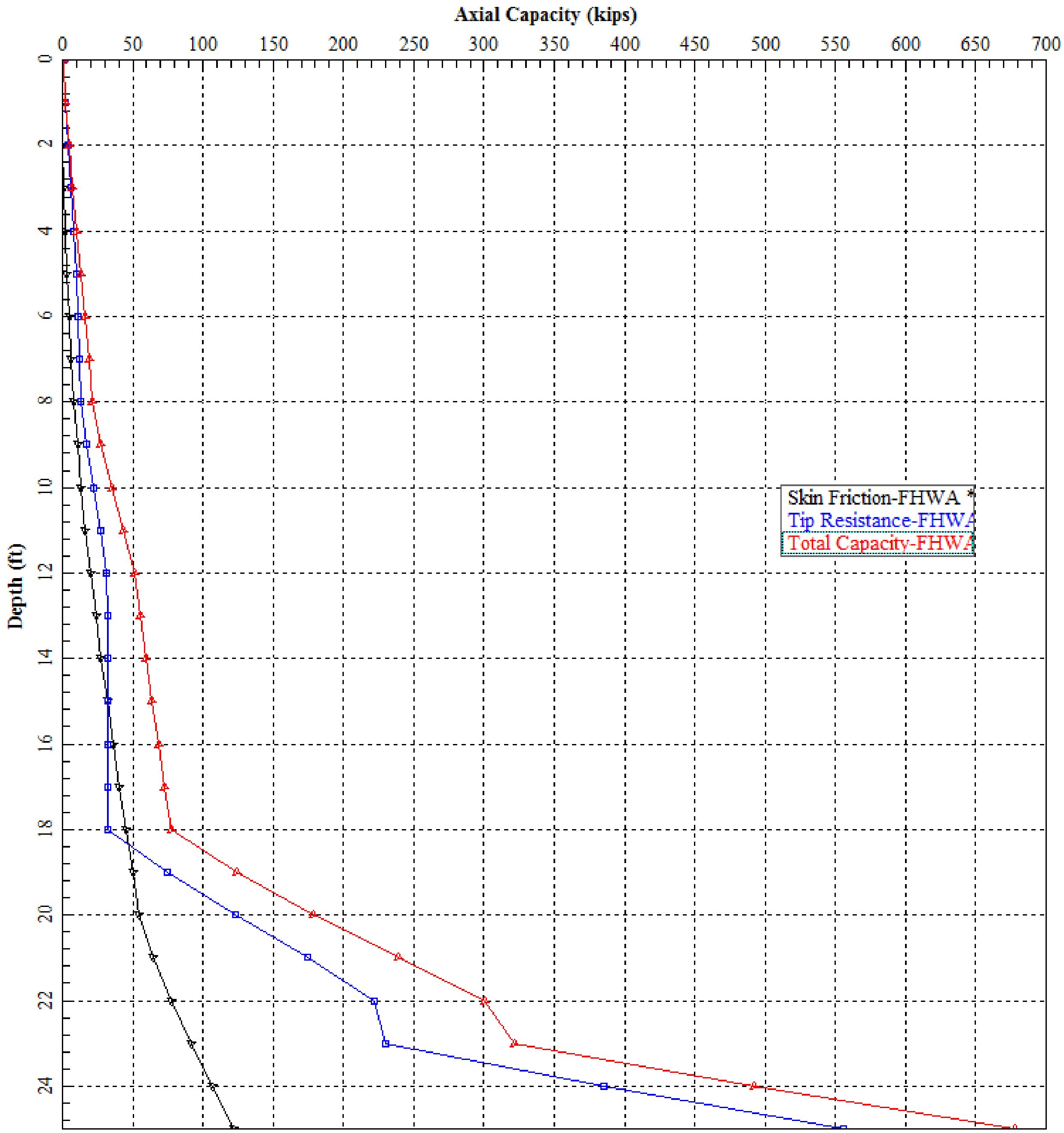
PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	1.0	1.0
1.00	0.1	2.0	2.1
2.00	0.5	4.1	4.6
3.00	1.2	6.1	7.3
4.00	2.1	8.1	10.2
5.00	3.3	10.0	13.3
6.00	4.8	11.5	16.3
7.00	6.5	12.5	18.9
8.00	8.5	12.9	21.3
9.00	10.7	17.0	27.7
10.00	13.2	21.9	35.1
11.00	16.3	26.9	43.3
12.00	20.0	31.5	51.5
13.00	23.8	32.0	55.7
14.00	27.7	32.1	59.8
15.00	31.9	32.1	63.9
16.00	36.1	32.1	68.2
17.00	40.6	32.1	72.7
18.00	45.2	32.1	77.3
19.00	50.0	74.3	124.3
20.00	54.9	123.4	178.3
21.00	64.2	174.4	238.6
22.00	78.0	222.4	300.3
23.00	92.2	229.7	321.9
24.00	106.8	385.1	492.0
25.00	122.0	556.2	678.2

Bridge 1-750 Abutment A
Nominal Resistance to be Measured During Driving = 390 kips
From APILE, 390 kips @ 23.5 FT (HWR)
Friction = 98 kips, Bearing = 292 kips

T/Pile EL ~ 35.5 FT
T/ Ex. Ground ~ 26 FT
B/Pile ~ 2.5 FT
Pile Length ~ 33 FT


Axial Capacity (kips)
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
Skin Friction-FHWA
Tip Resistance-FHWA
Total Capacity-FHWA

Checked by: R.Fernós 12/4/17

AXIALLY LOADING PILE ANALYSIS PROGRAM - APILE
VERSION 2013 - (C) COPYRIGHT ENSOFT,INC.,1987-2013.

Wilm. Viaduct, Bridge 750 Abutment B

DESIGNER : C. Troxel

DATE : 9/26/2017

Plugged Pile Tip

PILE PROPERTIES :

PERIMETER OF PILE WITH NONCIRCULAR SECTION= 47.60 IN.
TIP AREA OF PILE WITH NONCIRCULAR SECTION = 0.97 SQF
OUTSIDE DIAMETER OF CIRCULAR PILE = 0.00 IN.
INTERNAL DIAMETER OF CIRCULAR PILE = 0.00 IN.
PILE LENGTH = 27.00 FT.
MODULUS OF ELASTICITY = 0.290E+08 PSI

LENGTH OF SURFACE SECTION WITH ZERO SKIN FRICTION = 0.00 FT.
INCREMENT OF PILE LENGTH USED IN COMPUTATION = 1.00 FT.

SOIL INFORMATIONS :

DEPTH FT.	SOIL TYPE	LATERAL EARTH PRESSURE	EFFECTIVE UNIT WEIGHT LB/CF	FRICTION ANGLE DEGREES	BEARING CAPACITY FACTOR
0.00	SAND	0.00	120.00	30.00	0.00
5.00	SAND	0.00	120.00	30.00	0.00
5.00	SAND	0.00	58.00	32.00	0.00
15.00	SAND	0.00	58.00	34.00	0.00
15.00	SAND	0.00	63.00	40.00	0.00
25.00	SAND	0.00	63.00	40.00	0.00
25.00	CLAY	0.00	78.00	0.00	0.00
60.00	CLAY	0.00	78.00	0.00	0.00

MAXIMUM UNIT FRICTION KSF	MAXIMUM UNIT BEARING KSF	UNDISTURB SHEAR STRENGTH KSF	REMOLDED SHEAR STRENGTH KSF	BLOW COUNT	UNIT SKIN FRICTION KSF	UNIT END BEARING KSF
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	0.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	100.00	0.00	0.00	0.00	0.00
0.10E+08	0.10E+08	100.00	0.00	0.00	0.00	0.00

SET MAXIMUM UNIT FRICTION AND MAXIMUM UNIT BEARING
TO BE 0.10E+08 BECAUSE THE USER DOES NOT PLAN TO
LIMIT THE COMPUTED DATA.

```
*********************
* COMPUTATION RESULT *
*********************

                    *********************
                    *  FED. HWY. METHOD  *
                    *********************
```

PILE PENETRATION FT.	TOTAL SKIN FRICTION KIP	END BEARING KIP	ULTIMATE CAPACITY KIP
0.00	0.0	1.0	1.0
1.00	0.1	2.0	2.1
2.00	0.5	4.1	4.6
3.00	1.2	6.1	7.3
4.00	2.1	9.2	11.3
5.00	3.3	12.4	15.7
6.00	4.9	15.7	20.6
7.00	6.9	18.9	25.8
8.00	9.1	21.2	30.3
9.00	11.5	23.7	35.3
10.00	14.2	26.5	40.6
11.00	17.1	29.3	46.4
12.00	20.2	32.4	52.6
13.00	23.6	35.5	59.1
14.00	27.3	61.8	89.0
15.00	31.2	91.8	123.0
16.00	37.8	122.9	160.7
17.00	47.2	152.4	199.5
18.00	57.0	159.7	216.7
19.00	67.2	167.1	234.3
20.00	78.0	174.4	252.4
21.00	89.2	181.8	270.9
22.00	100.8	189.1	289.9
23.00	113.0	196.5	309.4
24.00	125.5	359.7	485.2
25.00	138.6	539.6	678.2
26.00	289.2	717.6	1006.7
27.00	577.1	875.0	1452.1

AN ASTERISK WILL BE PLACED IN THE END-BEARING COLUMN
IF THE TIP RESISTANCE IS CONTROLLED BY THE FRICTION
OF SOIL PLUG INSIDE AN OPEN-ENDED PIPE PILE.

<u>Bridge 1-750 Abutment B</u>
Nominal Resistance to be Measured During Driving = 390 kips
From APILE, 390 kips @ 23.5 FT (HWR)
Friction = 119 kips, Bearing = 271 kips

T/Pile EL ~ 43.0 FT
T/ Ex. Ground ~ 34 FT
B/Pile ~ 10.5 FT
Pile Length ~ 32.5 FT

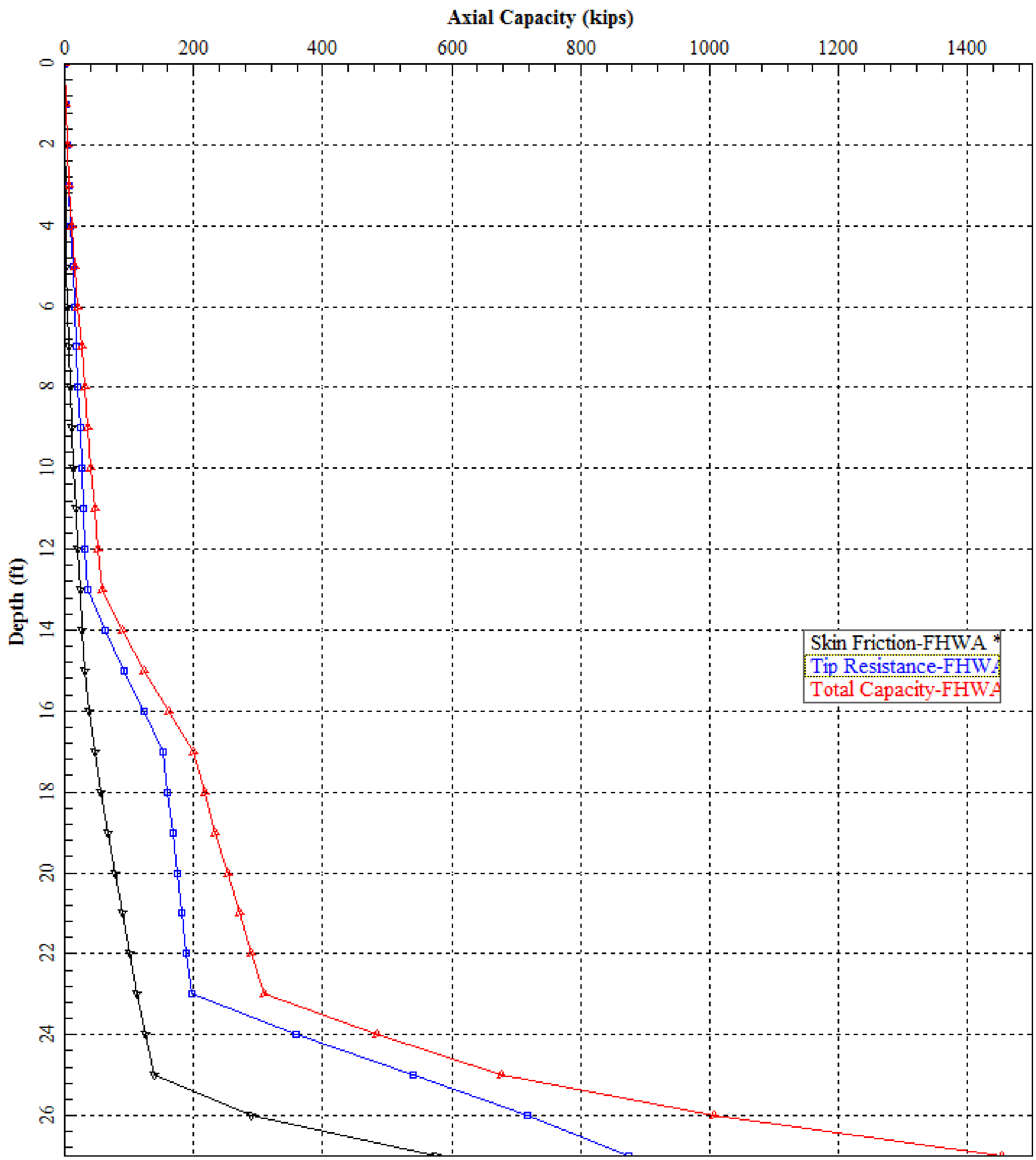

Axial Capacity (kips)
0
200
400
600
800
1000
1200
1400
Depth (ft)
0
2
4
6
8
10
12
14
16
18
20
22
24
26
Skin Friction-FHWA *
Tip Resistance-FHWA
Total Capacity-FHWA

Table 1: D30-02 Hammer (66 kip-ft), H12x53 pile and 40 ft pile length.

Whitman, Requardt and Associates, LLP — Feb 05 2018
Ramp D, South Abutment, Wilm. ViaductGRL — GRLWEAP(TM) Version 2003

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Blow Rate bpm	ENTHRU kips-ft
5.0	16.3	1.3	15.0	-1.0	0.000	0.000	63.3	0.0
10.0	34.7	4.7	30.0	2.1	12.466	-0.067	60.3	33.6
15.0	162.3	12.3	150.0	13.8	29.115	0.000	46.3	25.4
20.0	224.9	24.9	200.0	20.1	35.138	-0.301	44.4	25.1
25.0	291.5	41.5	250.0	27.0	39.201	-1.727	43.1	25.7
30.0	382.5	62.5	320.0	38.0	44.270	-2.834	41.3	27.9
35.0	437.6	87.6	350.0	47.9	46.912	-3.053	40.6	28.4
40.0	466.3	116.3	350.0	54.8	47.022	-2.320	40.4	28.4

Total Continuous Driving Time 21.00 minutes; Total Number of Blows 878

Max comp. stress, upper energy limit.

Table 2: D22 Hammer (40 kip-ft), H12x53 pile and 40 ft pile length.

Whitman, Requardt and Associates, LLP — Nov 14 2017
Ramp D, South Abutment, Wilm. ViaductGRL — GRLWEAP(TM) Version 2003

Gain/Loss 1 at Shaft and Toe 1.000 / 1.000

Depth ft	Ultimate Capacity kips	Friction kips	End Bearing kips	Blow Count blows/ft	Comp. Stress ksi	Tension Stress ksi	Blow Rate bpm	ENTHRU kips-ft
5.0	16.3	1.3	15.0	2.0	8.647	0.000	72.2	19.1
10.0	34.7	4.7	30.0	3.3	13.421	0.000	67.2	20.2
15.0	162.3	12.3	150.0	20.2	23.865	-0.671	53.9	16.2
20.0	224.9	24.9	200.0	29.2	28.209	-1.446	51.5	16.5
25.0	291.5	41.5	250.0	40.0	32.639	-2.317	49.2	17.6
30.0	382.5	62.5	320.0	63.7	39.094	-3.976	46.6	19.0
35.0	437.6	87.6	350.0	89.6	41.212	-4.233	45.5	19.5
40.0	466.3	116.3	350.0	113.5	41.199	-3.373	45.3	19.4

Total Continuous Driving Time 32.00 minutes; Total Number of Blows 1524

High blow count, lower energy limit.

Checked by: R.Fernós 12/4/17

```
================================================================================

                  LPile Plus for Windows, Version 2013-07.004

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

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                              All Rights Reserved

================================================================================

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--------------------------------------------------------------------------------
                            Files Used for Analysis
--------------------------------------------------------------------------------

Path to file locations:        N:\31987-004\Engineering\Design\Geotechnical\
Ramp Relocation\Calculations\LPILE\
Name of input data file:       LPILE 750 ABUT A 12x53.lp7d
Name of output report file:    LPILE 750 ABUT A 12x53.lp7o
Name of plot output file:      LPILE 750 ABUT A 12x53.lp7p
Name of runtime messeage file: LPILE 750 ABUT A 12x53.lp7r


--------------------------------------------------------------------------------
                           Date and Time of Analysis
--------------------------------------------------------------------------------

               Date:  November 14, 2017     Time:  11:17:29


--------------------------------------------------------------------------------
                                 Problem Title
--------------------------------------------------------------------------------

Project Name: WV Ramp D
Job Number: 31987-004
Client: DelDOT
Engineer: C. Troxel
Description: Check of lateral loads vs. deflection for plumb piles


--------------------------------------------------------------------------------
                          Program Options and Settings
--------------------------------------------------------------------------------

Engineering Units of Input Data and Computations:
 - Engineering units are US Customary Units (pounds, feet, inches)

Analysis Control Options:
- Maximum number of iterations allowed                  =          500
- Deflection tolerance for convergence                  =   1.0000E-05 in
- Maximum allowable deflection                          =     100.0000 in
- Number of pile increments                             =          100

Loading Type and Number of Cycles of Loading:
 - Cyclic loading specified
```

```
 - Number of cycles of loading = 4652007308841189376

Computational Options:
 - Use unfactored loads in computations (conventional analysis)
 - Compute pile response under loading and nonlinear bending properties of pile
   (only if nonlinear pile properties are input)
 - Analysis uses p-y modification factors for p-y curves
 - Loading by lateral soil movements acting on pile not selected
 - Input of shear resistance at the pile tip not selected
 - Computation of pile-head foundation stiffness matrix not selected
 - Push-over analysis of pile not selected
 - Buckling analysis of pile not selected

Output Options:
 - No p-y curves to be computed and reported for user-specified depths
 - Values of pile-head deflection, bending moment, shear force, and
   soil reaction are printed for full length of pile.
 - Printing Increment (nodal spacing of output points) = 1


--------------------------------------------------------------------------------
                    Pile Structural Properties and Geometry
--------------------------------------------------------------------------------

Total number of pile sections                          =          1

Total length of pile                                   =      32.00 ft

Depth of ground surface below top of pile              =       0.00 ft

Pile diameter values used for p-y curve computations are defined using 2 points.

p-y curves are computed using pile diameter values interpolated with depth over
the length of the pile.

Point         Depth             Pile
                X             Diameter
                ft               in
-----       ---------        -----------
  1           0.00000         11.8000000
  2         32.000000         11.8000000


Input Structural Properties:
----------------------------

Pile Section No. 1:

   Section Type                                        =    Elastic Pile
   Cross-sectional Shape                               =     Weak H-Pile
   Section Length                                      =     32.00000 ft
   Flange Width                                        =     12.00000 in
   Section Depth                                       =     11.80000 in
   Flange Thickness                                    =      0.43500 in
   Web Thickness                                       =      0.43500 in
   Section Area                                        =     15.19455 Sq. in
   Moment of Inertia                                   =    125.35497 in^4
   Elastic Modulus                                     =    29000000. lbs/in^2


--------------------------------------------------------------------------------
                      Ground Slope and Pile Batter Angles
--------------------------------------------------------------------------------

Ground Slope Angle                                     =        0.000 degrees
                                                       =        0.000 radians

Pile Batter Angle                                      =        0.000 degrees
                                                       =        0.000 radians
```

```
--------------------------------------------------------------------------------
                       Soil and Rock Layering Information
--------------------------------------------------------------------------------
The soil profile is modelled using 5 layers

Layer 1 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer          =      0.0000 ft
   Distance from top of pile to bottom of layer       =    15.00000 ft
   Effective unit weight at top of layer              =   125.00000 pcf
   Effective unit weight at bottom of layer           =   125.00000 pcf
   Friction angle at top of layer                     =    34.00000 deg.
   Friction angle at bottom of layer                  =    34.00000 deg.
   Subgrade k at top of layer                         =      0.0000 pci
   Subgrade k at bottom of layer                      =      0.0000 pci

   NOTE: Internal default values for subgrade k will be computed for this soil layer.

Layer 2 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer          =    15.00000 ft
   Distance from top of pile to bottom of layer       =    20.00000 ft
   Effective unit weight at top of layer              =    48.00000 pcf
   Effective unit weight at bottom of layer           =    48.00000 pcf
   Friction angle at top of layer                     =    30.00000 deg.
   Friction angle at bottom of layer                  =    30.00000 deg.
   Subgrade k at top of layer                         =      0.0000 pci
   Subgrade k at bottom of layer                      =      0.0000 pci

   NOTE: Internal default values for subgrade k will be computed for this soil layer.

Layer 3 is Piedmont residual soil

   Distance from top of pile to top of layer          =    20.00000 ft
   Distance from top of pile to bottom of layer       =    30.00000 ft
   Effective unit weight at top of layer              =    58.00000 pcf
   Effective unit weight at bottom of layer           =    58.00000 pcf
   The type of field test is the Standard Penetration Test (SPT)
   SPT N60 at top of layer                            =    30.00000 blows/ft
   SPT N60 at bottom of layer                         =    30.00000 blows/ft

Layer 4 is sand, p-y criteria by Reese et al., 1974

   Distance from top of pile to top of layer          =    30.00000 ft
   Distance from top of pile to bottom of layer       =    35.00000 ft
   Effective unit weight at top of layer              =    63.00000 pcf
   Effective unit weight at bottom of layer           =    63.00000 pcf
   Friction angle at top of layer                     =    40.00000 deg.
   Friction angle at bottom of layer                  =    40.00000 deg.
   Subgrade k at top of layer                         =      0.0000 pci
   Subgrade k at bottom of layer                      =      0.0000 pci

   NOTE: Internal default values for subgrade k will be computed for this soil layer.

Layer 5 is weak rock, p-y criteria by Reese, 1997

   Distance from top of pile to top of layer          =    35.00000 ft
   Distance from top of pile to bottom of layer       =    40.00000 ft
   Effective unit weight at top of layer              =    90.00000 pcf
   Effective unit weight at bottom of layer           =    90.00000 pcf
   Uniaxial compressive strength at top of layer      =      20000. psi
   Uniaxial compressive strength at bottom of layer   =      20000. psi
   Initial modulus of rock at top of layer            =    4000000. psi
   Initial modulus of rock at bottom of layer         =    4000000. psi
   RQD of rock at top of layer                        =    50.00000 %
   RQD of rock at bottom of layer                     =    70.00000 %
   k rm of rock at top of layer                       =      0.0000
   k rm of rock at bottom of layer                    =      0.0000
```

(Depth of lowest soil layer extends 8.00 ft below pile tip)

Summary of Soil Properties

Layer Num.	Layer Soil Type (p-y Curve Criteria)	Layer Depth ft	Effective Unit Wt. pcf	Angle of Friction deg.	Uniaxial qu psi
1	Sand (Reese, et al.)	0.00	125.000	34.000	--
		15.000	125.000	34.000	--
2	Sand (Reese, et al.)	15.000	48.000	30.000	--
		20.000	48.000	30.000	--
3	Piedmont Residual	20.000	58.000	--	--
		30.000	58.000	--	--
4	Sand (Reese, et al.)	30.000	63.000	40.000	--
		35.000	63.000	40.000	--
5	Weak Rock	35.000	90.000	--	20000.
		40.000	90.000	--	20000.

Layer Num.	RQD % or GSI	kpy pci	Rock Mass Rock Emass psi	krm	In-situ Test Type	In-situ Test Property
1	--	default	--	--	--	--
	--	default	--	--	--	--
2	--	default	--	--	--	--
	--	default	--	--	--	--
3	--	--	--	--	SPT	30.000
	--	--	--	--	SPT	30.000
4	--	default	--	--	--	--
	--	default	--	--	--	--
5	50.000	--	4000000.	0.00	--	--
	70.000	--	4000000.	0.00	--	--

p-y Modification Factors for Group Action

Distribution of p-y modifiers with depth defined using 2 points

Point No.	Depth X ft	p-mult	y-mult
1	0.000	1.0000	1.0000
2	0.000	1.0000	1.0000

Loading Type

Cyclic loading criteria were used for computation of p-y curves for all analyses.

Number of cycles of loading = 1000

```
--------------------------------------------------------------------------------
               Pile-head Loading and Pile-head Fixity Conditions
--------------------------------------------------------------------------------

Number of loads specified = 2

Load    Load         Condition                Condition            Axial Thrust          Compute
 No.    Type             1                        2                 Force, lbs      Top y vs. Pile Length
-----   ----    --------------------   ----------------------   ----------------   ---------------------
   1     4      y =      0.60000 in    M =       0.0000 in-lbs          180000.             Yes
   2     1      V =       10294. lbs   M =       0.0000 in-lbs          180000.             Yes

V = perpendicular shear force applied to pile head
M = bending moment applied to pile head
y = lateral deflection relative to pile axis
S = pile slope relative to original pile batter angle
R = rotational stiffness applie to pile head
Axial thrust is assumed to be acting axially for all pile batter angles.


--------------------------------------------------------------------------------
    Computations of Nominal Moment Capacity and Nonlinear Bending Stiffness
--------------------------------------------------------------------------------

Axial thrust force values were determined from pile-head loading conditions

Number of Pile Sections Analyzed = 1

Pile Section No. 1:
-------------------
Moment-curvature properties were derived from elastic section properties




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

Pile-head conditions are Displacement and Moment (Loading Type 4)
Displacement of pile head                             =     0.600000 inches
Moment at pile head                                   =          0.0 in-lbs
Axial load at pile head                               =     180000.0 lbs


Output Summary for Load Case No. 1:

Pile-head deflection             =      0.6000000 inches
Computed slope at pile head      =     -0.0100497 radians
Maximum bending moment           =        466798. inch-lbs
Maximum shear force              =         10294. lbs
Depth of maximum bending moment  =      4.8000000 feet below pile head
Depth of maximum shear force     =       0.000000 feet below pile head
Number of iterations             =              6
Number of zero deflection points =              4



--------------------------------------------------------------------------------
            Pile-head Deflection vs. Pile Length for Load Case 1
--------------------------------------------------------------------------------

Boundary Condition Type 4, Deflection and Moment

Deflection =         0.60000 in
Moment     =              0. in- lb
Axial Load =         180000.  lb
```

Pile Length feet	Pile Head Deflection inches	Maximum Moment ln-lbs	Maximum Shear lbs
32.0000	0.6000000	466798.	10294.
30.4000	0.6000000	466806.	10294.
28.8000	0.6000000	466839.	10297.
27.2000	0.6000000	466799.	10297.
25.6000	0.6000000	466803.	10298.
24.0000	0.6000000	466870.	10300.
22.4000	0.6000000	466592.	10299.
20.8000	0.6000000	466714.	10299.
19.2000	0.6000000	466641.	10298.
17.6000	0.6000000	466224.	10293.
16.0000	0.6000000	466123.	10289.
14.4000	0.6000000	466083.	10289.
12.8000	0.6000000	463285.	10249.
11.2000	0.6000000	439190.	9865.9564542
9.6000	0.6000000	360222.	-9621.9498315
8.0000	0.6000000	231884.	-8606.3245679
6.4000	0.6000000	112121.	-6521.4107156
4.8000	0.6000000	29149.	-4983.2445784

Computed Values of Pile Loading and Deflection
for Lateral Loading for Load Case Number 2

Pile-head conditions are Shear and Moment (Loading Type 1)

```
Shear force at pile head                          =      10294.0 lbs
Applied moment at pile head                       =          0.0 in-lbs
Axial thrust load on pile head                    =     180000.0 lbs
```

Output Summary for Load Case No. 2:

```
Pile-head deflection             =      0.6000237 inches
Computed slope at pile head      =     -0.0100501 radians
Maximum bending moment           =        466813. inch-lbs
Maximum shear force              =         10294. lbs
Depth of maximum bending moment  =      4.8000000 feet below pile head
Depth of maximum shear force     =       0.000000 feet below pile head
Number of iterations             =             13
Number of zero deflection points =              4
```

Pile-head Deflection vs. Pile Length for Load Case 2

Boundary Condition Type 1, Shear and Moment

```
Shear      =         10294.  lb
Moment     =             0. in- lb
Axial Load =        180000.  lb
```

Pile Length feet	Pile Head Deflection inches	Maximum Moment ln-lbs	Maximum Shear lbs
32.0000	0.6000237	466813.	10294.
30.4000	0.6000267	466823.	10294.
28.8000	0.5997047	466641.	10294.
27.2000	0.5996968	466596.	10294.

25.6000	0.5996487	466567.	10294.
24.0000	0.5994698	466514.	10294.
22.4000	0.5995782	466308.	10294.
20.8000	0.5995581	466422.	10294.
19.2000	0.5996301	466395.	10294.
17.6000	0.6000585	466262.	10294.
16.0000	0.6004455	466419.	10294.
14.4000	0.6004390	466375.	10294.
12.8000	0.6040603	465945.	10294.
11.2000	0.6422574	464198.	10294.
9.6000	0.8737422	472766.	-13300.

Summary of Pile Response(s)

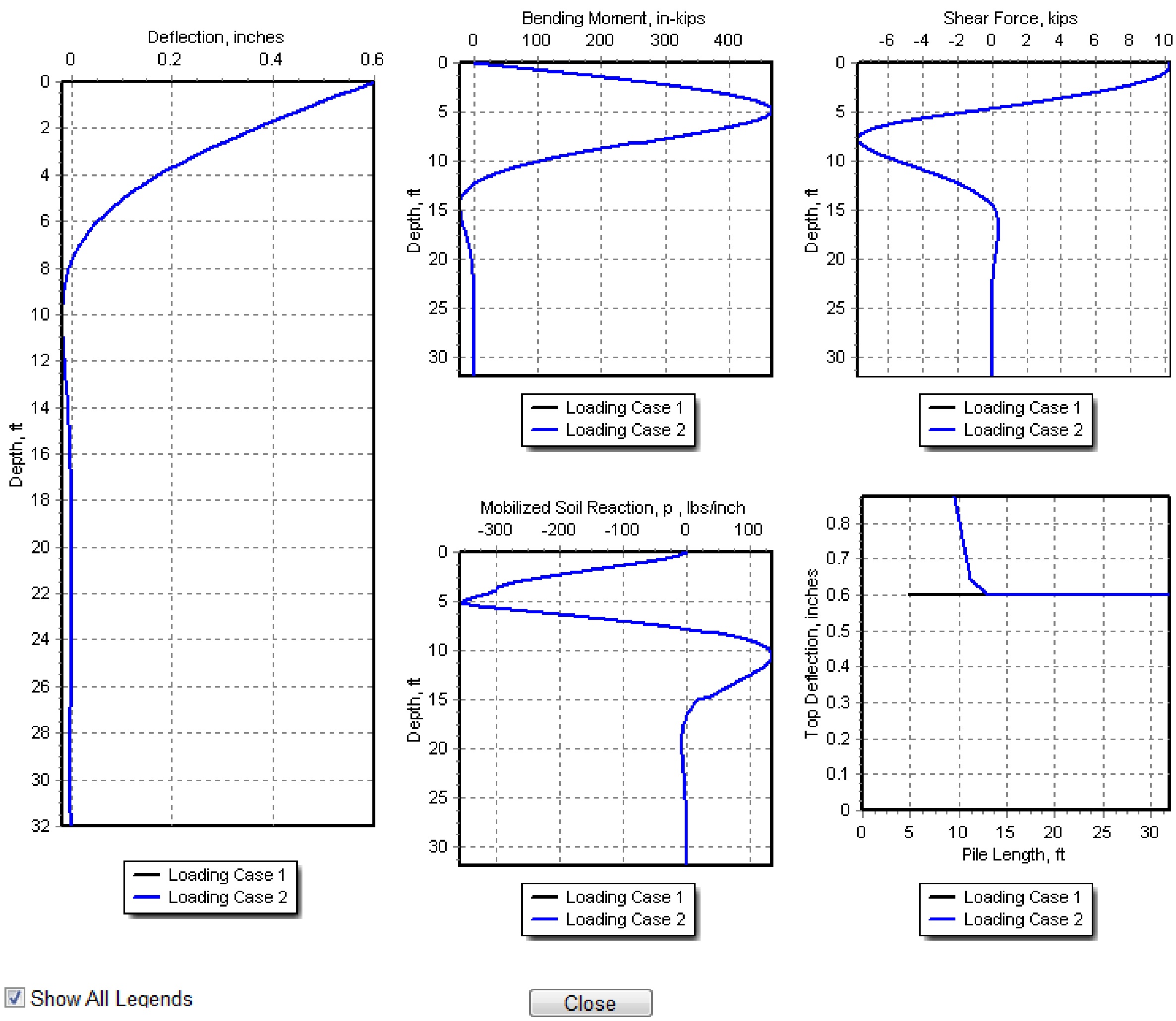
Definitions of Pile-head Loading Conditions:

Load Type 1: Load 1 = Shear, lbs, and Load 2 = Moment, in-lbs
Load Type 2: Load 1 = Shear, lbs, and Load 2 = Slope, radians
Load Type 3: Load 1 = Shear, lbs, and Load 2 = Rotational Stiffness, in-lbs/radian
Load Type 4: Load 1 = Top Deflection, inches, and Load 2 = Moment, in-lbs
Load Type 5: Load 1 = Top Deflection, inches, and Load 2 = Slope, radians

Load Case No.	Load Type No.	Pile-head Condition 1 V(lbs) or y(inches)	Pile-head Condition 2 in-lb, rad., or in-lb/rad.	Axial Loading lbs	Pile-head Deflection inches	Maximum Moment in Pile in-lbs
1	4	y = 0.6000	M = 0.000	180000.	0.60000000	466798.
2	1	V = 10294.	M = 0.000	180000.	0.60002366	466813.

Load Case No.	Load Type No.	Maximum Shear in Pile lbs	Pile-head Rotation radians
1	4	10294.	-0.01004974
2	1	10294.	-0.01005009

The analysis ended normally.


Deflection, inches
Depth, ft
Loading Case 1
Loading Case 2
Bending Moment, in-kips
Depth, ft
Loading Case 1
Loading Case 2
Shear Force, kips
Depth, ft
Loading Case 1
Loading Case 2
Mobilized Soil Reaction, p , lbs/inch
Depth, ft
Loading Case 1
Loading Case 2
Top Deflection, inches
Pile Length, ft
Loading Case 1
Loading Case 2
Show All Legends
Close

MICROPILE LENGTH - WEATHERED ROCK BOND

Design: CFT (2/18/2019)

Review: **R.Fernos (2/11/19)**

MICROPILE DIMENSIONS

OD after corrosion	9.455	in
Surface Area - Micropile	2.48	ft^2/ft

MICROPILE RESISTANCE - GEOTECHNICAL

Target Fact. Comp. Res.	200	kips
Comp. Resistance Factor[1]	0.55	--
Required Nominal Res.	364	kips
Tensile Resistance Factor[1]	0.55	--
Factored Tensile Resistance[3]	200	kips

GROUT-TO-GROUND RESISTANCE

	Res/HWR	Rock
Bond per Unit Area (PSI)[2]		50
Bond per Unit Area (KSF)		7.2
Bond per Unit Length (Kips/FT)		17.8
Req'd Bond Length (FT)		20
Req'd Plunge Length (FT)		0

MICROPILE LENGTH: PIERS 34D to 39D

			UNIT	PIER 34D	PIER 35D	PIER 36D	PIER 37D	PIER 38D	PIER 39D
		Boring	--	R1	R2	R3/R4	R4/R5	R5	R5/207/R6
1	ELEVATIONS	Ground Surface EL	FT	35	38	38	38	38	43
2		Bot/Pile Cap EL	FT	28	31	33	33.25	33	31.75
3		Approx. Top/Rock EL	FT	-5	-3	3	3	5	5
4		Casing Tip EL	FT	-5	-3	3	3	5	5
5		Micropile Tip EL	FT	-25	-23	-17	-17	-15	-15
6	LENGTHS	Footing Embedment	FT	1	1	1	1	1	1
7		Cased Length	FT	34	35	31	31	29	28
8		Uncased (Bond) Length	FT	20	20	20	20	20	20
9		Micropile Length	FT	54	55	51	52	49	48

Average Length: 52

REFERENCES/NOTES

1) AASHTO. (2014). *AASHTO LRFD Bridge Design Specifications.* 7th Edition. Washington, D.C. Publication Code: LRFDUS-7.

2) FHWA. (2005). Micropile Design and Construction - Reference Manual, Federal Highway Administration Reference No. FHWA-NHI-039.

3) **Resistances are maximum geotechnical values, resistance may be limited by structural capacity of micropile.

Table 5-3. Summary of Typical α_{bond} (Grout-to-Ground Bond) Values for Micropile Design.

Soil / Rock Description	Grout-to-Ground Bond Ultimate Strengths, kPa (psi)			
	Type A	Type B	Type C	Type D
Silt & Clay (some sand) (soft, medium plastic)	35-70 (5-10)	35-95 (5-14)	50-120 (5-17.5)	50-145 (5-21)
Silt & Clay (some sand) (stiff, dense to very dense)	50-120 (5-17.5)	70-190 (10-27.5)	95-190 (14-27.5)	95-190 (14-27.5)
Sand (some silt) (fine, loose-medium dense)	70-145 (10-21)	70-190 (10-27.5)	95-190 (14-27.5)	95- 240 (14-35)
Sand (some silt, gravel) (fine-coarse, med.-very dense)	95-215 (14-31)	120-360 (17.5-52)	145-360 (21-52)	145-385 (21-56)
Gravel (some sand) (medium-very dense)	95-265 (14-38.5)	120-360 (17.5-52)	145-360 (21-52)	145-385 (21-56)
Glacial Till (silt, sand, gravel) (medium-very dense, cemented)	95-190 (14-27.5)	95-310 (14-45)	120-310 (17.5-45)	120-335 (17.5-48.5)
Soft Shales (fresh-moderate fracturing, little to no weathering)	205-550 (30-80)	N/A	N/A	N/A
Slates and Hard Shales (fresh-moderate fracturing, little to no weathering)	515-1,380 (75-200)	N/A	N/A	N/A
Limestone (fresh-moderate fracturing, little to no weathering)	1,035-2,070 (150-300)	N/A	N/A	N/A
Sandstone (fresh-moderate fracturing, little to no weathering)	520-1,725 (75.5-250)	N/A	N/A	N/A
Granite and Basalt (fresh-moderate fracturing, little to no weathering)	1,380-4,200 (200-609)	N/A	N/A	N/A

Type A: Gravity grout only
Type B: Pressure grouted through the casing during casing withdrawal
Type C: Primary grout placed under gravity head, then one phase of secondary "global" pressure grouting
Type D: Primary grout place
grouting

Micropile Bond Estimates

Assume 50 psi for weathered rock and rock combination. This is the high end for pressure grouted sand and gravel bonds and low end for rock.

Micropile Geotechnical and Structural Resistance

Micropile resistance based on *2013 AASHTO LRFD Bridge Specification*, section 10.9 - Micropiles.

Micropile Dimensions and Strength

$D_0 := 9.625\ in$ — Micropile OD at install, 9-5/8"

$t_0 := 0.472\ in$ — Casing wall thickness at install

$cor := 0.085\ in$ — Assumed corrosion loss

$D := D_0 - 2\ cor = 9.455\ in$ — Micropile OD after corrosion loss

$t := t_0 - cor = 0.387\ in$ — Casing thickness after corrosion loss

$A_b := 2.25\ in^2$ — Area of steel #14 threaded bar

$f_y := 75\ ksi$ — Yield strength of reinforcing bar, grade 75

$f'_c := 4000\ psi$ — Compressive strength of grout

Grout-to-Ground Resistance

$\alpha_{b\_hwr} := 50\ psi = 7.2\ ksf$ — Bond zone is mix of weathred rock and rock. Assume upper end of pressure grouted sand/gravel and lower end of rock.

$R_f := 200\ kip$ — Factored micropile resistance, compression

$\varphi_{stat} := 0.55$ — Resistance factor for micropile in compression with successful load test, Table 10.5.5.2.5-1

$$L_{b\_soil} := \frac{R_f}{\varphi_{stat} \cdot \alpha_{b\_hwr} \cdot \pi \cdot D} = 20.4\ ft$$

Required bond length in weathered rock and rock mix

Structural Resistance

Cased length in axial compression (CC)

$A_g := \frac{\pi}{4} \left(D_0 - 2\ t_0\right)^2 = 59.2\ in^2$ Gross area of concrete

$A_b = 2.25\ in^2$ Area of steel bar

$A_c := (D - t) \cdot \pi \cdot t = 11.025\ in^2$ Area of steel casing

$R_{n\_CC} := 0.85\ \left(0.85 \cdot f'_c \cdot \left(A_g - A_b\right) + f_y \cdot \left(A_b + A_c\right)\right) = 1011\ kip$ Nominal structural resistance, cased compression. Eq. 10.9.3.10.2a-2

$\varphi_c := 0.75$ Resistance factor for micropile in compression Table 10.5.5.2.5-2

$R_{CC} := \varphi_c \cdot R_{n\_CC} = 758\ kip$ Factored structural resistance, cased compression

Uncased length in axial compression (CU)

$A_{g\_bond} := \frac{\pi}{4}\ D_0^{\ 2} = 72.76\ in^2$ Area of micropile bond. Assume bond area diameter is original diameter of casing.

$R_{n\_CU} := 0.85\ \left(0.85 \cdot f'_c\ \left(A_{g\_bond} - A_b\right) + f_y \cdot A_b\right) = 347\ kip$ Nominal structural resistance, uncased compression Eq. 10.9.3.10.2b-2

$R_{CU} := \varphi_c \cdot R_{n\_CU} = 260\ kip$ Factored structural resistance, uncased compression

Cased length in tension

$R_{n\_TC} := f_y \cdot \left(A_b + A_c\right) = 996\ kip$ Nominal structural resistance, cased tension Eq. 10.9.3.10.3a-2

$\varphi_{tu} := 0.80$ Resistance factor for micropile in tension Table 10.5.5.2.5-2

$R_{TC} := \varphi_{tu} \cdot R_{n\_TC} = 796\ kip$ Factored structural resistance, cased tension

Uncased length in tension

$R_{n\_TU} := f_y \cdot A_b = 169\ kip$ — Nominal structural resistance, cased tension
Eq. 10.9.3.10.3b-2

$\varphi_{tu} := 0.80$ — Resistance factor for micropile in tension
Table 10.5.5.2.5-2

$R_{TU} := \varphi_{tu} \cdot R_{n\_TU} = 135\ kip$ — Factored structural resistance, uncased tension

Micropile Test Loads

$RTL := 1.35 \cdot R_f = 270\ kip$ — Maximum test compression load for proof tests
This is greater than compressive resistance of uncased micropile
Will require plunge length

$\frac{RTL}{2} = 135\ kip$ — Maximum test tensile load for proof test reaction piles
This is 80% of the bar strength - okay

Plunge Load Transfer Length

$Q := 200\ kip$ — Maximum compression load on micropile

$T := 50\ kip$ — Maximum tensile load on micropile

$P_t := \max\left(Q - R_{CU}, T - R_{TU}, RTL - R_{CU}\right) = 9.59\ kip$ — Plunge load to be resisted by extending casing into bond length
Eq. 10.9.3.10.4-1

$\varphi_p := 0.60$ — Resistance factor for plunge length, assume as resistance factor for micropile in tension with load test
Table 10.5.5.2.5-1

$L_{p\_soil} := \frac{P_t}{\varphi_p \cdot \pi \cdot D \cdot \alpha_{b\_hwr}} = 0.9\ ft$ — Plunge length for casing in bond zone,
Plunge casing 1 foot

Table 4-5. Dimensions and Yield Strength of Common Micropile Pipe Types and Sizes.

API N-80 Pipe – Common Sizes					
Casing OD Wall(1), mm (in.)	139.7 (5.500)	139.7 (5.500)	177.8 (7.000)	177.8 (7)	244.5 (9.625)
Wall Thickness(1), mm (in.)	9.17 (0.361)	10.5 (0.415)	12.6 (0.498)	18.5 (0.73)	12.0 (0.472)
Area(2), mm^2 ($in.^2$)	3760 (5.83)	4280 (6.63)	6560 (10.2)	9280 (14.4)	8760 (13.6)
Yield Strength(3), kN (kip)	2,070 (466)	2,360 (530)	3,620 (814)	5,120 (1,151)	4,830 (1,086)
ASTM A519, A106 Pipe – Common Sizes(5)					
Casing OD Wall(1), mm (in.)	139.7 (5.50)	168.3 (6.625)	203.2 (8.00)	273.1 (10.75)	-
Wall Thickness(1), mm (in.)	12.7 (0.50)	12.7 (0.50)	12.7 (0.50)	16 (0.625)	-
Area(2), mm^2 ($in.^2$)	5,067 (7.85)	6,208 (9.62)	7,600 (11.8)	12,850 (19.9)	-
Yield Strength(3), kN (kip)	1,270 (286)	1,540 (346)	1,890 (425)	3,190 (717)	-

Notes: (1)Casing outside diameter (OD) and wall thickness (t) are nominal dimensions.
(2)Steel area is calculated as $A_s = (\pi/4) \times (OD^2 - ID^2)$.
(3)Nominal yield stress for API N-80 steel is F_y = 552 MPa (80 ksi).
(4)Nominal yield stress for ASTM A519 & A106 steel is F_y = 241 MPa (36 ksi).
(5)Other pipe sizes are manufactured but may not be readily available. Check for availability through suppliers.

Grade 75 All-Thread Rebar

Threads

Williams Grade 75 All-Thread Rebar has a cold rolled, continuous, rounded course thread form. Because of the full 360º concentric thread form, Williams All-Thread Rebar should only be bent under special provisions. Williams special thread (deformation) pattern projects ultra high relative rib area at 3 times that of conventional rebar. This provides for superior bond performance in concrete. Threads are available in both right and left hand. Grade 80-100 is available upon request.

Sizes

All-Thread Rebar is available in 11 diameters from #6 (20 mm) through #28 (89 mm). All diameters are available in continuous lengths up to 50’ (15.2 m).

Welding

Welding of All-Thread Rebar should be approached with caution since no specific provisions have been included to enhance its weldability. Refer to ANSI/AWS D1.4 for proper selections and procedures.

R61 Grade 75 All-Thread Rebar

ASTM A615*

Bar Designation Nominal Diameter & Pitch	Minimum Net Area Thru Threads	Minimum Ultimate Strength	Minimum Yield Strength	Nominal Weight	Approx. Thread Major Dia.	Part Number
#6 - 3/4” - 5 (19 mm)	0.44 in^2 (284 mm^2)	44 kips (196 kN)	33 kips (147 kN)	1.5 lbs./ft. (2.36 Kg/M)	7/8” (22.2 mm)	R61-06
#7 - 7/8” - 5 (22 mm)	0.60 in^2 (387 mm^2)	60 kips (267 kN)	45 kips (200 kN)	2.0 lbs./ft. (3.04 Kg/M)	1” (25.4 mm)	R61-07
#8 - 1” - 3-1/2 (25 mm)	0.79 in^2 (510 mm^2)	79 kips (351 kN)	59.3 kips (264 kN)	2.7 lbs./ft. (3.94 Kg/M)	1-1/8” (28.6 mm)	R61-08
#9 - 1-1/8” - 3-1/2 (29 mm)	1.00 in^2 (645 mm^2)	100 kips (445 kN)	75 kips (334 kN)	3.4 lbs./ft. (5.06 Kg/M)	1-1/4” (31.8 mm)	R61-09
#10 - 1-1/4” - 3 (32 mm)	1.27 in^2 (819 mm^2)	127 kips (565 kN)	95.3 kips (424 kN)	4.3 lbs./ft. (5.50 Kg/M)	1-3/8” (34.9 mm)	R61-10
#11 - 1-3/8” - 3 (36 mm)	1.56 in^2 (1006 mm^2)	156 kips (694 kN)	117 kips (521 kN)	5.3 lbs./ft. (7.85 Kg/M)	1-1/2” (38.1 mm)	R61-11
#14 - 1-3/4” - 3 (43 mm)	2.25 in^2 (1452 mm^2)	225 kips (1001 kN)	169 kips (750 kN)	7.65 lbs./ft. (11.8 Kg/M)	1-7/8” (47.6 mm)	R61-14
#18 - 2-1/4” - 3 (57 mm)	4.00 in^2 (2581 mm^2)	400 kips (1780 kN)	300 kips (1335 kN)	13.6 lbs./ft. (19.6 Kg/M)	2-7/16” (61.9 mm)	R61-18
#20 - 2-1/2” - 2-3/4 (64 mm)	4.91 in^2 (3168 mm^2)	491 kips (2184 kN)	368 kips (1637 kN)	16.7 lbs./ft. (24.8 Kg/M)	2-3/4” (69.9 mm)	R61-20
#24 - 3” - 2-3/4 (76 mm) *	6.82 in^2 (4400 mm^2)	682 kips (3034 kN)	512 kips (2277 kN)	24.0 lbs./ft. (35.8 Kg/M)	3-3/16” (81.0 mm)	R61-24
#28 - 3-1/2” - 2-3/4 (89 mm) *	9.61 in^2 (6200 mm^2)	961 kips (4274 kN)	720 kips (3206 kN)	32.7 lbs./ft. (48.6 Kg/M)	3-3/4” (95.3 mm)	R61-28

* The #24 and #28 diameter bars are not covered under ASTM A615.

All Couplings and Hex Nuts exceed 100% of the bar’s published ultimate strength and meet ACI 318 Section 25.5.7.1 for mechanical rebar connections.

Hex Nut

Round Collar Nut

R62 Stop-Type Coupling

ASTM A108

Bar Desig. & Nominal Dia.	Outside Diameter	Overall Length	Part Number
#6 - 3/4” (19 mm)	1-1/4” (31.8 mm)	2-3/4” (69.9 mm)	R62-06
#7 - 7/8” (22 mm)	1-3/8” (34.9 mm)	3” (76.2 mm)	R62-07
#8 - 1” (25 mm)	1-5/8” (41.3 mm)	3-1/2” (88.9 mm)	R62-08
#9 - 1-1/8” (29 mm)	1-7/8” (47.7 mm)	4” (102 mm)	R62-09
#10 - 1-1/4” (32 mm)	2” (50.8 mm)	4-1/2” (114 mm)	R62-10
#11 - 1-3/8” (36 mm)	2-1/4” (57.2 mm)	5” (127 mm)	R62-11
#14 - 1-3/4” (43 mm)	2-7/8” (73.0 mm)	6” (152 mm)	R62-14
#18 - 2-1/4” (57 mm)	3-1/2” (88.9 mm)	7-1/8” (181 mm)	R62-18
#20 - 2-1/2” (64 mm)	4” (102 mm)	8” (203 mm)	R62-20
#24 - 3” (76 mm)	5” (127 mm)	9-3/4” (246 mm)	R62-24
#28 - 3-1/2” (89 mm)	5-1/2” (140 mm)	12” (305 mm)	R62-28

R63 Hex Nut

ASTM A108 or A576

Bar Desig. & Nominal Dia.	Across Flats	Across Corners	Thickness	Part Number
#6 - 3/4” (19 mm)	1-1/4” (31.8 mm)	1.44” (36.6 mm)	1-1/8” (28.6 mm)	R63-06
#7 - 7/8” (22 mm)	1-7/16” (36.5 mm)	1.66” (42.2 mm)	1-1/4” (31.8 mm)	R63-07
#8 - 1” (25 mm)	1-5/8” (41.3 mm)	1.88” (47.8 mm)	1-3/8” (35.0 mm)	R63-08
#9 - 1-1/8” (29 mm)	1-7/8” (47.6 mm)	2.16” (54.9 mm)	1-1/2” (38.1 mm)	R63-09
#10 - 1-1/4” (32 mm)	2” (50.8 mm)	2.31” (58.7 mm)	2” (50.8 mm)	R63-10
#11 - 1-3/8” (36 mm)	2-1/4” (57.2 mm)	2.60” (66.0 mm)	2-1/8” (54.0 mm)	R63-11
#14 - 1-3/4” (43 mm)	2-3/4” (69.9 mm)	3.18” (80.8 mm)	2-1/2” (63.5 mm)	R63-14
#18 - 2-1/4” (57 mm)	3-1/2” (88.9 mm)	4.04” (103 mm)	3-3/4” (95.3 mm)	R63-18
#20 - 2-1/2” (64 mm)	4” (102 mm)	4.62” (117 mm)	3-3/4” (95.3 mm)	R63-20
* #24 - 3” (76 mm)	4-1/2” (114 mm)	O.D. 5” (127 mm)	4-3/8” (111 mm)	R64-24*
* #28 - 3-1/2” (89 mm)	5-1/2” (140 mm)	O.D. 6” (152 mm)	5-1/2” (140 mm)	R64-28*

* Round Collar Nut

Revised Soil Parameters

Design: CFT (8/14/2017)

Table DP - 1: Design Profile for Pier 33 and 34D.

Revised for Micropiles 12/13/18

Review:

Layer Description	Depth From - To	Elevation From - To	Soil Type	Typical N60 Range	Average N60	Effective Unit Weight[1]	Total Unit Weight[1], γ
	(Feet)	(Feet)	--	(BPF)	(BPF)	(pcf)	(pcf)
Fill	0 to 9	33 to 24	FILL	6 to 16	10	53	115
Q1: Fine-grained	9 to 38	24 to -5	CL - CH	8 to 12	10	43	105
Residual/HWR	38 to 48	-5 to -15	ML to SM	50	50	63	125
Rock	48 to 58	-15 to -25	GNEISS	--	--	73	135

Layer Description	Elevation From - To	Soil Friction Angle[1]	Soil Cohesion[1], c	Subgrade Modulus[2], k	Strain at 50%,100%[2], ε_{50}	Shear Modulus[2], G	Poisson's Ratio[2], ν
	(Feet)	(Degrees)	(psf)	(pci)	--	(ksi)	--
Fill	33 to 24	32	--	60	--	0.60	0.3
Q1: Fine-grained	24 to -5	--	1250	200	0.005, 0.015	1.50	0.4
Residual/HWR	-5 to -15	36	--	150	--	3.50	0.25
Rock	-15 to -25	45	--	--	--	9	0.2

			MICROPILES		
Layer Description	Elevation From - To	Uniaxial Comp. Strength, qu	Ult. Side Friction[2] MP, q_{u_side}	Ult. Bearing[2] H12X53 q_p	Unit End Bearing
	(Feet)	(ksf)	(psf)	(kips)	(ksi)
Fill	33 to 24	--	10	0	0
Q1: Fine-grained	24 to -5	--	100	50	0
Residual/HWR	-5 to -15	--	4000	0	0
Rock	-15 to -25	1000	10000	0	0

References

1) Bowles, Joseph E. (1979). *Physical and Geotechnical Properties of Soils.* McGraw Hill, Inc. New York, NY.

2) FB-MultiPier User Manual. Appendix: *FB-MultiPier Soil Parameter Table.* Software version 4.19.2.

Revised Soil Parameters

Design: CFT (8/14/2017)

Table DP - 2: Design Profile for Piers 35D through 38D.

Revised for Micropiles 12/13/18

Review:

Layer Description	Depth From - To	Elevation From - To	Soil Type	Typical N60 Range	Average N60	Effective Unit Weight[1]	Total Unit Weight[1], γ
	(Feet)	(Feet)	--	(BPF)	(BPF)	(pcf)	(pcf)
Fill	0 to 8	37 to 29	FILL	13 to 20	20	53	115
Q1: Fine-grained	8 to 27	29 to 10	CL - CH	2 to 12	7	43	105
Q2: Coarse-grained	27 to 34	10 to 3	SP to CL	10 to 25	15	53	115
Residual/HWR	34 to 39	3 to -2	ML to CL	15 to 50	40	63	125
Rock	39 to 62	-2 to -25	GNEISS	--	--	83	145

Layer Description	Elevation From - To	Soil Friction Angle[1]	Soil Cohesion[1], c	Subgrade Modulus[2], k	Strain at 50%[2], ε_{50}	Shear Modulus[2], G	Poisson's Ratio[2], v
	(Feet)	(Degrees)	(psf)	(pci)	--	(ksi)	--
Fill	37 to 29	32	--	60	--	0.60	0.25
Q1: Fine-grained	29 to 10	--	1250	200	0.01	1.50	0.4
Q2: Coarse-grained	10 to 3	32	--	60	--	0.75	0.25
Residual/HWR	3 to -2	40	--	150	--	3.50	0.25
Rock	-2 to -25	45	--	--	--	90	0.2

			STEEL PILES		
Layer Description	Elevation From - To	Uniaxial Comp. Strength, qu	Ult. Side Friction[2] MP, q_{u_side}	Ult. Bearing[2] H12X53 q_p	Unit End Bearing
	(Feet)	(ksf)	(psf)	(kips)	(ksi)
Fill	37 to 29	--	10		--
Q1: Fine-grained	29 to 10	--	100		--
Q2: Coarse-grained	10 to 3	--	100		--
Residual/HWR	3 to -2	--	4000	0	0
Rock	-2 to -25	1000	10000	0	0

References

1) Bowles, Joseph E. (1979). *Physical and Geotechnical Properties of Soils.* McGraw Hill, Inc. New York, NY.

2) FB-MultiPier User Manual. Appendix: *FB-MultiPier Soil Parameter Table.* Software version 4.19.2.

*Pier 35D weathered rock at EL -5, deeper than listed here

Use 25,000 psf for q_u in rock - this is 1% of the lab tested q_u and reflects the broken and variable nature of the rock on site.

Revised Soil Parameters

Design: CFT (8/14/2017)

Table DP - 3: Design Profile for Pier 39D and Abutment 40D.

Revised for Micropiles 12/13/18

Review:

Layer Description	Depth From - To	Elevation From - To	Soil Type	Typical N60 Range	Average N60	Effective Unit Weight[1]	Total Unit Weight[1], γ
	(Feet)	(Feet)	--	(BPF)	(BPF)	(pcf)	(pcf)
Fill	0 to 10	40 to 30	FILL	25	25	53	115
Q1: Fine-grained	10 to 25	30 to 15	CL - CH	10 to 30	20	53	115
Residual/HWR	25 to 35	15 to 5	ML to CL	25 to 40	30	63	125
Rock	35 to 45	5 to -5	GNEISS	--	--	83	145

Layer Description	Elevation From - To	Soil Friction Angle[1]	Soil Cohesion[1], c	Subgrade Modulus[2], k	Strain at 50%[2], ε_{50}	Shear Modulus[2], G	Poisson's Ratio[2], v
	(Feet)	(Degrees)	(psf)	(pci)	--	(ksi)	--
Fill	40 to 30	32	--	75	--	1.40	0.25
Q1: Fine-grained	30 to 15	--	1250	200	0.01	1.50	0.4
Residual/HWR	15 to 5	36	--	150	--	2.45	0.25
Rock	5 to -5	45	--	--	--	90	0.2

			STEEL PILES		
Layer Description	Elevation From - To	Uniaxial Comp. Strength, qu	Ult. Side Friction[2] MP, q_{u_side}	Ult. Bearing[2] H12X53 q_p	Unit End Bearing
	(Feet)	(ksf)	(psf)	(kips)	(ksi)
Fill	40 to 30	--	10		--
Q1: Fine-grained	30 to 15	--	100		--
Residual/HWR	15 to 5	--	4000	0	0
Rock	5 to -5	1000	10000	0	0

References

1) Bowles, Joseph E. (1979). *Physical and Geotechnical Properties of Soils.* McGraw Hill, Inc. New York, NY.

2) FB-MultiPier User Manual. Appendix: *FB-MultiPier Soil Parameter Table.* Software version 4.19.2.