FINAL GEOTECHNICAL DESIGN REPORT
WADSWORTH STREET BRIDGE
MAINEDOT WIN 16755.00
THOMASTON, MAINE

PREPARED FOR:
Calderwood Engineering, LLC
Richmond, Maine

PREPARED BY:
GZA GeoEnvironmental, Inc.
Portland, Maine

September 2014
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MaineDOT Soils Report No. 2014-19C

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GeoEnvironmental, Inc.

Engineers and Scientists

September 23, 2014
File No. 09.0025781.00

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Re: Final Geotechnical Design Report
Wadsworth Street Bridge
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Dear Eric:

GZA GeoEnvironmental, Inc. (GZA) is pleased to provide you with this Final Geotechnical Design Report for the Wadsworth Street Bridge in Thomaston, Maine. Our services were provided in accordance with our executed contract dated March 28, 2013, which incorporates GZA proposal dated January 4, 2013, and the Limitations contained in **Appendix A** of this report.

It has been a pleasure serving the Calderwood Engineering project team on this project. If you have any questions regarding the report, or if we can provide further assistance, please do not hesitate to contact the undersigned.

Very truly yours,

GZA GEOENVIRONMENTAL, INC.

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1.0 INTRODUCTION

This report presents the results of GZA GeoEnvironmental, Inc.'s (GZA's) subsurface explorations and geotechnical evaluations for replacement of the Wadsworth Street Bridge, which carries Wadsworth Street over the St. George River in Thomaston, Maine. Our services were provided in accordance with GZA's executed contract with Calderwood Engineering, LLC (Calderwood Engineering) dated March 28, 2013, which incorporates GZA's proposal dated January 4, 2013, and the Limitations contained in **Appendix A** of this report.

1.1 BACKGROUND

Bridge No. 2904 carries Wadsworth Street over the St. George River in Thomaston, Maine, as shown on the **Locus Plan, Figure 1**. GZA's understanding of the project is based on the 90 percent progress drawings prepared by Calderwood Engineering and Maine Department of Transportation (MaineDOT), provided to GZA on August 26, 2014, and recent conversations with Calderwood Engineering.

The project consists of replacing an existing, 235-foot-long, three-span bridge, the location of which is shown on the **Boring Location Plan, Figure 2**. The existing bridge is supported by abutments and two river piers. The abutments and piers appear to be supported on pile foundations.

The replacement bridge is planned to consist of a 280-foot-long, four-span bridge, supported on integral abutments and pile bent piers. The superstructure will be comprised of hybrid composite beams (fiber reinforced polymer and structural concrete) supporting a cast-in-place concrete deck. The alignment of the replacement bridge is approximately 50 feet upstream from the current location, as shown on **Figure 2**. The approaches will be widened toward the west to accommodate the new bridge, resulting in a maximum fill height of approximately 16 to 20 feet.

1.2 OBJECTIVES AND SCOPE OF SERVICES

The objectives of our work were to evaluate subsurface conditions and to provide final geotechnical engineering recommendations for the proposed bridge replacement. To meet these objectives, GZA completed the following Scope of Services:

- Conducted site visits to observe surficial conditions at the ground surface, assessed traffic, drilling access, and reviewed mapped surficial and bedrock geology of the site;
- Reviewed the available data from previous subsurface exploration programs and reviewed recovered rock core samples from the MaineDOT Bangor storage facility;
- Coordinated and observed a subsurface exploration program consisting of three test borings for the replacement bridge;
- Conducted a laboratory testing program to evaluate engineering properties of the site bedrock;
- Conducted geotechnical engineering analyses to evaluate foundation alternatives for the replacement bridge, embankment design considerations, and seismic design considerations;

- Developed geotechnical engineering recommendations including foundation design recommendations for the preferred foundation type; and
- Prepared this Final report summarizing our findings and design recommendations.

2.0 SUBSURFACE EXPLORATIONS

Previous exploration programs were completed by MaineDOT for the proposed bridge replacement in 2010 and 2011. GZA completed a final design subsurface exploration program in 2013. Details of these exploration programs are discussed below.

2.1 PREVIOUS EXPLORATIONS

Three test borings were drilled in 2010 and five in 2011 for the Wadsworth Street Bridge replacement. Details of the exploration programs are summarized below.

Three test borings, BB-TSGR-101 through BB-TSGR-103, were drilled between November 1 and November 3, 2010 by MaineDOT. Two of the test borings (BB-TSGR-101 and BB-TSGR-103) were drilled through the roadway on the landside of the existing south and north abutments, respectively, and one boring (BB-TSGR-102) was drilled in the river, through the existing bridge deck. The locations of the borings are shown on **Figure 2**. The test borings were drilled through the overburden soils and terminated approximately 10 feet into bedrock. Depths of borings ranged from approximately 35 to 65 feet below ground surface (bgs). The borings were drilled using 3- and 4-inch casing and drive and wash drilling techniques. Standard Penetration Testing (SPT) and split spoon sampling was generally conducted at 5-foot increments through the overburden in each boring. Draft logs of borings BB-TSGR-101 through BB-TSGR-103 were prepared by MaineDOT and provided to GZA.

Five test borings, BB-TSGR-201 through BB-TSGR-205, were drilled between September 27 and October 3, 2011 by Northern Test Boring, Inc. of Gorham, Maine. The test borings were completed using a barge-mounted drill rig in the river along the proposed bridge alignment, including one at each proposed pier and abutment location, as shown on **Figure 2**. The test borings were drilled through the overburden soils and terminated approximately 5 to 9 feet into bedrock. Depths of borings ranged from approximately 10 to 40 feet bgs. The borings were drilled using 3- and 4-inch casing and drive and wash drilling techniques. SPT and split spoon sampling was performed periodically and inconsistently in the borings. Draft logs of borings BB-TSGR-201 through BB-TSGR-205 were prepared by MaineDOT and provided to GZA.

2.2 REVIEW OF MAINEDOT LOGS

GZA requested access to the rock core samples from previous subsurface exploration programs to assess the bedrock classification and quality. After receiving approval from the MaineDOT Geotechnical Group, a GZA engineer visited MaineDOT's laboratory in Bangor, reviewed the available rock core specimens, and prepared an independent description for core samples from both the BB-TSGR-100 and -200 series borings, with the exception of BB-TSGR-102, which was not located at the time of our visit.

GZA also reviewed the draft test boring logs to assess consistency between soil visual descriptions and laboratory test reports. Where discrepancies were found, the visual descriptions were updated to correspond to the laboratory test results.

The final boring logs for the BB-TSGR-100 and -200 series borings, including GZA's revisions, are included in **Appendix B**.

2.3 DESIGN PHASE EXPLORATIONS

GZA completed a design phase subsurface investigation program consisting of three test borings along the proposed bridge alignment, including one at each pier location. The test borings (BB-TSGR-301 through BB-TSGR-303) were completed using a barge-mounted drill rig. The as-drilled locations of the test borings were surveyed by MaineDOT personnel. Mudline elevations were determined at each test boring at the start of drilling by MaineDOT personnel and were provided to GZA. Elevations referenced in this report are in feet and refer to North American Vertical Datum of 1988 (NAVD 1988). The boring locations are shown on **Figure 2.**

The test borings were drilled to depths ranging from approximately 14 to 32 feet bgs and were terminated approximately 5 to 6 feet into bedrock. Maine Test Boring, Inc. of Hermon, Maine provided drilling services and coordinated utility clearance. The drilling was completed between April 16 and April 18, 2013. GZA personnel monitored the drilling work and prepared logs of each boring that are included in **Appendix C**.

The borings were drilled using 3-inch and 4-inch casing and drive-and-wash drilling techniques. SPT and split-spoon sampling were generally performed at 5-foot typical intervals in the borings using a 24-inch-long, 1-3/8-inch inside diameter sampler. SPTs were conducted using a safety hammer and rope-and-cathead system, according to MaineDOT requirements. Bedrock cores were obtained using NQ2 wire-line coring equipment.

3.0 LABORATORY TESTING

MaineDOT conducted a soil laboratory testing program on split spoon samples collected from the BB-TSGR-100 and -200 series borings. The results of the soil laboratory tests are included in **Appendix D**. The soil testing program consisted of the following:

- Gradation analysis/AASHTO Classification/Frost Classification assessments and natural water contents of 11 soil samples;
- Hydrometer analysis/AASHTO Classification/Frost Classification assessments and natural water contents of 16 soil samples; and
- Atterberg limits testing of one soil sample.

GZA retained Thielsch Engineering's Geotechnical Laboratory in Cranston, Rhode Island to assess the engineering properties of the bedrock. The program included unconfined compression and modulus determinations on four representative bedrock core samples. Two rock core samples from the BB-TSGR-300 series and two from the BB-TSGR-200 were selected for testing. Results of the testing and photographs of each specimen after testing are included in **Appendix D**.

4.0 SUBSURFACE CONDITIONS

4.1 SURFICIAL AND BEDROCK GEOLOGY

According to the "Simplified Surficial Geologic Map of Maine" (Marvinney, 2003), surficial geologic units mapped at the Wadsworth Street bridge include the Presumpscot Formation, consisting of glaciomarine silt, clay and sand deposits, and Till.

According to the Maine Geological Survey's "Thomaston Prison Quarry, Thomaston, Maine" (Marvinney, 2002), bedrock at the Wadsworth Street bridge is of Ordovician and Precambrian ages. The Ordovician formation is described as schist and quartzite. The Precambrian is described as thinly bedded marble, quartzite, and schist.

4.2 SUBSURFACE SOIL PROFILE

Four soil units were encountered above bedrock in the explorations: Fill, Silt/Clay, River Bottom Deposit, and Glacial Till. The encountered thicknesses, generalized descriptions and engineering properties of the units encountered, in descending order from ground surface, are summarized in the following table. The subsurface conditions are also shown in relation to the bridge abutments and piers on **Interpretive Subsurface Profile, Figure 3**. Detailed descriptions of the materials encountered at specific locations are provided in the boring logs included in **Appendices B** and **C**.

Soil Unit	Approx. Encountered Thickness (ft)	Generalized Description and Properties
Fill	5 to 10	Brown, loose to dense, fine to coarse SAND, some Gravel, some Silt. *Encountered in borings BB-TSGR-101 and BB-TSGR-103.*
Silt/Clay	5 to 8	Gray to brown, very soft to stiff, Sandy SILT, trace Clay, occasional Gravel. *Encountered in borings BB-TSGR-101, BB-TSGR-103, potentially BB-TSGR-205, and BB-TSGR-303.*
River Bottom Deposit	4 to 25	Brown to dark gray, loose to medium dense, fine to coarse SAND, with varying amounts of Gravel and Silt. Occasional wood and shell fragments. Occasional cobbles/boulders noted in stratum. *Encountered in all river borings with adequate samples to assess stratum, including BB-TSGR-102, BB-TSGR-202, BB-TSGR-203, BB-TSGR-204, BB-TSGR-205, BB-TSGR 302 and BB-TSGR-303, and BB-TSGR-103.*
Glacial Till	2 to 12	Gray to brown, medium dense to very dense, fine to coarse SAND, with varying amounts of Silt and Gravel. *Encountered in all borings.* Cobbles/boulders were noted throughout this stratum, most prominently in borings BB-TSGR-102, BB-TSGR-204, BB-TSGR-205, and BB-TSGR-303
Top of Rock	Encountered Top of Rock: El. -13.9 to -41.5 *Encountered in all borings.*	

4.2.1 Bedrock

Bedrock descriptions are based on GZA's review of core collected from previous and design-phase test borings. The rock encountered in the borings consisted of Metapelite and Metasandstone. The Metapelite was described as moderately hard to very hard, fresh to

moderately weathered, fine grained and gray to green. Primary joints were extremely close to moderately spaced, moderately dipping to high angle, undulating, rough, fresh to decomposed, and partially open to tight. Secondary joints were close to moderately spaced, low angle to horizontal, undulating, rough, fresh, and tight to open. The RQD ranged from 0 to 76 percent, with an average of 58 percent.

The Metasandstone was described as very hard, fresh, fine grained, and white and gray. Primary joints were close to moderately spaced, low angle to horizontal, undulating, rough, fresh to discolored, and tight to partially open. Secondary joints were moderately spaced, moderately dipping to high angle, planar, rough, discolored, and tight. The RQD ranged from 67 to 83 percent, with an average of 68 percent.

Four laboratory unconfined compressive strength / secant modulus / Poisson's ratio tests were conducted on bedrock core samples of Metapelite and Metasandstone. The test results are included in **Appendix D**. The testing yielded unconfined compressive strengths ranging from 8.4 to 21.5 kips per square inch (ksi), Young's modulus values ranging from 4,990 to 7,440 ksi, and Poisson's ratios ranging from 0.07 to 0.21.

4.2.2 Groundwater

Groundwater levels recorded at each of the abutments varied tidally during the test borings due to the nature of the subsurface materials and the presence of free-draining, stone masonry retaining structures at the site. Two test borings (BB-TSGR-101 and -103) were drilled on land, through the existing approach embankments. Measured water levels at the time of drilling ranged from 18 to 19 feet bgs (El. -2.6 to El. -4.4). All other test borings were drilled within the limits of the river.

Groundwater levels are anticipated to fluctuate due to tide, season, precipitation, infiltration and construction activity in the area. Therefore, groundwater levels during and after construction are likely to vary from those encountered at the time of the test borings.

5.0 ENGINEERING EVALUATIONS

5.1 GENERAL

GZA has conducted geotechnical engineering evaluations in general accordance with the 2012 AASHTO LRFD Bridge Design Specifications, 6th Edition (AASHTO), with 2013 Interims, and the MaineDOT Bridge Design Guide (BDG), as referenced herein. Supporting calculations developed by GZA for the project are attached in **Appendix E** of this report.

5.2 APPROACH EMBANKMENTS

The abutments will require construction of new fill embankments immediately west (left) of the existing embankments, with fill heights up to 19 feet above existing grades on the landside of the new abutments. The maximum side slope angles are anticipated to be 2 horizontal to 1 vertical (2H:1V), or flatter, with loam and seed surface treatments above the tidal range. Portions of the side slopes within the river tidal range are anticipated to be constructed with riprap scour protection.

Subsurface conditions beneath the proposed approach embankments include existing fill, soft to stiff silt and clay (Silt / Clay), loose to dense silty sand (River Bottom Deposits) and/or glacial till over bedrock.

5.2.1 Settlement

At the south (Abutment 1) approach, the new embankment is anticipated to be underlain by 5 feet or less of stiff Silt / Clay over medium dense to dense Glacial Till and bedrock. Based on SPT N-values, the Silt / Clay at this location is judged to be highly overconsolidated, and moderately compressible under the proposed embankment. Total embankment settlement is estimated to be on the order of 1-½ inches where the thickest Silt/Clay is present, most of which will occur during construction. Consolidation of the Silt/Clay will occur relatively quickly as the embankment is constructed. Consequently, post-construction settlement is estimated to be on the order of ½ inch or less.

The north (Abutment 2) approach is underlain by approximately 10 feet of loose to medium dense River Bottom Sand, 7 feet of medium stiff to stiff Silt/Clay, an additional 8 feet of loose to medium dense River Bottom Sand, dense to very dense Glacial Till, and bedrock. Potential settlement resulting from embankment construction would occur primarily in the Silt/Clay stratum, and to a lesser extent in the River Bottom stratum.

Three split spoon samples were conducted in the Silt/Clay stratum near Abutment 2; one in boring BB-TSGR-103 from 30 to 32 feet bgs (El. -15.4 to El. -17.4) and two in BB-TSGR-205 at depths of 8 to 10 and 15 to 17 feet bgs (El. -12.2 to -14.2 and El. -19.2 to -21.2). Recorded SPT N-values ranged from weight of hammer to 4 blows per foot, and the measured fines content (percent passing the No. 200 sieve) ranged from 58 to 72 percent, and one Atterberg limits analysis indicated it was non-plastic, resulting in a general classification of sandy SILT. Two in situ vane shear tests were attempted in BB-TSGR-103, one immediately below the split spoon sample (at 32 feet bgs) and another at 37 feet bgs, but neither could be advanced, due to the presence of sand and or gravel.

Embankment settlement is a function of the grade raise and the overall thickness and compressibility of the deposits. Compressibility of the Silt/Clay was approximated for use in our evaluations based on the available data including SPT N-values, water content and sand content. Based on these data we judged the material to be lightly to moderately overconsolidated and of low to moderate compressibility. Given the high sand content and interbedded sand layers the rate of settlement was judged to be relatively high. Total embankment settlement is estimated to be on the order of 2 to 5 inches. Consolidation of the Silt/Clay will occur relatively quickly as the embankment is constructed, and elastic settlement of the sand will occur immediately as fill is placed. Therefore, we estimate that at least half of the settlement will occur during construction, resulting in approximately 1 to 3 inches of post-construction settlement.

Since more than 0.4 inch of settlement could occur after pile installation, downdrag loading should be considered in the design of the piles at both abutments, as discussed in **Section 5.4.2**.

5.2.2 Global Stability

The south (Abutment 1) approach embankment will be constructed per MaineDOT standard specifications and details using engineered fill and controlled methods. In our experience, conventional earthfill embankments constructed over relatively stiff/dense overburden soils meet the minimum required safety factors for global stability.

A global stability analysis was performed for the highest portion of the proposed embankment along the northern approach (Sta. 27+71.5) using the analytical software Slope-W and the Modified Bishop method. Subsurface layering was developed based on the results of borings BB-TSGR-103 and BB-TSGR-205, and representative soil properties were developed for the existing sand and clay deposit and glacial till using the data from these borings. The average engineering properties for the embankment fill assume that the embankment material and construction methodology will be in accordance with MaineDOT Standard Specifications. Considering the significant sand content in the Silt/Clay layer, the strength was modeled in the drained condition. An effective friction angle of 30 degrees was assigned to the Silt/Clay deposit. The output from Slope-W, including soil properties, layering, and critical slip surface, is presented in **Appendix E.** The results indicate the calculated factor of safety against circular slope failure of the north approach embankment is approximately 1.3, which is the AASHTO minimum safety factor for a slope that does not support a structure. Therefore, it is GZA's opinion that global stability is acceptable for the proposed grading along the northern approach.

Boring BB-TSGR-205 encountered a layer of very soft organic silt immediately below the mudline. This material would be unstable if left in place beneath the new embankment, and should be removed prior to embankment construction, in accordance with the recommendations presented in **Sections 6.2** and **7.3**. The global stability evaluation assumed that the very soft organic silt would be removed prior to embankment construction.

5.3 EVALUATION OF FOUNDATIONS

5.3.1 Foundation Type Assessment

The project team considered conventional driven steel piles (H-piles and pipe piles) and fiber reinforced polymer (FRP) piles. MaineDOT undertook a test pile program including pile installation, load testing, and extraction of FRP piles at a different site during design development for this project. Due to concerns about the drivability of the FRP piles, MaineDOT opted not to pursue their use for this project. Therefore, driven steel piles were selected for the project. Calderwood Engineering selected HP-section piles to support the integral abutments and pipe piles to support the piers. Where embedment is limited at Pier 1, rock anchors will be installed through selected pipe piles to enhance shear resistance at the pile tip.

5.3.2 Pile Design Considerations and Load and Resistance Factors

Evaluations were conducted for axial compressive geotechnical resistance of the piles. The axial geotechnical static resistance of piles was calculated using the Nordlund Thurman method (using the analytical software SPILE®) and/or the Meyerhof SPT method in accordance with AASHTO Article 10.7. Supporting calculations are provided in **Appendix E**. The results indicate the abutment piles will gain support through a combination of side friction in the overburden soil (primarily in glacial till) and end bearing in glacial till or on bedrock. The pier piles will gain support primarily through end bearing in glacial till or on bedrock, with typically 15 percent or less of the resistance derived from skin friction in overburden soil.

Based on our experience with similar soils, we anticipate that the piles will need to be driven into or near bedrock to achieve the required resistance. The side friction distribution was also used as an input in wave equation analyses conducted to assess the pile drivability for the abutments and the piers. It is our understanding that the pier and abutment piles will not be subject to axial

tensile (uplift) loading. Since the piles will gain support in primarily dense granular soil and/or bedrock, there is no reduction for group interaction in axial compression. It is our understanding that there are no design uplift loads on the piles.

By utilizing end bearing steel H-piles and pipe piles, total and differential settlement will be limited to elastic compression of the piles and should be less than ½ inch.

Piles should be designed at the strength limit state considering the structural resistance of the piles and a resistance factor (ϕ_c) of 0.50 for H-piles and 0.6 for pipe piles, per AASHTO Section 10.7.3.2.3 and Section 6.5.4.2 for hard driving condition; the geotechnical resistance of the piles; and the potential loss of lateral support due to scour at the design flood event, if any. In GZA's experience for end bearing piles on bedrock, the structural resistance or drivability resistance will control the geotechnical static resistance of the pile.

Pile installation will be controlled based on dynamic pile testing with signal matching analysis. The piles should be driven to a nominal resistance calculated by dividing the maximum factored pile load by a resistance factor of 0.65, per AASHTO Table 10.5.5.2.3-1.

AASHTO LRFD load factors should be applied to horizontal earth pressure (EH), vertical earth pressure (EV) and earth surcharge (ES) loads using the load factors for permanent loads (γ_p) provided in AASHTO Table 3.4.1-2 for strength and extreme limit state design. A load factor (γ_p) of 1.0 should be applied to downdrag loads in cohesive and cohesionless downdrag zones. A load factor of 1.5 may be applied to the passive pressure used to design the integral backwall (end diaphragm) to account for deformation of the backwall into the soil as a result of thermal expansion of the integral bridge deck.

5.3.3 Corrosion

The pier piles will extend through seawater (including the splash zone, inter-tidal zone, low-water zone, and immersion zone), and through the riverbed soils (buried zone). We understand that Calderwood Engineering and MaineDOT are developing coating details to inhibit pile section loss associated with corrosion. For unprotected piles, the corrosion rate has been found to vary between different zones for piles driven through seawater. The corrosion rates for steel piles installed through brackish water, as presented in the 2006 FWHA Publication FHWA NHI-05-042, Design and Construction of Driven Pile Foundations, are listed in the table below.

CORROSION RATES FOR UNPROTECTED STEEL PILES IN SEAWATER		
Portion of Pile	**Typical corrosion loss (inches/year)**	**95% maximum probable corrosion loss (inches/year)**
Splash Zone (near high water level)	0.0035	0.0071
Inter-tidal Zone (between high and low water)	0.0016	0.0043
Low Water Zone (near low water level)	0.0035	0.0071
Immersion Zone (between low water and river bed)	0.002	0.0055
Buried Zone	0.0008	0.002

Based on our discussion with Calderwood Engineering, we understand that a corrosion loss of 1/8 inch around the outside of the pipe piles has been included in the structural design, extending from the bottom of the pile cap to 5 feet below the mudline. GZA's lateral pile evaluations also

considered 1/8-inch corrosion section loss. Once the coating deteriorates, the fastest anticipated corrosion rates would be in the splash zone and low water zone. Based on the typical corrosion loss rates presented above, the time required to incur 1/8-inch corrosion loss is approximately 35 years, plus the initial time required to fully breach the protective coating.

The abutment piles will be installed on land; therefore, corrosion was not considered in the design.

5.4 EVALUATION OF ABUTMENT PILES

5.4.1 Pile Type

The abutments are planned to be supported on ASTM A572, Grade 50 (fy=50 ksi) steel H-piles. Each abutment will include a single row of six, HP14x117 piles.

5.4.2 Downdrag

Given the potential for greater than 0.4 inch of settlement to occur relative to the abutment piles, the piles should be designed to resist downdrag loading.

Side friction contributing to downdrag load was estimated using the β-method in accordance with NAVFAC DM 7.2-211, and as recommended by Sandford et al, "Bitumen Coatings Reduce Downdrag on Piles for Route 1 Interchange Bridges." Beta values were assumed to be 0.35, 0.3, and 0.23 for the new fill, the river bottom sand, and the marine clay, respectively. Based on past practice, a load factor of 1.0 was applied to the calculated downdrag resistance, which was added to the maximum factored load provided by Calderwood Engineering.

The evaluations are presented in **Appendix E**, and the results are summarized below:

ABUTMENT 1 DOWNDRAG LOADING	
Downdrag Component (LRFD 10.7.3.7)	**Load or Resistance (kips)**
Nominal Downdrag Load (DD)	15
Factored Downdrag Load (γ_p DD)	15
Maximum Factored Pile Load, Axial Compression	162
Total Factored Load (Rn / ϕ)	177
Nominal Driving Resistance Required (Rndr)	272

ABUTMENT 2 DOWNDRAG LOADING	
Downdrag Component (LRFD 10.7.3.7)	**Load or Resistance (kips)**
Nominal Downdrag Load (DD)	66
Factored Downdrag Load (γ_p DD)	66
Maximum Factored Pile Load, Axial Compression	162
Total Factored Load (Rn / ϕ)	228
Nominal Driving Resistance Required (Rndr)	351

5.4.3 Pile Loading Data

The maximum factored axial compressive pile load for the strength condition provided by Calderwood Engineering is 162 kips per pile. Considering downdrag loads calculated and presented above, the total factored load is 177 kips at Abutment 1 and 228 kips at Abutment 2. Considering the resistance factor of 0.65 for drivability, the required nominal pile resistance is 272 kips at Abutment 1 and 351 kips at Abutment 2.

5.4.4 Design Subsurface Profiles

GZA developed design subsurface profiles for use in evaluating abutment foundations. The top of each profile is defined at the bottom of the integral abutment; material adjacent to The profiles are summarized in the following tables.

DESIGN SUBSURFACE PROFILE – ABUTMENT 1 (Borings BB-TSGR-101 and -201)			
Strata Designation	**Thickness (feet)**	**Total Unit Weight (pcf)**	**Representative Φ' (°) / Su (psf) for layer**
New Fill	6	125	34º
Silt/Clay	4	118	1,000 psf
Glacial Till	11	130	38º
Depth to Bedrock	21	--	--

DESIGN SUBSURFACE PROFILE – ABUTMENT 2 (Borings BB-TSGR-103 and -205)			
Strata Designation	**Thickness (feet)**	**Total Unit Weight (pcf)**	**Representative Φ' (°) / Su (psf) for layer**
New Fill	4	125	34º
River Bottom Deposit	11	120	30º
Silt/Clay	7	118	1,000 psf
River Bottom Deposit	7	120	30º
Glacial Till	12	130	38º
Bedrock	41	--	--

5.4.5 Wave Equation Analyses

The wave equation analyses for the abutments used the design soil profiles presented in **Section 5.4.4**, with an embedded pile length of approximately 21 feet at Abutment 1 and approximately 41 feet at Abutment 2. The pile type and section were established based on lateral loading and combined stress considerations. Therefore, the wave equation analyses set out to find a driving system that could install the proposed HP14x117 piles to resist the axial design loads presented in **Section 5.4.3** (i.e., the required nominal pile resistance). It was assumed that the shorter, Abutment 1 piles will encounter harder driving conditions, with an assumed toe quake of 0.04 inches, and the longer Abutment 2 piles will encounter moderate driving conditions, with an assumed toe quake of 0.10 inches. The evaluation for both used an APE D19-42 open-end diesel pile driving hammer with a manufacturer's rated energy of approximately 47,100 foot-pounds. The hammer has adjustable fuel settings to reduce the maximum energy, allowing installation of

piles with different sections and resistance requirements with a single hammer. A fuel setting corresponding to a chamber pressure of 1,247 psi, which is the lowest available, was used for the analysis. The results of the wave equation analyses are summarized below.

WAVE EQUATION ANALYSIS RESULTS – ABUTMENTS				
Condition	**Driving System**	**Required Nominal Geotechnical Resistance (kips)**	**Maximum Driving Stress (ksi)**	**Final Penetration Resistance (blows per inch)**
Abutment 1 (L=21 feet)	APE D19-42 (47,100 ft-lb)	272	18	4
Abutment 2 (L=41 feet)	APE D19-42 (47,100 ft-lb)	351	19	5

Since the driving stresses do not exceed the limiting driving stress of 45 ksi for ASTM A572 steel (50 ksi yield stress), and the calculated penetration resistance is within the MaineDOT range of 3 and 15 blows per inch, the analyzed hammer system is judged acceptable to install the piles to the nominal resistance noted. It is GZA's experience that the optimum range of penetration resistance for diesel pile hammers is in the range of 6 to 10 blows per inch. Based on the estimated final penetration resistance of 4 to 5 blows per inch to achieve the required factored pile resistance for the APE D19-42 and the fact that a reduced chamber pressure was used, a hammer with a lower rated energy could also be used to install the piles. However, this hammer was selected in an effort to choose one hammer that could be used to install the abutment and pier piles to their required nominal resistances (see **Section 5.5.3** for pier piles).

It is noted that the HP14x117 piles can be driven to a final penetration resistance greater than 15 blows per inch with the APE D19-42 hammer without approaching the limiting driving stress of 45 ksi. Our calculations indicate nominal geotechnical drivability resistances on the order 650 kips at Abutment 1 and 550 kips at Abutment 2 could be achieved with the selected hammer with a final penetration resistance of about 15 bpi.

Hard driving is anticipated in rock fill layers, glacial till and/or on or near the bedrock surface. Recommendations are provided for tip protection in **Section 6.4** to reduce the potential for driving damage.

5.4.6 Lateral Pile Resistance – Abutment Piles

We understand that lateral pile analyses have been completed by Calderwood Engineering using L-Pile® software that use non-linear P-Y curves to model soil reactions. We recommend that the pile analysis results be used to check that the combined axial and bending demand meet AASHTO LRFD structural requirements. It is anticipated that the controlling analysis will be based on the maximum thermal deflection of 0.57 inches, as provided by Calderwood Engineering. GZA conducted preliminary lateral pile evaluations using L-Pile 5.0 to assess maximum pile stress and fixity, initially considering HP14x89 and HP14x117. Abutment 1 piles achieved a pinned condition, with negligible lateral translation at the pile toe under the design loads. Abutment 2 piles achieved a fixed condition.

Geotechnical design parameters were developed for L-Pile® analyses corresponding to the design soil engineering properties tabulated in **Section 5.4.4**. The recommended design parameters are presented below.

The structural engineer should evaluate the minimum required pile embedment and combined stresses for the integral abutment bridge. The recommended representative geotechnical parameters for L-Pile® analyses are provided in the tables below. The top of the upper layer (Fill) corresponds to the bottom of the proposed pile cap, and the bottom lower layer (Glacial Till) corresponds to anticipated top of rock. Therefore, the combined layer thicknesses represent the anticipated embedded pile lengths (21 feet for Abutment 1 and 41 feet for Abutment 2).

SOIL PARAMETERS FOR L-PILE® INPUT ABUTMENT 1				
Soil Model	**Layer Thickness**	**k (pci) / E50**	**φ' (deg)/ Su (psf)**	**Unit Weight γ_e (pci)**
Reese Sand	2	150	34	0.072
Reese Sand	4	60	34	0.036
Stiff Clay	4	E_{50} = 0.01	1000 psf	0.032
Reese Sand	11	90	38	0.039

SOIL PARAMETERS FOR L-PILE® INPUT ABUTMENT 2				
Soil Model	**Layer Thickness (ft)**	**k (pci) / E50**	**φ' (deg)/ Su (psf)**	**γ_e (pci)**
Reese Sand	4	60	34	0.036
Reese Sand	11	20	30	0.033
Stiff Clay	7	E_{50} = 0.01	1000 psf	0.032
Reese Sand	7	20	30	0.033
Reese Sand	12	90	38	0.039

5.4.7 Lateral Earth Pressure

Thermal expansion of the bridge will cause the backwalls and wingwalls of the integral abutment to move toward the backfill, which will result in earth pressures ranging from at-rest to passive earth pressure. The material properties will be controlled by the backfill material, which is proposed to consist of BDG Type 4 soil. Soil properties for Type 4 soil are provided in **Section 6.3** of this report.

A report prepared for the Virginia Transportation Research Council (VTRC) entitled, "The Behavior of Integral Abutment Bridges," evaluated earth pressures on abutment backwalls and concluded that Rankine or Coloumb passive earth pressure could be conservatively used for design of most integral abutment bridges.

Lateral earth pressure evaluations for abutments are based on the BDG and the VTRC reference indicated above and summarized below:

- Passive earth pressure coefficients were developed using Rankine theory for Type 4 soil.
- AASHTO Commentary C3.10.9.1 specifies that bridges in Seismic Design Category 1 are not required to include acceleration-augmented (earthquake-induced) soil pressures for design.

Design lateral earth pressure recommendations are provided in **Section 6.3** of this report.

5.5 EVALUATION OF PIER PILES

5.5.1 Pile Type and Loading Data

The river piers are proposed to be supported on PP24x0.75 (battered) and PP24x0.625 (plumb), ASTM A252, Grade 3 Modified (fy=50 ksi) steel pipe piles. In order to eliminate potential mix-up during installation, the use of the single, heavier wall thickness piles should be considered for both plumb and batter piles. Each pier cap will be supported on a single row of five piles each, consisting of three plumb piles and two battered piles. Each outside pile will be battered at an inclination of 2.5:12. The maximum factored axial load for the strength condition provided by Calderwood Engineering is 442 kips per pile. Considering a resistance factor of 0.65, the required nominal pile resistance is approximately 680 kips.

5.5.2 Design Subsurface Profiles

GZA developed design subsurface profiles for use in evaluating pier foundations. The profiles are summarized in the following tables.

DESIGN SUBSURFACE PROFILE – PIER 1 (Boring BB-TSGR-301)			
Strata Designation	**Thickness (feet)**	**Total Unit Weight (pcf)**	**Representative Φ' (°) / Su (psf) for layer**
River Bottom	2	120	30º
Glacial Till	3	130	38º
Depth to Bedrock	5	--	--

DESIGN SUBSURFACE PROFILE – PIER 2 (Boring BB-TSGR-302)			
Strata Designation	**Thickness (feet)**	**Total Unit Weight (pcf)**	**Representative Φ' (°) / Su (psf) for layer**
River Bottom	14	120	30º
Glacial Till	5	130	38º
Depth to Bedrock	19	--	--

DESIGN SUBSURFACE PROFILE – PIER 3 (Boring BB-TSGR-303)			
Strata Designation	**Thickness (feet)**	**Total Unit Weight (pcf)**	**Representative Φ' (°) / Su (psf) for layer**
River Bottom Deposit	6	120	30º
Silt/Clay	9	115	300 psf
River Bottom Deposit	4	120	30º
Glacial Till	7	130	38º
Bedrock	26	--	--

5.5.3 Wave Equation Analyses

The wave equation analyses for the piers used the design soil profiles for Piers 1 and 3 presented in **Section 5.4.5**, with embedded pile lengths of approximately 5 feet at Pier 1 and 26 feet at Pier 3. The pile type and section were established based on lateral loading and combined stress considerations. Therefore, evaluation set out to find a driving system that could

install the proposed PP24x0.625 piles to resist the design loads presented in **Section 5.5.1** (i.e., the required pile resistance). Drivability was not evaluated for PP24x0.75 piles, as it is assumed that the larger pile section could be driven to an equal or greater drivability resistance as the smaller section.

Because Pier 2 pile lengths will be between the Pier 1 (short) and Pier 3 (long) pile lengths, we concluded that Pier 2 would not control drivability considerations. It was assumed that the shorter Pier 1 piles will encounter hard driving conditions, with an assumed toe quake of 0.04 inches, and the longer Pier 3 piles will encounter moderate driving conditions, with an assumed toe quake of 0.10 inches. The evaluation used an APE D19-42 open-end diesel pile driving hammer with a manufacturer's rated energy of approximately 47,100 foot-pounds and a fuel setting corresponding to the maximum chamber pressure of 1,710 psi. The results of the wave equation analyses are summarized below.

WAVE EQUATION ANALYSIS RESULTS				
Condition (D corresponds to pile embedment below mudline)	**Driving System**	**Required Nominal Drivability Resistance (kips)**	**Maximum Driving Stress (ksi)**	**Final Penetration Resistance (blows per inch)**
Pier 1 (D = 5 feet)	APE D19-42 (47,100 ft-lb)	680	33	7
Pier 3 (D = 26 feet)	APE D19-42 (47,100 ft-lb)	680	28	9

Since the driving stresses do not exceed the limiting driving stress of 45 ksi for ASTM A252 Grade 3 Modified (fy = 50 ksi) steel, and the calculated penetration resistance is within the MaineDOT range of between 3 and 15 blows per inch, the analyzed hammer system is judged acceptable to install the piles to the nominal resistance noted. It is GZA's experience that the optimum range of penetration resistance for diesel pile hammers is in the range of 6 to 10 blows per inch.

It should be noted that the PP24x0.625 piles could be driven to a final penetration resistance greater than 15 blows per inch with the APE D19-42 hammer without approaching the limiting driving stress of 45 ksi. Our calculations indicate nominal geotechnical drivability resistances on the order 1,000 kips at Pier 1 and 800 kips at Pier 3 could be achieved with the selected hammer with a final penetration resistance of about 15 bpi.

Hard driving is anticipated in rock fill layers, glacial till and/or on or near the bedrock surface. Recommendations are provided for tip protection in **Section 6.4** to reduce the potential for driving damage.

5.5.4 Lateral Pile Resistance – Pier Piles

Calderwood Engineering requested that GZA conduct lateral pile analyses to assess the effective depth to fixity for the pier piles for use in their structural analyses. Due to the overburden thickness of 5 feet at Pier 1, it was concluded that the pile tips would require anchors to prevent lateral translation; therefore, analysis of lateral soil resistance on those piles was not warranted. For Piers 2 and 3, GZA conducted preliminary lateral pile evaluations using GROUP® software (Version 8.0) to assess maximum pile stress and fixity, considering PP24x0.625 plumb and battered piles, in the currently proposed configuration. The design parameters used in our analyses are presented in the tables below.

SOIL PARAMETERS FOR GROUP® INPUT PIER 2				
Soil Model	**Layer Thickness**	**k (pci)**	**φ' (deg)**	**Unit Weight γ_e (pci)**
Distance from bottom of pile cap to mudline = 28 feet				
Reese Sand	14	20	30°	0.033
Reese Sand	5	90	38°	0.039

SOIL PARAMETERS FOR GROUP® INPUT PIER 3				
Soil Model	**Layer Thickness**	**k (pci) / E50**	**φ' (deg) / Su (psf)**	**Unit Weight γ_e (pci)**
Distance from bottom of pile cap to mudline = 23 feet				
Reese Sand	6	20	30°	0.033
Soft Clay	9	E_{50} = 0.02	300 psf	0.030
Reese Sand	4	20	30°	0.033
Reese Sand	7	90	38°	0.039

The evaluation was conducted for two load cases: Service I and Extreme II, which were judged to impose the highest vertical and lateral loads on the piers for unfactored and factored conditions, respectively. GZA conducted initial evaluations on a non-corroded section. The results of these analyses are summarized below:

- Piers 2 and 3 achieve a pinned condition at 43 feet below the bottom of the pile cap, which is approximately 4 and 6 feet above the pile tip at Piers 2 and 3, respectively.
- The depth to pinned condition was not significantly different between the analyzed Service and Extreme load cases, but maximum deflection and pile stress were greater for the Extreme load case.
- Pier 2, Extreme II load case controls the pier pile lateral design (excluding Pier 1) based on deflection and pile stress considerations. The maximum deflection and combined stress for this load case were approximately 0.9 inches and 21 ksi, respectively.

GZA subsequently conducted an analysis assuming a corroded pile section for Pier 2, Extreme Load Case. The depth to fixity was 43 feet below the pile cap, the same as the uncorroded analysis, but the maximum deflection and pile stress increased to 1.4 inches and 26 ksi, respectively. Recommended depths to a pinned pile condition are presented in **Section 6.4.2**.

Calderwood Engineering selected PP24x0.75 battered piles to support the piers. GZA did not conduct additional GROUP analyses for the updated pile types.

5.5.5 Rock Anchors

We understand that the Pier 1 piles will be driven open-ended, cleaned-out and will have rock anchors installed to enhance constraint of at the pile toe due to the limited overburden thickness. We understand that the rock anchors will not be subject to direct uplift loading under service, strength or extreme load case conditions, but since they will be resisting shear deformation a rock anchor design tension load of 50 kips has been selected by Calderwood Engineering as an anchor design basis. Design recommendations for rock anchors are provided in **Section 6.5** of this report.

5.6 SEISMIC DESIGN CONSIDERATIONS

The proposed bridge configuration, consisting of pile bent piers and integral abutments, is anticipated to primarily experience earthquake forces resulting from embedment of the abutment piles and integral abutments. Therefore, the subsurface profile considered for seismic design includes the proposed approach fills (including backfill behind and beneath abutments), and the soil and rock conditions encountered at Abutments 1 and 2.

The soil profile at the abutments consists of a mixture of soft to stiff silt to sandy silt and loose to very dense sand, with bedrock generally ranging from about 35 to 55 feet below grade. LRFD allows consideration of the rock within the upper 100 feet of the profile. However, the profile used to determine the site class was conservatively evaluated by including only the soil, ignoring the beneficial effect of the rock, thereby reducing the effective thickness of the profile and neglecting the bedrock in the upper 100 feet.

Because of the mixed soil profile, GZA used a property common to all layers in the profile, shear wave velocity (V_s), to evaluate the seismic site class. Values of V_s were estimated for the individual layers using available in situ and laboratory testing data. GZA's evaluations indicate that the average V_s over the soil profile feet could range between approximately 620 and 650 feet/second. Since these fall between 600 and 1,200 feet/second, Table C3.10.3.1-1 indicates that the proposed bridge should be assigned to Site Class D.

Seismic design parameters are provided in **Section 6.1**.

5.7 FROST PENETRATION

Fill soils are anticipated to be present at the abutments, either as existing fill or imported backfill. Based on the MaineDOT BDG Section 5.2.1, the Freezing Index for the site is 1300, and with low to moderate moisture content (±15 percent) soils, the estimated depth of frost penetration is 5.5 feet.

6.0 RECOMMENDATIONS

6.1 SEISMIC DESIGN

The United States Geological Survey Seismic Design Maps application, Version 3.10, was used to develop parameters for bridge design. Based on the site coordinates, the software provided the recommended AASHTO Response Spectra (Site Class D) for a 7 percent probability of exceedance in 75 years. These results are summarized as follows:

SITE CLASS D SEISMIC DESIGN PARAMETERS	
Parameter	**Design Value**
Fpga	1.6
Fa	1.6
Fv	2.4
As (Period = 0.0 sec)	0.10 g
SDs (Period = 0.2 sec)	0.21 g
SD1 (Period = 1.0 sec)	0.10 g

Per AASHTO Article 3.10.6, the site is assigned to Seismic Zone 1 based on a calculated SD1 of 0.10 g. Per AASHTO Article 4.7.4, bridges in Seismic Zone 1 need not be analyzed for seismic loads, but the minimum requirements specified in AASHTO Articles 4.7.4.4 and 3.10.9 apply.

6.2 EMBANKMENT CONSTRUCTION

The widened approach embankments will be constructed primarily in the inter-tidal zone (Abutment 1 approach) or below low water level (Abutment 2 approach). Therefore, fill placement at Abutment 1 could be completed in-the-dry working with the tides. At Abutment 2, we anticipate that fill placement would need to be conducted either inside of a cofferdam to be placed in-the-dry or will be conducted in-the-wet. Embankment construction in the wet should address permitting considerations for work in the river, which we anticipate would include the use of a silt curtain at a minimum.

The current subgrade material beneath the Abutment 1 approach is anticipated to consist of medium dense sand and gravel. Therefore, conventional embankment construction procedures should be suitable, provided it can be unwatered. The current mudline beneath the Abutment 2 approach is anticipated to be underlain by 2 feet or more of very soft, organic silt. This material should be assumed to consist of "Muck" and should be fully removed in accordance with the MaineDOT Standard Specifications, Section 203.05 to expose suitable, inorganic soil as confirmed by the Resident and/or Engineer. Additional construction considerations are presented in **Section 7.3**.

6.3 ABUTMENT AND WINGWALL DESIGN

- Backfill behind new abutments should consist of MaineDOT Section 703.19, Granular Borrow for Underwater Backfill, BDG Type 4 soil. Recommended soil properties for Type 4 soils to be used as backfill are as follows:
  - Internal Friction Angle of Soil = 32;
  - Soil Total Unit Weight = 125 pcf; and
  - Coefficient of Passive Earth Pressure, K_p= 3.3 (use for design of backwalls and wingwalls);
- Live load surcharge should be applied as a uniform lateral surcharge pressure using the equivalent fill height values developed in accordance with AASHTO Article 3.11.6.4 based on the abutment/wingwall height and distance from the wall backface to the edge of traffic.
- Foundation drainage should be provided in accordance with Section 5.4.1.4 of the BDG.
- We recommend the use of French drains on the uphill side of abutments and wing walls to prevent buildup of differential hydrostatic pressure. Foundation drains should be sloped to drain by gravity and should daylight on the side slope of the approach embankment and/or through weep holes in the abutments.
- Up to about 3 inches of post-construction settlement may occur beneath the Abutment 2 approach embankment. To limit the impact of this settlement on the bridge structure and limit the future abrupt transition from the approach roadway to the bridge (e.g., bump), we recommend use of an approach slab to ease the transition between the pile supported

bridge and the approach embankment. We recommend that Calderwood Engineers consider the anticipated differential settlement and confirm the suitability of the approach slab to survive these deformations without overstress.

6.4 PILE DESIGN

6.4.1 Abutment Pile Design

- The proposed abutments may be supported on HP14x117 ASTM A572, Grade 50 steel (50 ksi yield stress) H-piles driven to the required nominal resistance, anticipated to be developed primarily in end-bearing on or near the bedrock surface.
- Pile installation should be controlled based on dynamic pile testing with signal matching analysis.
- The Abutment 1 piles should be driven to a nominal resistance calculated by dividing the maximum factored pile load of 162 kips by a resistance factor of 0.65, resulting in a required nominal pile resistance of 249 kips.
- The Abutment 2 piles should be designed to resist downdrag loads. Therefore, the piles should be driven to a nominal resistance calculated by dividing the maximum factored pile load of 215 kips by a resistance factor of 0.65, resulting in a required nominal pile resistance of 331 kips.
- Preliminary wave equation analyses indicate that the proposed ASTM A572 Grade 50 HP14x117 piles can be driven to a nominal resistance of 249 kips for 21-foot-long piles at Abutment 1 and 331 kips for 41-foot-long piles at Abutment 2 using a diesel hammer with a rated energy of about 47,000 foot-pounds, without exceeding the allowable driving stress of 45 ksi (0.9Fy for 50 ksi steel). The final penetration resistance was 4 to 5 blows per inch to achieve the required nominal pile resistance, both of which are within the MaineDOT range of 3 to 15 blows per inch. In GZA's experience, the preferred range of final penetration resistance is 6 to 10 blows per inch. A smaller hammer could be used to achieve the required nominal resistance within the preferred penetration resistance range, but the hammer size should satisfy the requirements for abutment and pier piles.
- For the purpose of estimating quantities, it should be assumed that the piles will penetrate to approximately El. -15 at Abutment 1 and El. -38 at Abutment 2. These assume that the top of rock elevations encountered in borings BB-TSGR-101 and BB-TSGR-201 are representative for Abutment 1, and top of rock elevations encountered in borings BB-TSGR-103 and BB-TSGR-205 are representative for Abutment 2, and the piles may penetrate a short distance into the bedrock (1 foot or less).
- We recommend that lateral pile analyses show that the piles meet at least a pinned condition (no translation) under design loads, if a fixed condition is not evident.
- We recommend that one pile at each abutment be dynamically tested to assess driving stress at the maximum factored load and that the combination of dynamic testing and signal matching be used to develop penetration resistance for production driving.
- Splices should be made in accordance with MaineDOT Standard Specification Section 501.09 – Splicing Piles.
- To limit driving damage and enhance the ability of the H-piles to achieve a pinned condition, we recommend that the H-piles be fitted with Associated Pile and Fitting, Rock Injector, HP-80500 Pile Points.

6.4.2 Pier Pile Design

- The proposed piers may be supported on PP24x0.75 (battered) or PP24x0.625 (plumb), ASTM A252, Grade 3 Modified (fy = 50 ksi) steel pipe piles driven to the required nominal resistance, anticipated to be developed primarily in end-bearing on or near the bedrock surface.
- Pile installation should be controlled based on dynamic pile testing with signal matching analysis.
- The piles should be driven to a nominal resistance calculated by dividing the maximum factored pile load of 442 kips by a resistance factor of 0.65, resulting in a required nominal pile resistance of 680 kips.
- Preliminary wave equation analyses indicate that the proposed ASTM A252 Grade 3 Modified (fy = 50 ksi) PP24x0.625 piles can be driven to a nominal resistance of 680 kips for 5-foot-embedded piles at Pier 1 and 26-foot-embedded piles at Pier 3 using a diesel hammer with a rated energy of about 47,000 foot-pounds without exceeding the allowable driving stress of 45 ksi (0.9Fy for 50 ksi steel). The final penetration resistance was 7 to 9 blows per inch to achieve the required nominal pile resistance, both of which are within the MaineDOT range of 3 to 15 blows per inch. PP24x0.75 piles could be driven to an equal or higher nominal resistance.
- For the purpose of estimating quantities, it should be assumed that the piles will penetrate to average tip elevations of approximately El. -21 at Pier 1, El. -39 at Pier 2, and El. -42 at Pier 3. These assume that the top of rock elevations encountered in borings BB-TSGR-301, BB-TSGR-302, and BB-TSGR-303 are representative for Pier 1, Pier 2, and Pier 3, respectively, and the piles may penetrate a short distance into the bedrock.
- We recommend that one pile per pile type at each pier be dynamically tested to assess driving stress at the maximum factored load and that the combination of dynamic testing and signal matching be used to develop penetration resistance for production driving.
- Splices should be made in accordance with MaineDOT Standard Specification Section 501.09 – Splicing Piles.
- To limit driving damage, the steel pipe piles should be fitted with protective driving tips. For Pier 1 piles that will have rock anchors installed through the pipe, the tips should consist of outside cutting shoes. Pipe piles at other locations should be fitted with 60-degree conical pile tips.
- The Pier 1 piles will be cleaned out following initial installation, prior to rock anchor installation. Pile cleanout has the potential to be detrimental to the available pile resistance. Therefore, Pier 1 piles should be re-driven to the required penetration resistance following clean-out. Dynamic pile testing at Pier 1 should be conducted on the re-drive following cleanout.
- Pier piles should be filled with Class “A” concrete and reinforced in accordance with the requirements of Calderwood Engineering.

6.5 ROCK ANCHOR DESIGN

- Rock anchor design should be in accordance with Chapter 11 of AASHTO and the current edition of "Recommendations for Prestressed Rock and Soil Anchors," by the Post-Tensioning Institute (PTI).
- Based on the unconfined compressive strength and our experience with rock anchors in similar formations, we recommend a nominal grout-rock bond stress of 150 psi. A factored grout-rock bond stress of 75 psi is recommended for Strength Limit State design.
- The resistance of the rock mass is typically checked assuming the weight of a conical rock mass extending upward from the midpoint of the bonded length, to the top of rock, and assuming an included cone angle of 75 degrees. A buoyant unit weight of 105 pcf is recommended for bedrock assuming that water levels will be at or above the top of rock. For the planned 12-foot-long anchor, the conical rock mass must extend 9 feet below the top of the anchor zone to provide sufficient weight to resist a 50 kip factored load. However, because uplift is not anticipated and 50 kips is used only as a design basis, the rock mass resistance is considered to be acceptable. Therefore, a 12-foot-long bond length is suitable for the rock anchors.
- We recommend that all-thread bar-type anchors be used rather than wire strand-type. It has been our experience that corrosion protection is more straightforward and reliable, that they are more readily installed and tested, and that the lock-off is more reliable.
- The anchors should be provided with double corrosion protection systems in accordance with PTI Class I protection criteria. Either two-stage grouting (i.e., anchor is grouted from the tip to the top of the bonded length, tested and tensioned prior to placing the grout through the unbonded length) or one-stage grouting (i.e., use of pregrouted encapsulations and or greased sleeve bond breakers) are acceptable for anchor installation.
- Rock anchors should be tested in accordance with AASHTO Article 11.9.8.1, which is based on PTI criteria. We recommend that proof tests be performed on every anchor and that a performance test be performed on at least one anchor per foundation location. The maximum test load has traditionally been set at 1.33 multiplied by the design load (1.33 DL) per PTI standards, where DL is an unfactored load. We recommend that the maximum test load (1.33 DL) be established as the maximum of: (1) 1.33 multiplied by the maximum nominal Service load combination, or (2) 1.0 multiplied by the maximum factored Strength load combination. Rock anchor test load increments should then be established using ratios of DL designated in PTI.
- The recommended maximum test load (1.33 times service load or 1.0 time factored load) is the minimum acceptable test load to confirm adequate anchor capacity. Calderwood Engineering may select a higher maximum test load if desired. However, the maximum test load should be less than or equal to the guaranteed ultimate tensile strength multiplied by a resistance factor of 0.8, and the maximum test load should not create a grout-rock bond stress exceeding the maximum factored grout-rock bond strength.

7.0 CONSTRUCTION CONSIDERATIONS

This section provides guidance regarding quality control during pile installation, excavation, dewatering, and foundation subgrade preparation and protection. These items are given in the paragraphs that follow.

7.1 PILE INSTALLATION CONTROL

We recommend that the pile installation be controlled using wave equation analysis and field logging of the pile installation and that final penetration resistance be based on dynamic pile testing with signal matching analysis. As previously noted, the piles should be driven to a nominal capacity calculated by dividing the maximum factored pile load by a resistance factor of 0.65, per AASHTO Table 10.5.5.2.3-1.

AASHTO Table 10.5.5.2.3-1 requires that at least one load test with signal matching be performed per substructure to use a resistance factor of 0.65. Therefore, it is recommended that eight (8) PDA tests with Signal Matching be completed for the project, including one pile at each abutment (two total) and one pile per pile type at each pier (six total). If only PP24x0.75 pipe piles are used, the PDA tests on pier piles would be reduced from six tests to three.

Consistent with MaineDOT practice, one restrike test is recommended at the first abutment and the first pier constructed. Additional PDA testing may be recommended if unanticipated conditions are encountered during installation, including early pile take-up, pile driving out-of-plumb, or otherwise unexplained variations in hammer performance.

7.2 PILE INSTALLATION AND CONCRETE FILL

Pipe piles at Piers 2 and 3 will be driven closed-ended, with a conical tip. Therefore, it is anticipated that reinforcing cages and concrete fill can be placed immediately following installation as desired by the Contractor.

Pipe piles at Pier 1 will be driven open-ended. Outside-flange, cast steel driving shoes are recommended to protect the pile tips against driving damage and allow for easier installation of rock anchors (as compared to inside-flange driving shoes). Piles should be cleaned out using air-lift methods to the top of bedrock prior to installation of rock anchors. The cleaned-out piles should be re-driven to the required penetration resistance. Dynamic pile testing with signal matching analysis should be conducted during the re-drive.

A concrete plug should be poured in the lower portion of the pile following cleanout and prior to anchor installation, to promote development of a seal between the rock anchor casing and top of bedrock.

7.3 PILE OBSTRUCTIONS

Pre-drilling, pre-excavation or spudding may be necessary to bypass potential obstructions, such as the boulders and rock fill encountered in boring drilled near Piers 2 and 3.

7.4 EXCAVATION, TEMPORARY LATERAL SUPPORT AND DEWATERING

Excavation will likely be required to a depth of about 2 feet below mudline beneath the Abutment 2 approach to remove a layer of very soft, organic silt. The exposed soil beneath the organic silt will likely consist of saturated sandy silt or silty sand, which could be difficult to manage during fill placement. If fill placement at Abutment 2 is conducted in-the-wet, it may be feasible to place a very thick initial lift of well-graded gravel or blasted rock fill before compacting to get the embankment work level up to the inter-tidal zone, from where it can be worked around the tide cycle.

Excavations for abutment foundations are anticipated to extend up to 7 feet below existing grades on the east side of each abutment, extending into the existing embankment side slope. We anticipate that sufficient space is available to prepare excavation slopes as sloped, open cuts. If encroachment into the road is such that traffic cannot be maintained, a sheetpile wall would be required to facilitate abutment construction. In all cases, temporary excavations should comply with Occupational Safety and Health Administration excavation safety requirements.

Groundwater is anticipated to vary tidally and will likely be above the bottom of pile cap level at Abutment 2 during high tide. Work that is required to be completed in the dry below high tide should be phased to avoid high tide condition.

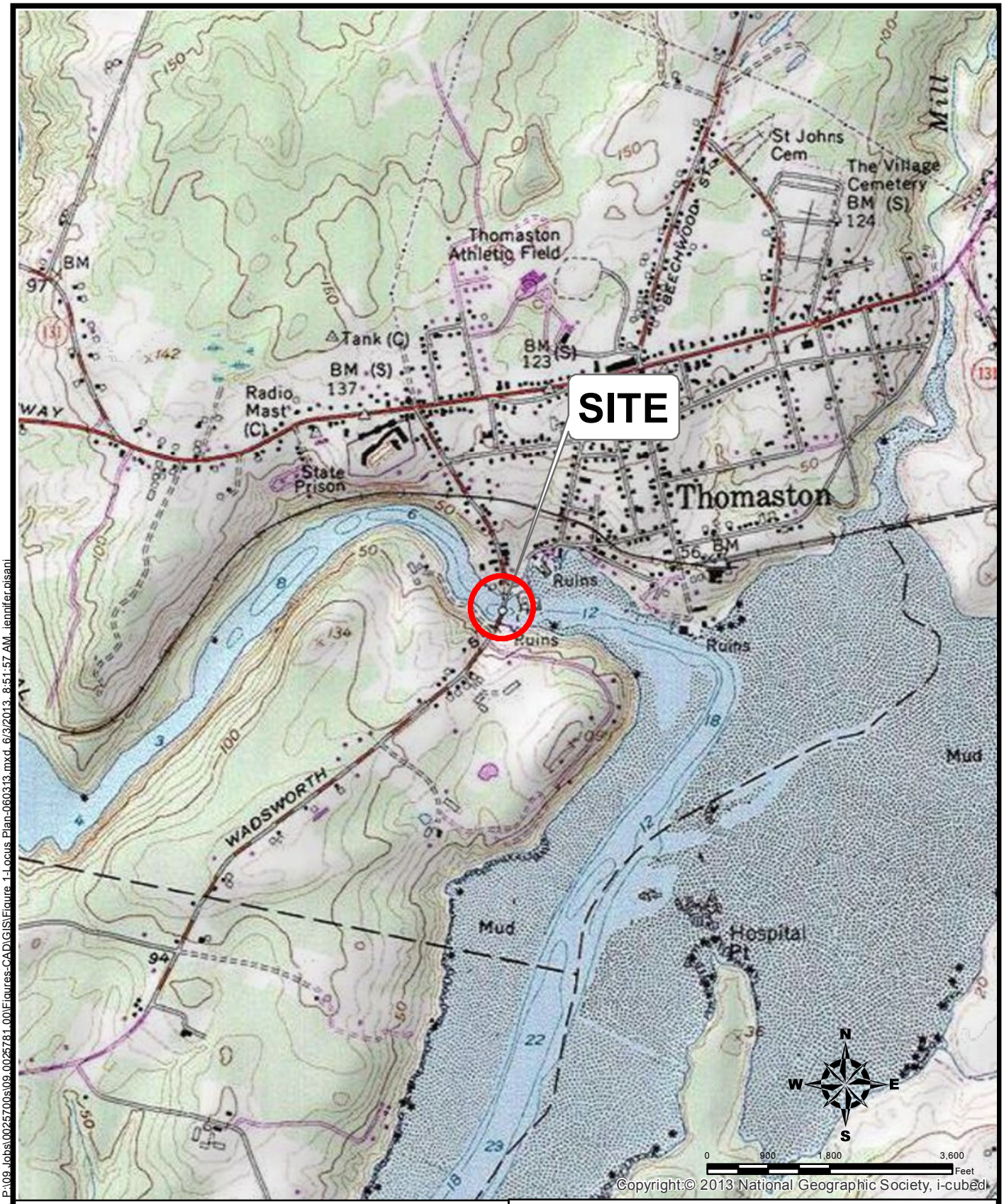
The contractor should be responsible for controlling groundwater, surface runoff, infiltration and water from all other sources by methods which preserve the undisturbed condition of the subgrade and permit foundation construction in-the-dry. Discharge of pumped groundwater and river water should comply with all local, State, and federal regulations.

7.5 REUSE OF ON-SITE MATERIALS

Based on the test boring results, the three fill samples tested had greater than 20 percent passing the No. 200 sieve, indicating the fill does not meet MaineDOT specifications for Granular Borrow and/or Granular Borrow for Underwater Backfill and is unsuitable for use as structural backfill. The material is considered suitable for use as Common Borrow.

If the contractor wishes to reuse excavated material as embankment fill or in other areas, we recommend that the proposed material be stockpiled and tested for grain size distribution. Stockpiled materials meeting the appropriate MaineDOT specifications may be reused on the project.

FIGURES



UNLESS SPECIFICALLY STATED BY WRITTEN AGREEMENT, THIS DRAWING IS THE SOLE PROPERTY OF GZA GEOENVIRONMENTAL, INC. (GZA). THE INFORMATION SHOWN ON THE DRAWING IS SOLELY FOR THE USE BY GZA'S CLIENT OR THE CLIENT'S DESIGNATED REPRESENTATIVE FOR THE SPECIFIC PROJECT AND LOCATION IDENTIFIED ON THE DRAWING. THE DRAWING SHALL NOT BE TRANSFERRED, REUSED, COPIED, OR ALTERED IN ANY MANNER FOR USE AT ANY OTHER LOCATION OR FOR ANY OTHER PURPOSE WITHOUT THE PRIOR WRITTEN CONSENT OF GZA, ANY TRANSFER, REUSE, OR MODIFICATION TO THE DRAWING BY THE CLIENT OR OTHERS, WITHOUT THE PRIOR WRITTEN EXPRESS CONSENT OF GZA, WILL BE AT THE USER'S SOLE RISK AND WITHOUT ANY RISK OR LIABILITY TO GZA.

SOURCE : THIS MAP CONTAINS THE ESRI ARCGIS ONLINE USA TOPOGRAPHIC MAP SERVICE, PUBLISHED DECEMBER 12, 2009 BY ESRI ARCIMS SERVICES AND UPDATED AS NEEDED. THIS SERVICE USES UNIFORM NATIONALLY RECOGNIZED DATUM AND CARTOGRAPHY STANDARDS AND A VARIETY OF AVAILABLE SOURCES FROM SEVERAL DATA PROVIDERS

WADSWORTH STREET BRIDGE REPLACMENT
MAINE DOT WIN 16755
THOMASTON, MAINE

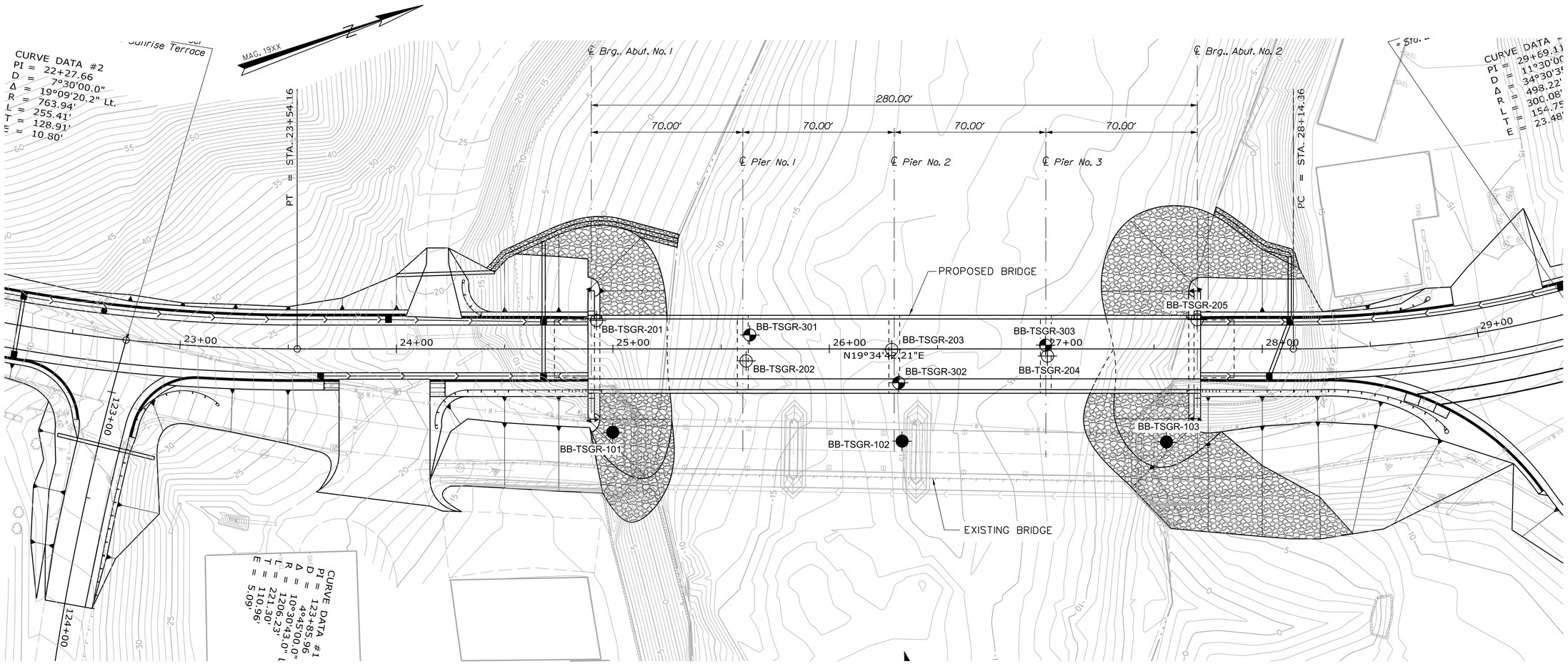
PREPARED BY:
GZA GeoEnvironmental, Inc.
Engineers and Scientists
www.gza.com

PREPARED FOR:
CALDERWOOD ENGINEERING

LOCUS PLAN

PROJ MGR: ARB	REVIEWED BY: ARB	CHECKED BY: CLS	FIGURE 1
DESIGNED BY: ARB	DRAWN BY: JAP	SCALE: 1 in = 2,000 ft	
DATE: JUNE 2014	PROJECT NO. 09.0025781.00	REVISION NO.	

© 2013 - GZA GeoEnvironmental, Inc. P:\09_Jobs\0025700s\09.0025781.00\Figures-CAD\GIS\Figure 1-Locus Plan-060313.mxd, 6/3/2013, 8:51:57 AM, jennifer.oisani



PLAN

25 0 25 50

Scale in Feet

LEGEND:

BB-TSGR-303 — DESIGNATION AND LOCATION OF 300-SERIES TEST BORING (DRILLED BY MAINE TEST BORING FOR GZA)

BB-TSGR-205 — DESIGNATION AND LOCATION OF 200-SERIES TEST BORING (DRILLED BY NORTHERN BORING FOR MAINEDOT)

BB-TSGR-103 — DESIGNATION AND LOCATION OF 100-SERIES TEST BORING (DRILLED BY/FOR MAINEDOT)

NOTES:

1) BASE MAP DEVELOPED FROM ELECTRONIC FILES PROVIDED BY MAINEDOT ON MARCH 29, 2013 (FILES INCLUDED BRIDGE.DWG, CONTOURS.DWG, TOPO.DWG, AND WETLANDS.DWG).

2) AS-DRILLED MUDLINE ELEVATIONS AND BORING LOCATIONS FOR BB-TSGR-300 SERIES WATER BORINGS WERE DETERMINED BY MAINEDOT BY SURVEYING THE TOP OF CASING AT THE BEGINNING OF DRILLING WHEN IT TOUCHED THE MUDLINE.

3) MUDLINE ELEVATIONS AND BORING LOCATIONS FOR BB-TSGR-100 AND -200 SERIES BORINGS WERE PROVIDED BY MAINEDOT.

4) THE BB-TSGR-300 SERIES BORINGS WERE PERFORMED BY MAINE TEST BORING OF HERMON, MAINE BETWEEN APRIL 16 AND 18, 2013 AND OBSERVED BY GZA PERSONNEL.

5) THE BB-TSGR-200 SERIES TEST BORINGS WERE PERFORMED BY NORTHERN TEST BORING AND OBSERVED BY MAINEDOT BETWEEN SEPTEMBER 27 AND OCTOBER 3, 2011.

6) THE BB-TSGR-100 SERIES TEST BORINGS WERE PERFORMED AND OBSERVED BY MAINEDOT BETWEEN NOVEMBER 1 AND 3, 2010.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-1675(500)

WIN 16755.00

BRIDGE NO. 2904

GEOTECH PLANS

		BY	DATE
PROJ. MANAGER	MAP		
DESIGN-DETAILED	ARB	ETL/ATV	SEPT 2014
CHECKED-REVIEWED	CLS		SEPT 2014
DESIGN2-DETAILED2	------	------	------
DESIGN3-DETAILED3	------	------	------
REVISIONS 1	------	------	------
REVISIONS 2	------	------	------
REVISIONS 3	------	------	------
REVISIONS 4	------	------	------
FIELD CHANGES	------	------	------

SIGNATURE

P.E. NUMBER

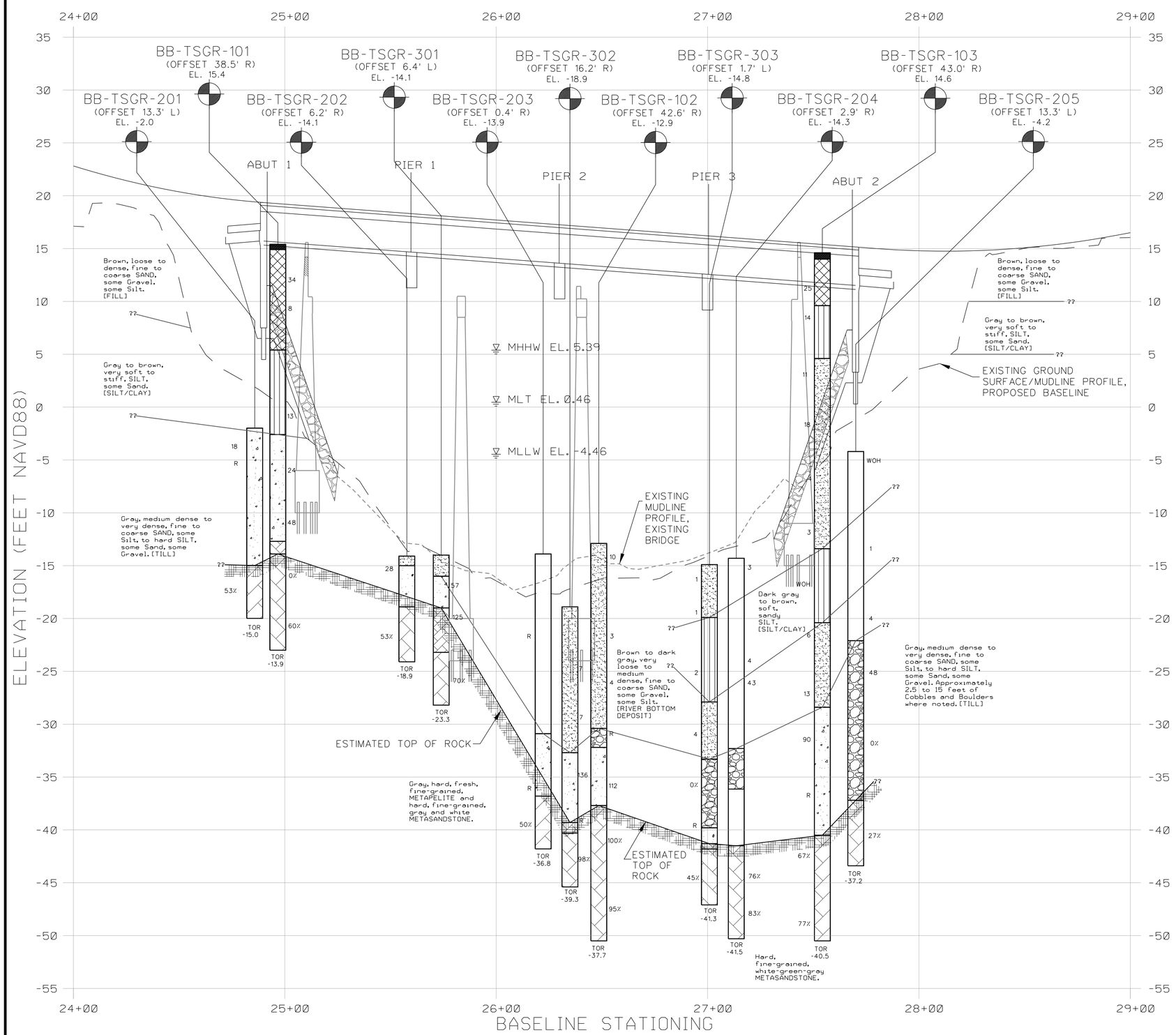
DATE

WADSWORTH STREET BRIDGE
ST. GEORGE RIVER
THOMASTON KNOX COUNTY

BORING LOCATION PLAN

FIGURE 2

Filename: ...\CAD\11_Boring-Plan.dgn Division: HIGHWAY Username: common Date:9/10/2014



NOTES:

1) BASELINE PROFILE DEVELOPED FROM ELECTRONIC FILE PROVIDED BY MAINEDOT ON MARCH 29, 2013 ENTITLED Profile.dgn.

2) THIS GENERALIZED INTERPRETIVE SOIL PROFILE IS INTENDED TO CONVEY TRENDS IN SUBSURFACE CONDITIONS. THE BOUNDARIES BETWEEN STRATA ARE APPROXIMATE AND IDEALIZED, AND HAVE BEEN DEVELOPED BY INTERPRETATIONS OF WIDELY SPACED EXPLORATIONS AND SAMPLES. ACTUAL SOIL TRANSITIONS MAY VARY AND ARE PROBABLY MORE ERRATIC. FOR MORE SPECIFIC INFORMATION REFER TO THE EXPLORATION LOGS.

LEGEND:

BB-TSGR-303 — BOREHOLE DESIGNATION
(OFFSET 1.7' L) — OFFSET FROM BASELINE
EL. -14.8 — GROUND SURFACE/MUDLINE ELEVATION AT BORING LOCATION

WOH — WEIGHT OF HAMMER

30 — ENERGY CORRECTED STANDARD PENETRATION TEST (SPT) N-VALUE

R — REFUSAL

30% — RQD OF CORE RUN ROCK CORE

TOR -41.3 — TOP OF CORED BEDROCK (FT-NAVD 1988)

BORING STICK HATCH LEGEND:

FILL
SILT/CLAY
RIVER BOTTOM DEPOSIT
UNKNOWN (LACK OF SAMPLING)
COBBLES & BOULDERS
GLACIAL TILL
WEATHERED ROCK
CORED BEDROCK

VERTICAL SCALE IN FEET
0 5 10 20 30
0 10 20 30
HORIZONTAL SCALE IN FEET

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-1675(500)

WIN 16755.00

BRIDGE NO. 2904

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	MAP		
DESIGN-DETAILED	ARB	ETL/AIV	SEPT 2014
CHECKED-REVIEWED	CLS		
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

WADSWORTH STREET BRIDGE
ST. GEORGE RIVER
THOMASTON — KNOX COUNTY

INTERPETIVE SUBSURFACE PROFILE

FIGURE 3

Filename: ...\CAD\12-Profile.dgn Division: HIGHWAY Username: common Date:9/10/2014

APPENDIX A

LIMITATIONS

LIMITATIONS

Use of Report

1. GZA GeoEnvironmental, Inc. (GZA) prepared this report on behalf of, and for the exclusive use of our Client for the stated purpose and location identified in the Proposal for Services and/or Report. Use of this report, in whole or in part, at other locations, or for other purposes, may lead to inappropriate conclusions; and we do not accept any responsibility for the consequences of such use. Further, reliance by any party not expressly identified in the agreement, for any use, without our prior written permission, shall be at that party's sole risk, and without any liability to GZA.

Standard of Care

2. GZA's findings and conclusions are based on the work conducted as part of the Scope of Services set forth in Proposal for Services and/or Report, and reflect our professional judgment. These findings and conclusions must be considered not as scientific or engineering certainties, but rather as our professional opinions concerning the limited data gathered during the course of our work. If conditions other than those described in this report are found at the subject location, or the design has been altered in any way, GZA shall be so notified and afforded the opportunity to revise the report,as appropriate, to reflect the unanticipated changed conditions .

3. GZA's services were performed using the degree of skill and care ordinarily exercised by qualified professionals performing the same type of services, at the same time, under similar conditions, at the same or a similar property. No warranty, expressed or implied, is made.

Subsurface Conditions

4. The generalized soil profile provided in our Report is based on widely-spaced subsurface explorations and are intended only to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and were based on our assessment of subsurface conditions. The composition of strata, and the transitions between strata, may be more variable and more complex than indicated. For more specific information on soil conditions at a specific location refer to the exploration logs.

5. In preparing this report, GZA relied on certain information provided by the Client, state and local officials, and other parties referenced therein which were made available to GZA at the time of our evaluation. GZA did not attempt to independently verify the accuracy or completeness of all information reviewed or received during the course of this evaluation.

6. Water level readings have been made in test holes as described in the report at the specified times and under the stated conditions. These data have been reviewed and interpretations have been made in this Report. Fluctuations in the level of the groundwater however occur due to temporal or spatial variations in areal recharge rates, soil heterogeneities, the presence of subsurface utilities, and/or natural or artificially induced perturbations. The water table encountered in the course of the work may differ from that indicated in the Report.

7. GZA's services did not include an assessment of the presence of oil or hazardous materials at the property. Consequently, we did not consider the potential impacts (if any) that contaminants in soil or groundwater may have on construction activities, or the use of structures on the property.

8. Recommendations for foundation drainage, waterproofing, and moisture control address the conventional geotechnical engineering aspects of seepage control. These recommendations may not preclude an environment that allows the infestation of mold or other biological pollutants.

Compliance with Codes and Regulations

9. We used reasonable care in identifying and interpreting applicable codes and regulations. These codes and regulations are subject to various, and possibly contradictory, interpretations. Compliance with codes and regulations by other parties is beyond our control.

APPENDIX B

PREVIOUS TEST BORING LOGS

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wadsworth Street Bridge #2904 carries Wadsworth Street over St. George River
Location: Thomaston, Maine

Boring No.: BB-TSGR-101
PIN: 16755.00

Driller:	MaineDOT	**Elevation (ft.)**	15.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/2/10; 07:30-13:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	24+99.9, 38.5 Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	18.0 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	14.9	Pavement — 0.5	
	1D	24/15	2.0 - 4.0	11/13/11/8	24	34			Brown, moist, dense, fine to coarse SAND, some gravel, some silt.	G#240060 A-1-b, SM WC=5.7%
								10.9	4.5	
5	2D	24/24	5.0 - 7.0	4/3/3/4	6	8	24		Brown, wet, loose, Silty fine to coarse SAND, trace gravel, trace clay.	G#240061 A-4, SC-SM WC=12.5%
							25			
							24			
							42			
							57			
10	3D	24/15	10.0 - 12.0	7/3/2/5	5	7	17	5.4	10.0 — Brown, wet, medium stiff, sandy SILT, trace gravel.	G#240062 A-4, ML WC=23.5%
							37			
							41			
							36			
							60			
15	4D	24/8	15.0 - 17.0	5/5/4/3	9	13	6	0.4	15.0 — Brown, wet, stiff, fine to coarse Sandy SILT, trace gravel, trace clay. Changed to NW Casing at 15.0 ft bgs.	G#240063 A-4, CL-ML WC=13.5%
							14			
							18			
							25	-2.6	18.0	
							30			
20	5D	24/10	20.0 - 22.0	3/4/13/14	17	24	7		Grey, wet, medium dense, fine to coarse SAND, some gravel, some silt, trace clay, (Till).	G#240064 A-2-4, SC-SM WC=10.5%
							48			
							47			
							69			
25							112	-8.6	24.0	

Remarks:

GZA reviewed and edited draft logs provided by MaineDOT. Soil descriptions were edited as needed to make visual descriptions consistent with laboratory test data; GZA did not visually observe collected soil samples. Rock RQD and descriptions were edited based on a visit by a GZA engineer to MaineDOT's laboratory in Bangor on April 17, 2013 to observe the core samples based on our observations.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wadsworth Street Bridge #2904 carries Wadsworth Street over St. George River
Location: Thomaston, Maine

Boring No.: BB-TSGR-101
PIN: 16755.00

Driller:	MaineDOT	**Elevation (ft.)**	15.4	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/2/10; 07:30-13:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	24+99.9, 38.5 Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	18.0 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/19	25.0 - 27.0	5/14/20/25	34	48	14			Grey-brown, wet, dense, Silty fine to coarse SAND, little gravel, trace clay, (Till).	G#240065 A-4, SC-SM WC=11.0%
							20				
							21				
							60	-12.7		28.1	
	R1	49.2/27	29.3 - 33.4	RQD = 0%			45			Top of Bedrock at Elev -12.7 Weathered BEDROCK. Roller Coned ahead of casing to 29.3 ft bgs.	
30							NQ-2 63			R1:Hard to moderately hard, moderately weathered, fine grained, gray, METAPELITE. Joints are extremely close to close, high angle, undulating, rough, fresh to decomposed, partially open to open, healed joints in intact pieces. Hydrothermally altered. Rock Mass Quality = Very Poor. R1:Core Times (min:sec) 29.3-30.3 ft (3:30) 30.3-31.3 ft (4:10) 31.3-32.3 ft (5:20) 32.3-33.3 ft (6:08) 33.3-33.4 ft (2:00) 55% Recovery Core Blocked	
							51				
							54				
	R2	60/48	33.4 - 38.4	RQD = 60%							
35								-23.0		R2:Hard, fresh, fine grained, gray, METAPELITE. Primary joints are extremely close to moderately spaced, moderately dipping to high angle, undulating, rough, fresh, partially open, occasional silt seam. Secondary joints are close to moderately spaced, low angle to horizontal, undulating, rough, fresh, tight to open. Highly fractured zones 33.4-34.1 and 36.0-36.8. Rock Mass Quality = Fair. R2:Core Times (min:sec) 33.4-34.4 ft (3:20) 34.4-35.4 ft (7:25) 35.4-36.4 ft (4:10) 36.4-37.4 ft (4:30) 37.4-38.4 ft (6:40) 80% Recovery	
40										38.4	
45										**Bottom of Exploration at 38.40 feet below ground surface.**	
50											

Remarks:

GZA reviewed and edited draft logs provided by MaineDOT. Soil descriptions were edited as needed to make visual descriptions consistent with laboratory test data; GZA did not visually observe collected soil samples. Rock RQD and descriptions were edited based on a visit by a GZA engineer to MaineDOT's laboratory in Bangor on April 17, 2013 to observe the core samples based on our observations.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wadsworth Street Bridge #2904 carries Wadsworth Street over St. George River
Location: Thomaston, Maine

Boring No.: BB-TSGR-102
PIN: 16755.00

Driller:	MaineDOT	**Elevation (ft.)**	-12.9	**Auger ID/OD:**	N/A
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/2/10-11/3/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	26+33.6, 42.6 Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	River Boring,
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
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Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/8	0.0 - 2.0	3/4/3/6	7	10	[a]HP		[a]HP=Hydraulic Push Brown, wet, loose, fine to coarse SAND, some gravel, some silt, shells.	
							HP			
							HP			
							HP			
							8		WOOD in wash water from 4.5-5.5 ft bgs.	
5							14			
							7	-18.9	6.0	
	2D	24/12	7.0 - 9.0	2/2/WOH/WOH	2	3	3		Grey, wet, very loose, fine to coarse SAND, some gravel, trace silt.	G#240066 A-1-b, SW-SM WC=15.3%
							3			
							17			
10							23			
							11			
	3D	24/15	12.0 - 14.0	1/1/2/4	3	4	6		Grey, wet, very loose, fine to coarse SAND, little gravel, little silt.	G#240067 A-1-b, SW-SM WC=25.3%
							14			
							15			
15							17			
							26			
	MD	6/0	17.0 - 17.5	50(6")	---		33		Changed to NW Casing at 17.0 ft bgs. BOULDER from 17.5-19.3 ft bgs. Roller Coned ahead to 22.0 ft bgs.	
							17			
							19			
20							17			
							60	-33.9	21.0	
	4D	21.6/18	22.0 - 23.8	19/33/47/50(3.6)	80	112	50		Grey, wet, hard, fine to coarse, sandy SILT, little gravel, trace clay, (Till). Roller Coned ahead to 24.8 ft bgs.	G#240068 A-4, CL-ML WC=12.7%
							65			
25	R1	56.4/56.4	24.8 - 29.5	RQD = 100%			[b]100	-37.7	[b]100 blows for 0.8 ft. 24.8	

Remarks:

12" Concrete Bridge Deck.
28.3 ft from Bridge Deck to Ground.
GZA reviewed and edited draft logs provided by MaineDOT. Soil descriptions were edited as needed to make visual descriptions consistent with laboratory test data; GZA did not visually observe collected soil samples.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wadsworth Street Bridge #2904 carries Wadsworth Street over St. George River
Location: Thomaston, Maine

Boring No.: BB-TSGR-102
PIN: 16755.00

Driller:	MaineDOT	**Elevation (ft.)**	-12.9	**Auger ID/OD:**	N/A
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/2/10-11/3/10	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	26+33.6, 42.6 Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	River Boring,
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							NQ-2			Top of Bedrock at Elev. -37.7 ft.	
										R1:Bedrock: Grey, green and white, fresh, mixture of SCHIST and MARBLE with kyanite crystals and pyrite. (Rocks of Islesboro)	
										Rock Mass Quality = Excellent.	
										R1:Core Times (min:sec)	
										24.8-25.8 ft (3:30)	
										25.8-26.8 ft (3:15)	
										26.8-27.8 ft (3:25)	
										27.8-28.8 ft (3:30)	
	R2	60/60	29.5 - 34.5	RQD = 95%						28.8-29.5 ft (3:00) 100% Recovery	
30										Core Blocked	
										R2:Bedrock: Dark grey, fresh, SCHIST with fragments of marble. (Rocks of Islesboro)	
										Rock Mass Quality = Excellent.	
										R2:Core Times (min:sec)	
										29.5-30.5 ft (3:30)	
										30.5-31.5 ft (4:05)	
										31.5-32.5 ft (3:15)	
										32.5-33.5 ft (3:40)	
								-47.4		33.5-34.5 ft (3:20) 100% Recovery	
35										34.5	
										Bottom of Exploration at 34.50 feet below ground surface.	
40											
45											
50											

Remarks:

12" Concrete Bridge Deck.
28.3 ft from Bridge Deck to Ground.
GZA reviewed and edited draft logs provided by MaineDOT. Soil descriptions were edited as needed to make visual descriptions consistent with laboratory test data; GZA did not visually observe collected soil samples.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wadsworth Street Bridge #2904 carries Wadsworth Street over St. George River
Location: Thomaston, Maine

Boring No.: BB-TSGR-103
PIN: 16755.00

Driller:	MaineDOT	**Elevation (ft.)**	14.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/1/10; 08:30-15:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	27+54.3, 43.0 Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	19.0 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	14.0	Pavement (0.6)	
	1D	24/14	2.0 - 4.0	5/6/12/15	18	25			Brown, damp, medium dense, fine to coarse SAND, some gravel, some silt.	G#240069 A-2-4, SM WC=8.0%
5	2D	24/15	5.0 - 7.0	7/5/5/5	10	14	25	9.6	Brown, moist, stiff, fine to coarse, sandy SILT, some gravel. (5.0)	G#240070 A-4, ML WC=12.0%
							28			
							44			
							31			
							28			
10	3D	24/10	10.0 - 12.0	4/4/4/4	8	11	6	4.6	Brown, wet, medium dense, fine to coarse SAND, some gravel, little silt. (10.0)	G#240071 A-1-b, SM WC=13.4%
							22			
							26			
							21			
							19			
15	4D	24/8	15.0 - 17.0	3/5/8/5	13	18	12		Similar to above.	
							40			
							23			
							30	-3.9	(18.5)	
							19			
20	5D	24/9	20.0 - 22.0	2/1/2/1	3	4	19		Olive, wet, very loose, Silty, fine to course SAND, trace gravel, trace clay.	G#240072 A-4, SC-SM WC=13.9%
							16			
							31			
							32			
25							37			

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wadsworth Street Bridge #2904 carries Wadsworth Street over St. George River
Location: Thomaston, Maine

Boring No.: BB-TSGR-103
PIN: 16755.00

Driller:	MaineDOT	**Elevation (ft.)**	14.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/1/10; 08:30-15:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	27+54.3, 43.0 Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	19.0 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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MV = Unsuccessful Insitu Vane Shear Test attempt
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SSA = Solid Stem Auger
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RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
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N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	6D	24/6	25.0 - 27.0	2/1/1/1	2	3	32		Grey, wet, very loose, Silty fine to coarse SAND, some gravel, trace clay.	
							31			
							40			
							38	-13.4	28.0	
							39			
30	7D	24/20	30.0 - 32.0	WOH/WOH/WOH/ WOH	---		36		Grey, wet, very soft, SILT, some fine sand, little clay.	G#240073 A-4, ML WC=36.2% Non-Plastic
							42			
	MV1		32.0 - 32.0	Would not push			41		Failed 55x110 mm vane attempt.	
							42			
							44			
35	MV2	24/6	35.0 - 35.0	Would not push	4	6	3	-20.4	35.0 Failed 55x110 mm vane attempt.	
	8D		35.0 - 37.0	2/2/2/2					Brown, wet, loose, Silty, fine to coarse SAND, little gravel, shells. Changed to NW Casing at 35.0 ft bgs.	
							5			
							8			
							13			
							18			
40	9D	24/15	40.0 - 42.0	WOH/3/6/5	9	13	22		Brown, wet, medium dense, fine to coarse SAND, some gravel, little silt.	G#240074 A-1-b, SM WC=29.5%
							59			
							77			
							99	-28.4	43.0	
							110			
45	10D	24/20	45.5 - 47.5	21/33/31/34	64	90	40		Grey, wet, hard, fine to coarse Sandy SILT, trace gravel, trace clay, (Till). Roller Coned ahead to 50.0 ft bgs.	G#240075 A-4, CL-ML WC=13.0%
							27			
							21			
							19			
50							30			

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wadsworth Street Bridge #2904 carries Wadsworth Street over St. George River
Location: Thomaston, Maine

Boring No.: BB-TSGR-103
PIN: 16755.00

Driller:	MaineDOT	**Elevation (ft.)**	14.6	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD 88	**Sampler:**	Standard Split
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	11/1/10; 08:30-15:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	27+54.3, 43.0 Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	19.0 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
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N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
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Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
50	11D	6/6	50.0 - 50.5	50(6")	---		69	-35.4		Grey, wet, very dense, fine to coarse SAND, some silt, some gravel. Roller Coned ahead to 55.1 ft bgs. (50.0)	G#245451 A-2-4, SM WC=12.6%
							55				
							43				
							43				
							40				
55	R1	60/60	55.1 - 60.1	RQD = 67%			a30 NQ-2	-40.5		a30 blows for 0.1 ft. (55.1) Top of Bedrock at Elev. -40.5 ft. R1:Very hard, fresh, fine grained, white, METASANDSTONE. Primary joints are close to moderately spaced, low angle to horizontal, undulating, rough, fresh to discolored, tight to partially open. Secondary joints are moderately spaced, high angle, planar, rough, discolored, tight, healed fractures throughout. Rock Mass Quality = Fair. R1:CoreTimes (min:sec) 55.1-56.1 ft (2:25) 56.1-57.1 ft (2:25) 57.1-58.1 ft (2:05) 58.1-59.1 ft (2:00) 59.1-60.1 ft (2:00) 100% Recovery	
60	R2	60/60	60.1 - 65.1	RQD = 77%						R2: Very hard, fresh, fine grained, white, METASANDSTONE. Primary joints are close to moderately spaced, low angle, undulating, rough, fresh to discolored, tight. Secondary joints are moderately spaced, moderately dipping, planar, rough, discolored, tight, healed fractures throughout. Rock Mass Quality = Good. R2:CoreTimes (min:sec) 60.1-61.1 ft (2:20) 61.1-62.1 ft (2:15) 62.1-63.1 ft (2:00) 63.1-64.1 ft (2:00) 64.1-65.1 ft (2:00) 100% Recovery	
65								-50.5		(65.1) **Bottom of Exploration at 65.10 feet below ground surface.**	
70											
75											

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wadsworth Street Bridge #2904 carries Wadsworth Street over St. George River
Location: Thomaston, Maine

Boring No.: BB-TSGR-201
PIN: 16755.00

Driller:	Northern Test Boring	**Elevation (ft.)**	-1.99	**Auger ID/OD:**	N/A
Operator:	Mike/Nick	**Datum:**	NAVD88	**Sampler:**	Standard Split
Logged By:	B. Wilder	**Rig Type:**	Diedrick D-50 on Raft	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/30/11; 12:30-16:30	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	24+91.7, 13.3 ft Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	Tidal Water
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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SSA = Solid Stem Auger
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RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
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N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/12	0.0 - 2.0	5/7/6/4	13	18	30		Brown, very wet, medium dense, gravelly, fine to coarse SAND, trace silt.	G#261759 A-1-a, GP-GM WC=17.6%
							27			
							111	-4.0	2.0	
	2D		3.5 - 4.8	20/27/40(3.6")	---		OPEN HOLE		Grey, wet, very dense, silty, fine to coarse SAND, trace gravel. Roller Coned ahead to 13.0 ft bgs, set in NW Casing. Occasional Cobbles.	G#261760 A-4, SC-SM WC=10.1%
5										
							50			
10							62			
							88			
							106	-15.0	13.0	
	R1	60/57	13.0 - 18.0	RQD = 53%			NQ-2		Top of Bedrock at Elev. -14.99 ft. R1: Very hard, fresh, fine grained, gray to green, METAPELITE. Primary joints are very close to close, low angle to moderately dipping, undulating, rough, discolored, tight to partially open. Secondary joints are moderately spaced, high angle, planar, rough, fresh to discolored, partially open, silt seam, core consistently pitted, occasional calcite stringers. Rock Mass Quality = Fair. R1:Core Times (min:sec) 130.-14.0 ft (4:00) 14.0-15.0 ft (4:15) 15.0-16.0 ft (5:00) 16.0-17.0 ft (5:00) 17.0-18.0 ft (5:30) 95% Recovery Could not get back down, ran out of time with the tide.	
15								-20.0	18.0	
									Bottom of Exploration at 18.00 feet below ground surface.	
20										
25										

Remarks:

Auto Hammer #149
Raft deck to ground ±10.0 ft.
Started Boring at High Tide, had only 3 hours to work on boring.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wadsworth Street Bridge #2904 carries Wadsworth Street over St. George River
Location: Thomaston, Maine

Boring No.: BB-TSGR-202
PIN: 16755.00

Driller:	Northern Test Boring	**Elevation (ft.)**	-14.1	**Auger ID/OD:**	N/A
Operator:	Mike/Nick	**Datum:**	NAVD88	**Sampler:**	Standard Split
Logged By:	B. Wilder	**Rig Type:**	Diedrick D-50 on Raft	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/27/11; 12:30-17:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	25+61.5, 6.2 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	Tidal Water
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/14	0.0 - 2.0	8/11/9/24	20	28	44	-15.1	Black, wet, medium dense, fine to medium SAND, little Silt, shells, (Muck). 1.0	G#261761 A-1-b, SC-SM WC=22.1%
							122		Grey-brown, wet, medium dense, fine to coarse SAND, little silt.	
							175			
							216			
	R1	62.4/62.4	4.8 - 10.0	RQD = 53%			NQ-2	-18.9	4.8 Top of Bedrock at Elev. -18.9 ft. R1: Medium hard, fresh to moderately weathered, fine grained, gray, METAPELITE. Primary joints are close to moderately spaced, high angle, planar, rough, discolored, tight. Secondary joints are close to moderately spaced, low angle to horizontal, undulating, rough, fresh, tight, highly weathered seam 6.6-6.8', foliated along primary joints, calcite stringers throughout. Rock Mass Quality = Fair. R1:Core Times (min:sec) 4.8-5.8 ft (5:06) 5.8-6.8 ft (5:15) 6.8-7.8 ft (4:30) 7.8-8.8 ft (5:00) 8.8-9.8 ft (5:25) 9.8-10.0 ft (3:00) 100% Recovery Casing was too crooked to get back down, barge moved.	q_p=2,270 ksf
5										
10								-24.1	10.0 **Bottom of Exploration at 10.00 feet below ground surface.**	
15										
20										
25										

Remarks:

Auto Hammer #149
Raft deck to ground ±20.0 ft.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wadsworth Street Bridge #2904 carries Wadsworth Street over St. George River
Location: Thomaston, Maine

Boring No.: BB-TSGR-203
PIN: 16755.00

Driller:	Northern Test Boring	**Elevation (ft.)**	-13.9	**Auger ID/OD:**	N/A
Operator:	Mike/Nick	**Datum:**	NAVD88	**Sampler:**	Standard Split
Logged By:	B. Wilder	**Rig Type:**	Diedrick D-50 on Raft	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/30/11; 06:00-11:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	26+28.9, 0.36 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	Tidal Water
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							[a]HP		[a]Hydraulic Push Current was to strong to get sample from 0.0-2.0 ft bgs. 2.0 ft of black muck in wash water.	
								-15.9	2.0	
							8			
							42			
5							36			
							25			
	1D	15.6/12	7.0 - 8.3	2/2/50(3.6")	---		OPEN HOLE		Grey, wet, dense, fine to coarse SAND, little gravel, little silt, (Marine).	G#261762 A-1-b, SC-SM WC=24.7%
10									Roller Coned ahead to 20.0 ft bgs, set in NW Casing.	
							20			
15							30			
							28			
							47	-30.9	17.0	
							50			
							88			
20	2D	16.8/13	20.0 - 21.4	11/16/50(4.8")	---				Grey, wet, dense, silty, fine to coarse SAND, some gravel, (Till). Roller Coned ahead to 22.9 ft bgs.	G#261763 A-4, SC-SM WC=9.1%
	R1	60/49	22.9 - 27.9	RQD = 50%			NQ-2	-36.8	22.9	q_p=1,211 ksf
25									Top of Bedrock at Elev. -36.8 ft. R1: Hard, fresh, fine grained, gray, METAPELITE. Primary joints are close to moderately spaced, low angle to horizontal, undulating, rough, fresh to discolored, tight	

Remarks:

Auto Hammer #149
Raft deck to ground ±15.0 ft.
Started Boring at Mean Tide, spuds where not long enough at High Tide.
GZA reviewed and edited draft logs provided by MaineDOT. Soil descriptions were edited as needed to make visual descriptions consistent with laboratory test data; GZA did not visually observe collected soil samples. Rock RQD and descriptions were edited based on a visit by a GZA engineer to MaineDOT's laboratory in Bangor on April 17,

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wadsworth Street Bridge #2904 carries Wadsworth Street over St. George River
Location: Thomaston, Maine

Boring No.: BB-TSGR-203
PIN: 16755.00

Driller:	Northern Test Boring	**Elevation (ft.)**	-13.9	**Auger ID/OD:**	N/A
Operator:	Mike/Nick	**Datum:**	NAVD88	**Sampler:**	Standard Split
Logged By:	B. Wilder	**Rig Type:**	Diedrick D-50 on Raft	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/30/11; 06:00-11:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	26+28.9, 0.36 ft Rt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	Tidal Water
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
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N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25								-41.8		to partially open. Secondary joints are moderately spaced, moderately dipping, planar, rough, fresh, tight, occasional calite stringers throughout. Rock Mass Quality = Fair. R1:Core Times (min:sec) 22.9-23.9 ft (3:50) 23.9-24.9 ft (5:00) 24.9-25.9 ft (5:30) 25.9-26.9 ft (5:00) 26.9-27.9 ft (5:30) 82% Recovery Attempted R2, Core Barrel plugged up, ran out of time with the tide.	
30										27.9 **Bottom of Exploration at 27.90 feet below ground surface.**	
35											
40											
45											
50											

Remarks:

Auto Hammer #149
Raft deck to ground ±15.0 ft.
Started Boring at Mean Tide, spuds where not long enough at High Tide.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wadsworth Street Bridge #2904 carries Wadsworth Street over St. George River
Location: Thomaston, Maine

Boring No.: BB-TSGR-204
PIN: 16755.00

Driller:	Northern Test Boring	**Elevation (ft.)**	-14.3	**Auger ID/OD:**	N/A
Operator:	Mike/Nick	**Datum:**	NAVD88	**Sampler:**	Standard Split
Logged By:	B. Wilder	**Rig Type:**	Diedrick D-50 on Raft	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/3/11; 07:00-12:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	27+00.9, 3.9 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	Tidal Water
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
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N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	MD	24/0	0.0 - 2.0	1/1/1/1	2	3	[a]HP			Black sandy MUCK in wash. [a]Hydraulic Push	
							13				
							13	-16.3		2.0 Grey, wet, fine to medium SAND, some gravel in wash.	
							14				
							16				
5							5				
							7				
							WOC				
							WOC				
	MD	24/0	9.0 - 11.0	4/1/2/2	3	4	26				
10							37				
	1D	24/10	11.0 - 13.0	8/16/15/8	31	43	92			Grey, wet, dense, fine to coarse SAND, trace silt, trace gravel, shells, (Marine). Roller Coned ahead to 20.0 ft bgs.	G#261839 A-1-b, SP WC=28.3%
							OPEN HOLE				
15											
										0.3-0.5 ft Cobbles from 18.0-19.4 ft bgs.	
20										Cobble from 20.0-20.6 ft bgs. Roller Coned ahead to 27.2 ft bgs.	
25											

Remarks:

Auto Hammer #149
Raft deck to ground ±15.0 ft.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wadsworth Street Bridge #2904 carries Wadsworth Street over St. George River
Location: Thomaston, Maine

Boring No.: BB-TSGR-204
PIN: 16755.00

Driller:	Northern Test Boring	**Elevation (ft.)**	-14.3	**Auger ID/OD:**	N/A
Operator:	Mike/Nick	**Datum:**	NAVD88	**Sampler:**	Standard Split
Logged By:	B. Wilder	**Rig Type:**	Diedrick D-50 on Raft	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/3/11; 07:00-12:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	27+00.9, 3.9 ft Rt.	**Casing ID/OD:**	HW	**Water Level*:**	Tidal Water
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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WOR = weight of rods
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S_u = Insitu Field Vane Shear Strength (psf)
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LL = Liquid Limit
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C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25											
	R1	50.4/50.4	27.2 - 31.4	RQD = 76%			NQ-2	-41.5		27.2 Top of Bedrock at Elev. -41.5 ft. R1: Top 12" gray, fine grained, METAPELITE. Bottom-very hard, fresh, fine grained, white/gray, METASANDSTONE. Joints are close to moderately spaced, low angle, undulating, rough, fresh, tight to partially open. Rock Mass Quality = Good. R1:Core Times (min:sec) 27.2-28.2 ft (3:00) 28.2-29.2 ft (3:30) 29.2-30.2 ft (3:45) 30.2-31.2 ft (4:00) 31.2-31.4 ft (2:00) 100% Recovery Core Blocked	
30	R2	55.2/53	31.4 - 36.0	RQD = 83%						R2: Very hard, fresh, fine grained, white and gray, METASANDSTONE. Joints are moderately spaced, low angle to moderately dipping, undulating, rough, fresh, partially open. Rock Mass Quality = Good. R2:Core Times (min:sec) 31.4-32.4 ft (3:30) 32.4-33.4 ft (4:00) 33.4-34.4 ft (4:15) 34.4-35.4 ft (4:20) 35.4-36.0 ft (5:00) 96% Recovery	
35								-50.3		36.0 **Bottom of Exploration at 36.00 feet below ground surface.**	
40											
45											
50											

Remarks:

Auto Hammer #149
Raft deck to ground ±15.0 ft.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wadsworth Street Bridge #2904 carries Wadsworth Street over St. George River
Location: Thomaston, Maine

Boring No.: BB-TSGR-205
PIN: 16755.00

Driller:	Northern Test Boring	**Elevation (ft.)**	-4.2	**Auger ID/OD:**	N/A
Operator:	Mike/Nick	**Datum:**	NAVD88	**Sampler:**	Standard Split
Logged By:	B. Wilder	**Rig Type:**	Diedrick D-50 on Raft	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/29/11; 09:00-14:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	27+70.1, 13.3 ft Lt.	**Casing ID/OD:**	HW	**Water Level*:**	Tidal Water
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/13	0.0 - 2.0	WOH/WOH/WOH/ WOH	---		[a]HP		Black, saturated, organic SILT, some sand, (Marine Muck). [a]Hydraulic Push	G#261764 A-4, OL WC=75.6%
								-6.7	2.5	
5										
							8			
	2D	24/14	8.0 - 10.0	WOH/WOH/1/1	1	1	15		Grey, wet, very soft, sandy SILT, trace clay, (Marine).	G#261765 A-4, CL-ML WC=36.8%
							15			
10							17			
							15			
							16			
							24	-17.2	13.0	
							36			
15	3D	24/24	15.0 - 17.0	1/1/2/2	3	4	36		Grey-brown, soft, SILT, some sand, (Marine).	G#261766 A-4, ML WC=35.5%
							29			
							26	-22.1	Cobble from 17.6-17.9 ft bgs. 17.9	
							85			
20	4D	21.6/6	19.5 - 21.3	14/14/20/50(3.6)	34	48	[b]70		[b]70 blows for 0.5 ft. Grey, wet, dense, sandy SILT, little gravel, (Till).	G#261767 A-4, CL-ML WC=13.3%
								-25.5	21.3	
	R1	155/18	21.3 - 34.2	RQD = 0%			NQ-2		R1: Apparent cobbles and boulders from approximately 21.3-33.0'; no recovery over this interval. R1:Core Times (min:sec) 21.3-22.3 ft (1:00) 22.3-23.3 ft (0:45) 23.3-24.3 ft (1:00) 24.3-25.3 ft (0:40)	
25										

Remarks:

Auto Hammer #149
Raft deck to ground ±15.0 ft.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wadsworth Street Bridge #2904 carries Wadsworth Street over St. George River
Location: Thomaston, Maine

Boring No.: BB-TSGR-205
PIN: 16755.00

Driller:	Northern Test Boring	**Elevation (ft.)**	-4.2	**Auger ID/OD:**	N/A
Operator:	Mike/Nick	**Datum:**	NAVD88	**Sampler:**	Standard Split
Logged By:	B. Wilder	**Rig Type:**	Diedrick D-50 on Raft	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	9/29/11; 09:00-14:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	27+70.1, 13.3 ft Lt.	**Casing ID/OD:**	HW	**Water Level*:**	Tidal Water
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										25.3-26.3 ft (0:30) 26.3-27.3 ft (0:40) 27.3-28.3 ft (0:15) 28.3-29.3 ft (0:30) 29.3-30.3 ft (0:30) 30.3-31.3 ft (1:00) 31.3-32.3 ft (2:00) 32.3-33.3 ft (4:00)	
30											
								-37.2		33.0 Top of Bedrock at Elev. -37.2 ft. R1 Continued: Gray, METAPELITE. Rock Mass Quality = Very Poor. 33.3-34.2 ft (3:00) Core Blocked	
	R2	60/60	34.2 - 39.2	RQD = 27%						R2: Hard, fresh, fine grained, gray, METAPELITE. Primary joints are close, low angle, undulating, rough, fresh, partially open, silt seam at 34.9'. Secondary joints are close, high angle, undulating, rough, fresh, tight, calcite stringers throughout. Rock Mass Quality = Poor. R2:Core Times (min:sec) 34.2-35.2 ft (5:00) 35.2-36.2 ft (4:20) 36.2-37.2 ft (4:45) 37.2-38.2 ft (5:00) 38.2-39.2 ft (5:00) 100% Recovery Could not get back down and ran out of time with the tide.	
35											
								-43.4		39.2 **Bottom of Exploration at 39.20 feet below ground surface.**	
40											
45											
50											

Remarks:

Auto Hammer #149
Raft deck to ground ±15.0 ft.
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APPENDIX C

DESIGN PHASE TEST BORING LOGS

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wadsworth Street Bridge
Location: Thomaston, Maine

Boring No.: BB-TSGR-301
PIN: 16755

Driller:	Maine Test Boring	**Elevation (ft.)**	-14.1	**Auger ID/OD:**	
Operator:	Mike Porter	**Datum:**	NAVD88	**Sampler:**	Standard SS
Logged By:	Evan Lonstein (GZA)	**Rig Type:**	CME 45 on Barge	**Hammer Wt./Fall:**	140 lb/30"
Date Start/Finish:	04/16/13	**Drilling Method:**	HW/NW Drive	**Core Barrel:**	NQ2
Boring Location:	N 208416.8 E 1623777.1	**Casing ID/OD:**	4"/4.5", 3"/3.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
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N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N'_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							5			No sampling.	
							14			-POSSIBLE RIVER BOTTOM DEPOSIT-	
								-16.1		2.0	
	1D	24/1	2.0 - 4.0	7-27-30-26	57	57	25			Gray, wet, very dense, GRAVEL. Drill spoils consist of gray Sand and Gravel. Roller bit action indicates significant gravel content.	
							70			Gray-brown SILT in wash water at 4.0-4.5 feet.	
							125				
5	2D	24/12	5.0 - 7.0	73-87-38-54	125	125	RC	-19.1		-POSSIBLE TILL-	
										5.0	
										Black, wet, very dense, Silty GRAVEL, little Sand.	
										-WEATHERED ROCK-	
										Advanced roller cone to 8.8', telescoped NW casing, and cleaned out to 9.2' with roller cone.	
	R1	60/60	9.2 - 14.2	RQD = 70%			NQ2	-23.3		9.2	
10										Hard, fresh, fine-grained, gray METAPELITE. Primary joints are close to moderately spaced, low angle to moderately dipping, stepped, rough, fresh, tight to partially open. Secondary joints are moderately spaced, near horizontal, undulating, rough, fresh to discolored, tight. Rock Mass Quality = Fair	qp = 3,091 ksf
								-28.3		R1: Core Times (min/ft): 3.5, 3.0, 2.5, 3.0, 3.0 100% Recovery.	
15										14.2	
										Bottom of Exploration at 14.20 feet below ground surface.	
20											
25											
30											

Remarks:

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wadsworth Street Bridge
Location: Thomaston, Maine

Boring No.: BB-TSGR-302
PIN: 16755

Driller:	Maine Test Boring	**Elevation (ft.)**	-18.9	**Auger ID/OD:**	
Operator:	Mike Porter	**Datum:**	NAVD88	**Sampler:**	Standard SS
Logged By:	Evan Lonstein (GZA)	**Rig Type:**	CME 45 on Barge	**Hammer Wt./Fall:**	140 lb/30"
Date Start/Finish:	04/17/13	**Drilling Method:**	HW/NW Drive	**Core Barrel:**	NQ2
Boring Location:	N 208474.2 E 1623814.0	**Casing ID/OD:**	4"/4.5", 3"/3.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							5			Drill cuttings include gray to olive Sand, Gravel, and Silt, with wood from 0' to 5.0'.	
							3				
							27				
							2				
							6				
5	1D	24/6	5.0 - 7.0	6-4-3-2	7	7	16			Gray, wet, loose, fine to coarse SAND, some Gravel, some Silt, shell fragments.	
							6				
							5				
							6			-RIVER BOTTOM DEPOSIT-	
							5				
10	2D	24/1	10.5 - 12.5	7-3-4-4	7	7	3			Dark gray, wet, loose, fine to coarse SAND/GRAVEL. Insufficient recovery for classification. Drill cuttings include wood from 10' - 13'.	
							5				
							5				
							21	-32.7		13.8	
	3D	24/21	14.6 - 16.6	50-69-67-63	136	136	92			Increased drilling resistance at 13.8'. Gray-olive, wet, very dense, fine to medium SAND, some Gravel, little Silt.	
15							5				
							6			-TILL-	
							11				
							30			Gray-olive, wet, very dense, fine to medium SAND, some Gravel, little Silt.	
							47				
20	4D	5/5	20.4 - 20.8	100/5"	R		RC	-39.3		20.4	
	R1	61/61	21.4 - 26.5	RQD = 98%			NQ2	-40.3		Refusal at 20.4'. Telescoped NW casing to 20.4', advanced roller cone to 21.4', and began coring. 21.4	
										Hard, fine-grained, fresh, gray, METAPELITE. Joints are very close to wide, low angle, stepped, rough, fresh, tight to open. Rock Mass Quality = Excellent R1 Core Times (min/ft): 3.0, 2.5, 3.0, 3.0, 2.5 100% Recovery.	
25											
								-45.4		26.5	
										Bottom of Exploration at 26.50 feet below ground surface.	
30											

Remarks:

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wadsworth Street Bridge
Location: Thomaston, Maine

Boring No.: BB-TSGR-303
PIN: 16755

Driller:	Maine Test Boring	**Elevation (ft.)**	-14.8	**Auger ID/OD:**	
Operator:	Mike Porter	**Datum:**	NAVD88	**Sampler:**	Standard SS
Logged By:	Evan Lonstein (GZA)	**Rig Type:**	CME 45 on Barge	**Hammer Wt./Fall:**	140 lb/30"
Date Start/Finish:	04/18/13	**Drilling Method:**	HW/NW Drive	**Core Barrel:**	NQ2
Boring Location:	N 208544.0 E 1623827.3	**Casing ID/OD:**	4"/4.5", 3"/3.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/0	0.0 - 2.0	WOH-WOH-1-5	1	1	3		No recovery. Drill cuttings include brown to gray Sand, Gravel, and Silt from 0'-5'. -POSSIBLE RIVER BOTTOM DEPOSIT-	
							3			
							5			
							6			
							7			
5	2D	24/17	5.0 - 7.0	8-1-WOH-1			7	-19.8	5.0 — Dark gray, wet, very soft, organic SILT, little fine Sand, organic odor. -SILT / CLAY- Roller bit bouncing occasionally, cuttings include wood.	
							8			
							7			
							7			
							6			
10	3D	24/4	10.0 - 12.0	2-1-1-1	2	2	5		Dark gray to brown, wet, very soft to soft, organic SILT, some fine to coarse Sand, little Gravel, organic odor, some wood fragments, possible Peat.	
							WOH			
							3			
							3	-27.8	13.0	
							3			
15	4D	24/6	15.3 - 17.3	1-1-3-3	4	4	6		Dark gray, wet, loose, fine to coarse SAND, little Silt, occasional wood pieces. -RIVER BOTTOM DEPOSIT-	
							7			
							16			
	R1	60/14	18.4 - 23.4	RQD = 0%			115/" NQ2	-33.2	18.4 — Drill action indicates cobbles and boulders at 18.4', advance core barrel. Recover indicates rock pieces with lengths less than 4.0", and gray Sand. R1: Core Times (min/ft): 3.0, 1.0, 0.5, 1.0, 1.0 -COBBLES AND BOULDERS/POSSIBLE TILL-	
20										
							140			
							65 /0.2			
25	5D	2/2	25.0 - 25.2	50/2"	R		RC	-39.8	25.0 — Gray, wet, dense, fine to coarse SAND, some Silt, trace Gravel. -TILL-	
								-41.3	Advanced roller cone to 26.5', telescoped NW casing, cleaned out with roller cone to 27' and began coring.	
	R2	62/62	27.0 - 32.2	RQD = 45%					26.5 — Hard, fresh, fine-grained, white/green/gray METASANDSTONE. Primary joints are very close to	
30										qp = 2,001 ksf

Remarks:

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Wadsworth Street Bridge
Location: Thomaston, Maine

Boring No.: BB-TSGR-303
PIN: 16755

Driller:	Maine Test Boring	**Elevation (ft.)**	-14.8	**Auger ID/OD:**	
Operator:	Mike Porter	**Datum:**	NAVD88	**Sampler:**	Standard SS
Logged By:	Evan Lonstein (GZA)	**Rig Type:**	CME 45 on Barge	**Hammer Wt./Fall:**	140 lb/30"
Date Start/Finish:	04/18/13	**Drilling Method:**	HW/NW Drive	**Core Barrel:**	NQ2
Boring Location:	N 208544.0 E 1623827.3	**Casing ID/OD:**	4"/4.5", 3"/3.5"	**Water Level*:**	Tidal
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR = weight of rods
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
30								-47.0		moderately spaced, low angle, undulating, rough, fresh to discolored, partially open. Secondary joints are very close to moderately spaced, moderately dipping, undulating, rough, fresh to discolored, partially open. Rock Mass Quality = Poor R2: Rock Core Times (min/ft): 3.0, 2.5, 2.0, 2.0, 2.5 100% Recovery.	
35										32.2 **Bottom of Exploration at 32.20 feet below ground surface.**	
40											
45											
50											
55											
60											

Remarks:

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

APPENDIX D

LABORATORY TEST RESULTS

Maine DOT, Materials Testing & Exploration, 219 Hogan Road, Bangor, Maine 04401

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
240060	BB-TSGR-101/1D	GEOTECHNICAL (DISTURBED)	11/2/2010	11/30/2010

Sample Type: **GEOTECHNICAL** Location: Station: **24+99.9** Offset, ft: **38.5** **RT** Dbfg, ft: **2.0-4.0**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	**100.0**
¾ in. [19.0 mm]	**91.8**
½ in. [12.5 mm]	**85.4**
⅜ in. [9.5 mm]	**81.0**
¼ in. [6.3 mm]	**76.7**
No. 4 [4.75 mm]	**73.0**
No. 10 [2.00 mm]	**64.0**
No. 20 [0.850 mm]	**54.4**
No. 40 [0.425 mm]	**45.0**
No. 60 [0.250 mm]	**37.6**
No. 100 [0.150 mm]	**31.3**
No. 200 [0.075 mm]	**23.3**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %

Plastic Limit (T 90), %

Plasticity Index (T 90), %

Specific Gravity, Corrected to 20°C (T 100)

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %

5.7

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **12/22/2010**

Paper Copy: Lab File; Project File; Geotech File

Maine DOT, Materials Testing & Exploration, 219 Hogan Road, Bangor, Maine 04401

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
240061	**BB-TSGR-101/2D**	**GEOTECHNICAL (DISTURBED)**	**11/1/2010**	**11/30/2010**

Sample Type: **GEOTECHNICAL** Location: Station: **24+99.9** Offset, ft: **38.5** **RT** Dbfg, ft: **5.0-7.0**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	**100.0**
½ in. [12.5 mm]	**96.6**
⅜ in. [9.5 mm]	**95.5**
¼ in. [6.3 mm]	**94.4**
No. 4 [4.75 mm]	**92.9**
No. 10 [2.00 mm]	**88.4**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**76.3**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**49.1**
[0.0313 mm]	**31.5**
[0.0207 mm]	**25.2**
[0.0126 mm]	**18.9**
[0.0091 mm]	**15.7**
[0.0063 mm]	**12.6**
[0.0032 mm]	**9.5**
[0.0014 mm]	**6.3**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %

Plastic Limit (T 90), %

Plasticity Index (T 90), %

Specific Gravity, Corrected to 20°C (T 100): **2.72**

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %: **12.5**

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **12/22/2010**

Paper Copy: Lab File; Project File; Geotech File

Maine DOT, Materials Testing & Exploration, 219 Hogan Road, Bangor, Maine 04401

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
240062	BB-TSGR-101/3D	GEOTECHNICAL (DISTURBED)	11/2/2010	11/30/2010

Sample Type: **GEOTECHNICAL** Location: Station: **24+99.9** Offset, ft: **38.5 RT** Dbfg, ft: **10.0-12.0**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	**100.0**
¾ in. [19.0 mm]	**96.5**
½ in. [12.5 mm]	**94.4**
⅜ in. [9.5 mm]	**93.5**
¼ in. [6.3 mm]	**91.2**
No. 4 [4.75 mm]	**89.9**
No. 10 [2.00 mm]	**85.1**
No. 20 [0.850 mm]	**79.7**
No. 40 [0.425 mm]	**74.5**
No. 60 [0.250 mm]	**70.6**
No. 100 [0.150 mm]	**66.4**
No. 200 [0.075 mm]	**58.9**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %
Plastic Limit (T 90), %
Plasticity Index (T 90), %
Specific Gravity, Corrected to 20°C (T 100)

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %

23.5

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **12/22/2010**

Paper Copy: Lab File; Project File; Geotech File

Maine DOT, Materials Testing & Exploration, 219 Hogan Road, Bangor, Maine 04401

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
240063	**BB-TSGR-101/4D**	**GEOTECHNICAL (DISTURBED)**	**11/1/2010**	**11/30/2010**

Sample Type: **GEOTECHNICAL** Location: Station: **24+99.9** Offset, ft: **38.5 RT** Dbfg, ft: **15.0-17.0**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	**100.0**
⅜ in. [9.5 mm]	**97.0**
¼ in. [6.3 mm]	**94.7**
No. 4 [4.75 mm]	**93.0**
No. 10 [2.00 mm]	**88.8**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**74.1**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**52.9**
[0.0284 mm]	**33.7**
[0.0193 mm]	**26.6**
[0.0113 mm]	**25.2**
[0.0086 mm]	**16.9**
[0.0062 mm]	**14.0**
[0.0032 mm]	**8.4**
[0.0013 mm]	**5.6**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %

Plastic Limit (T 90), %

Plasticity Index (T 90), %

Specific Gravity, Corrected to 20°C (T 100)

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %

13.5

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **1/7/2011**

Paper Copy: Lab File; Project File; Geotech File

Maine DOT, Materials Testing & Exploration, 219 Hogan Road, Bangor, Maine 04401

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
240064	**BB-TSGR-101/5D**	**GEOTECHNICAL (DISTURBED)**	**11/1/2010**	**11/30/2010**

Sample Type: **GEOTECHNICAL** Location: Station: **24+99.9** Offset, ft: **38.5** **RT** Dbfg, ft: **20.0-22.0**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	**100.0**
¾ in. [19.0 mm]	**81.1**
½ in. [12.5 mm]	**76.0**
⅜ in. [9.5 mm]	**74.5**
¼ in. [6.3 mm]	**73.0**
No. 4 [4.75 mm]	**71.7**
No. 10 [2.00 mm]	**68.8**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**51.7**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**28.1**
[0.0303 mm]	**18.3**
[0.0198 mm]	**15.1**
[0.0119 mm]	**12.9**
[0.0086 mm]	**10.8**
[0.0062 mm]	**8.6**
[0.0031 mm]	**6.5**
[0.0013 mm]	**4.3**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %

Plastic Limit (T 90), %

Plasticity Index (T 90), %

Specific Gravity, Corrected to 20°C (T 100): **2.77**

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %: **10.5**

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **12/22/2010**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
240065	**BB-TSGR-101/6D**	**GEOTECHNICAL (DISTURBED)**	**11/1/2010**	**11/30/2010**

Sample Type: **GEOTECHNICAL** Location: Station: **24+99.9** Offset, ft: **38.5 RT** Dbfg, ft: **25.0-27.0**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	**100.0**
¾ in. [19.0 mm]	**97.4**
½ in. [12.5 mm]	**93.6**
⅜ in. [9.5 mm]	**91.2**
¼ in. [6.3 mm]	**88.4**
No. 4 [4.75 mm]	**86.8**
No. 10 [2.00 mm]	**80.5**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**68.1**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**44.6**
[0.0303 mm]	**28.2**
[0.0200 mm]	**24.3**
[0.0118 mm]	**18.0**
[0.0089 mm]	**15.4**
[0.0063 mm]	**10.2**
[0.0032 mm]	**6.4**
[0.0014 mm]	**5.2**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %

Plastic Limit (T 90), %

Plasticity Index (T 90), %

Specific Gravity, Corrected to 20°C (T 100): **2.69**

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %: **11.0**

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **12/22/2010**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
240066	**BB-TSGR-102/2D**	**GEOTECHNICAL (DISTURBED)**	**11/2/2010**	**11/30/2010**

Sample Type: **GEOTECHNICAL** Location: Station: **26+33.6** Offset, ft: **42.6** **RT** Dbfg, ft: **7.0-9.0**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	**100.0**
½ in. [12.5 mm]	**94.4**
⅜ in. [9.5 mm]	**87.1**
¼ in. [6.3 mm]	**78.6**
No. 4 [4.75 mm]	**73.7**
No. 10 [2.00 mm]	**60.6**
No. 20 [0.850 mm]	**46.8**
No. 40 [0.425 mm]	**31.4**
No. 60 [0.250 mm]	**20.0**
No. 100 [0.150 mm]	**13.8**
No. 200 [0.075 mm]	**9.8**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %

Plastic Limit (T 90), %

Plasticity Index (T 90), %

Specific Gravity, Corrected to 20°C (T 100)

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %

15.3

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **12/17/2010**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
240067	BB-TSGR-102/3D	GEOTECHNICAL (DISTURBED)	11/2/2010	11/30/2010

Sample Type: **GEOTECHNICAL** Location: Station: **26+33.6** Offset, ft: **42.6 RT** Dbfg, ft: **12.0-14.0**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	100.0
½ in. [12.5 mm]	94.6
⅜ in. [9.5 mm]	88.9
¼ in. [6.3 mm]	83.5
No. 4 [4.75 mm]	81.0
No. 10 [2.00 mm]	70.7
No. 20 [0.850 mm]	54.9
No. 40 [0.425 mm]	31.0
No. 60 [0.250 mm]	21.7
No. 100 [0.150 mm]	15.7
No. 200 [0.075 mm]	10.4

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %

Plastic Limit (T 90), %

Plasticity Index (T 90), %

Specific Gravity, Corrected to 20°C (T 100)

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %

25.3

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **12/17/2010**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
240068	**BB-TSGR-102/4D**	**GEOTECHNICAL (DISTURBED)**	**11/1/2010**	**11/30/2010**

Sample Type: **GEOTECHNICAL** Location: Station: **26+33.6** Offset, ft: **42.6** **RT** Dbfg, ft: **22.0-23.8**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	**100.0**
¾ in. [19.0 mm]	**97.6**
½ in. [12.5 mm]	**94.7**
⅜ in. [9.5 mm]	**90.8**
¼ in. [6.3 mm]	**86.8**
No. 4 [4.75 mm]	**85.9**
No. 10 [2.00 mm]	**81.9**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**74.1**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**51.7**
[0.0284 mm]	**31.6**
[0.0193 mm]	**25.0**
[0.0120 mm]	**17.1**
[0.0086 mm]	**14.5**
[0.0064 mm]	**10.6**
[0.0032 mm]	**7.9**
[0.0014 mm]	**5.2**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %

Plastic Limit (T 90), %

Plasticity Index (T 90), %

Specific Gravity, Corrected to 20°C (T 100): **2.65**

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %: **12.7**

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

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Reported by: **FOGG, BRIAN** Date Reported: **12/22/2010**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
240075	**BB-TSGR-103/10D**	**GEOTECHNICAL (DISTURBED)**	**11/1/2010**	**11/30/2010**

Sample Type: **GEOTECHNICAL** Location: Station: **27+54.3** Offset, ft: **43.0** **RT** Dbfg, ft: **45.5-47.5**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	**100.0**
¾ in. [19.0 mm]	**97.1**
½ in. [12.5 mm]	**96.6**
⅜ in. [9.5 mm]	**94.8**
¼ in. [6.3 mm]	**92.8**
No. 4 [4.75 mm]	**91.5**
No. 10 [2.00 mm]	**87.3**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**78.4**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**51.9**
[0.0292 mm]	**29.8**
[0.0200 mm]	**23.0**
[0.0121 mm]	**16.2**
[0.0090 mm]	**10.8**
[0.0065 mm]	**9.5**
[0.0033 mm]	**5.4**
[0.0014 mm]	**4.1**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %

Plastic Limit (T 90), %

Plasticity Index (T 90), %

Specific Gravity, Corrected to 20°C (T 100)

2.67

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %

13.0

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **12/22/2010**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
245451	BB-TSGR-103/11D	GEOTECHNICAL (DISTURBED)	11/1/2010	11/30/2010

Sample Type: **GEOTECHNICAL** Location: Station: **27+54.3** Offset, ft: **43.0 RT** Dbfg, ft: **50.0-50.5**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	**100.0**
¾ in. [19.0 mm]	**96.9**
½ in. [12.5 mm]	**88.4**
⅜ in. [9.5 mm]	**84.2**
¼ in. [6.3 mm]	**80.9**
No. 4 [4.75 mm]	**78.7**
No. 10 [2.00 mm]	**73.8**
No. 20 [0.850 mm]	**69.7**
No. 40 [0.425 mm]	**66.3**
No. 60 [0.250 mm]	**63.2**
No. 100 [0.150 mm]	**54.6**
No. 200 [0.075 mm]	**30.0**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %

Plastic Limit (T 90), %

Plasticity Index (T 90), %

Specific Gravity, Corrected to 20°C (T 100)

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %

12.6

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **12/17/2010**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
240069	**BB-TSGR-103/1D**	**GEOTECHNICAL (DISTURBED)**	**11/1/2010**	**11/30/2010**

Sample Type: **GEOTECHNICAL** Location: Station: **27+54.3** Offset, ft: **43.0** **RT** Dbfg, ft: **2.0-4.0**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	**100.0**
1 in. [25.0 mm]	**100.0**
¾ in. [19.0 mm]	**90.5**
½ in. [12.5 mm]	**85.6**
⅜ in. [9.5 mm]	**82.3**
¼ in. [6.3 mm]	**76.2**
No. 4 [4.75 mm]	**73.1**
No. 10 [2.00 mm]	**63.1**
No. 20 [0.850 mm]	**52.7**
No. 40 [0.425 mm]	**43.8**
No. 60 [0.250 mm]	**38.6**
No. 100 [0.150 mm]	**33.6**
No. 200 [0.075 mm]	**26.2**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %
Plastic Limit (T 90), %
Plasticity Index (T 90), %
Specific Gravity, Corrected to 20°C (T 100)

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %

8.0

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **12/17/2010**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
240070	**BB-TSGR-103/2D**	**GEOTECHNICAL (DISTURBED)**	**11/1/2010**	**11/30/2010**

Sample Type: **GEOTECHNICAL** Location: Station: **27+54.3** Offset, ft: **43.0** **RT** Dbfg, ft: **5.0-7.0**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	**100.0**
¾ in. [19.0 mm]	**95.0**
½ in. [12.5 mm]	**88.4**
⅜ in. [9.5 mm]	**87.1**
¼ in. [6.3 mm]	**83.4**
No. 4 [4.75 mm]	**81.3**
No. 10 [2.00 mm]	**76.1**
No. 20 [0.850 mm]	**70.7**
No. 40 [0.425 mm]	**65.1**
No. 60 [0.250 mm]	**60.8**
No. 100 [0.150 mm]	**57.3**
No. 200 [0.075 mm]	**57.1**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %	

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %
Plastic Limit (T 90), %
Plasticity Index (T 90), %
Specific Gravity, Corrected to 20°C (T 100)

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %

12.0

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **12/22/2010**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
240071	**BB-TSGR-103/3D**	**GEOTECHNICAL (DISTURBED)**	**11/1/2010**	**11/30/2010**

Sample Type: **GEOTECHNICAL** Location: Station: **27+54.3** Offset, ft: **43.0** **RT** Dbfg, ft: **10.0-12.0**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	**100.0**
⅜ in. [9.5 mm]	**94.9**
¼ in. [6.3 mm]	**86.1**
No. 4 [4.75 mm]	**81.4**
No. 10 [2.00 mm]	**66.7**
No. 20 [0.850 mm]	**46.4**
No. 40 [0.425 mm]	**31.9**
No. 60 [0.250 mm]	**26.5**
No. 100 [0.150 mm]	**22.0**
No. 200 [0.075 mm]	**15.3**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %	

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %
Plastic Limit (T 90), %
Plasticity Index (T 90), %
Specific Gravity, Corrected to 20°C (T 100)

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %

13.4

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **12/17/2010**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
240072	**BB-TSGR-103/5D**	**GEOTECHNICAL (DISTURBED)**	**11/1/2010**	**11/30/2010**

Sample Type: **GEOTECHNICAL** Location: Station: **27+54.3** Offset, ft: **43.0** **RT** Dbfg, ft: **20.0-22.0**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	**100.0**
½ in. [12.5 mm]	**95.3**
⅜ in. [9.5 mm]	**94.0**
¼ in. [6.3 mm]	**92.2**
No. 4 [4.75 mm]	**90.3**
No. 10 [2.00 mm]	**85.9**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**73.4**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**46.4**
[0.0297 mm]	**28.8**
[0.0197 mm]	**23.3**
[0.0118 mm]	**19.2**
[0.0086 mm]	**15.0**
[0.0063 mm]	**13.7**
[0.0032 mm]	**9.6**
[0.0014 mm]	**5.5**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %

Plastic Limit (T 90), %

Plasticity Index (T 90), %

Specific Gravity, Corrected to 20°C (T 100): **2.67**

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %: **13.9**

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **12/22/2010**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
240073	**BB-TSGR-103/7D**	**GEOTECHNICAL (DISTURBED)**	**11/1/2010**	**11/30/2010**

Sample Type: **GEOTECHNICAL** Location: Station: **27+54.3** Offset, ft: **43.0** **RT** Dbfg, ft: **30.0-32.0**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	**100.0**
No. 10 [2.00 mm]	**99.9**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**99.2**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**65.4**
[0.0301 mm]	**47.1**
[0.0200 mm]	**37.7**
[0.0118 mm]	**33.0**
[0.0086 mm]	**25.9**
[0.0063 mm]	**23.6**
[0.0031 mm]	**18.9**
[0.0014 mm]	**9.4**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %	

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %
28
Plastic Limit (T 90), %
29
Plasticity Index (T 90), %
NP
Specific Gravity, Corrected to 20°C (T 100)
2.64

Loss on Ignition (T 267)	
Loss, %	H2O, %

Water Content (T 265), %
36.2

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

non plastic

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **12/22/2010**

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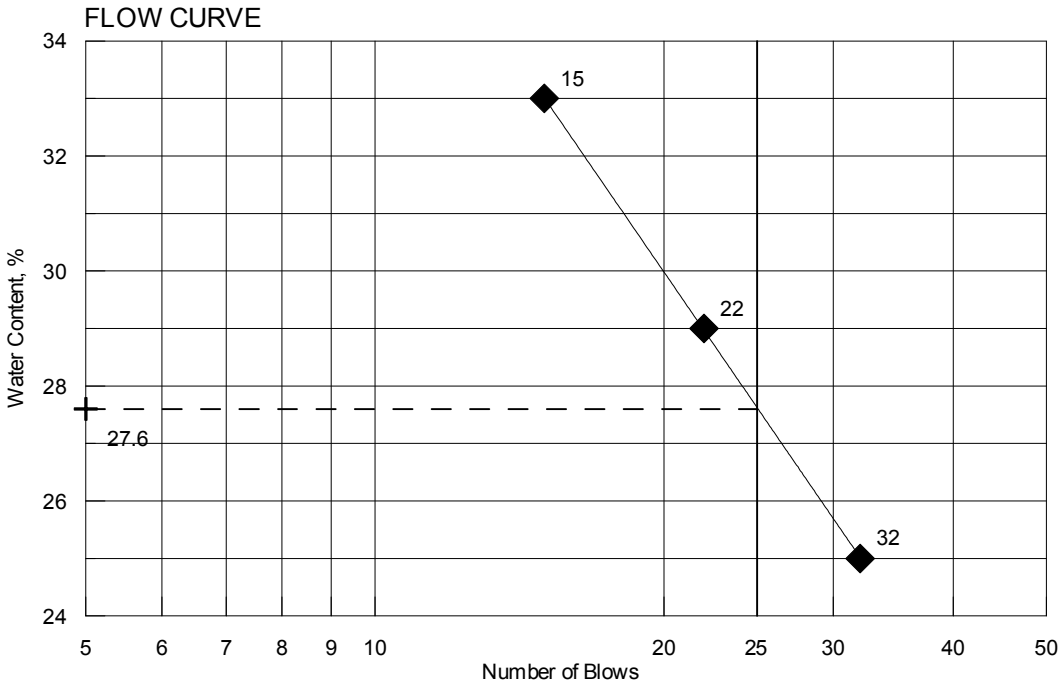
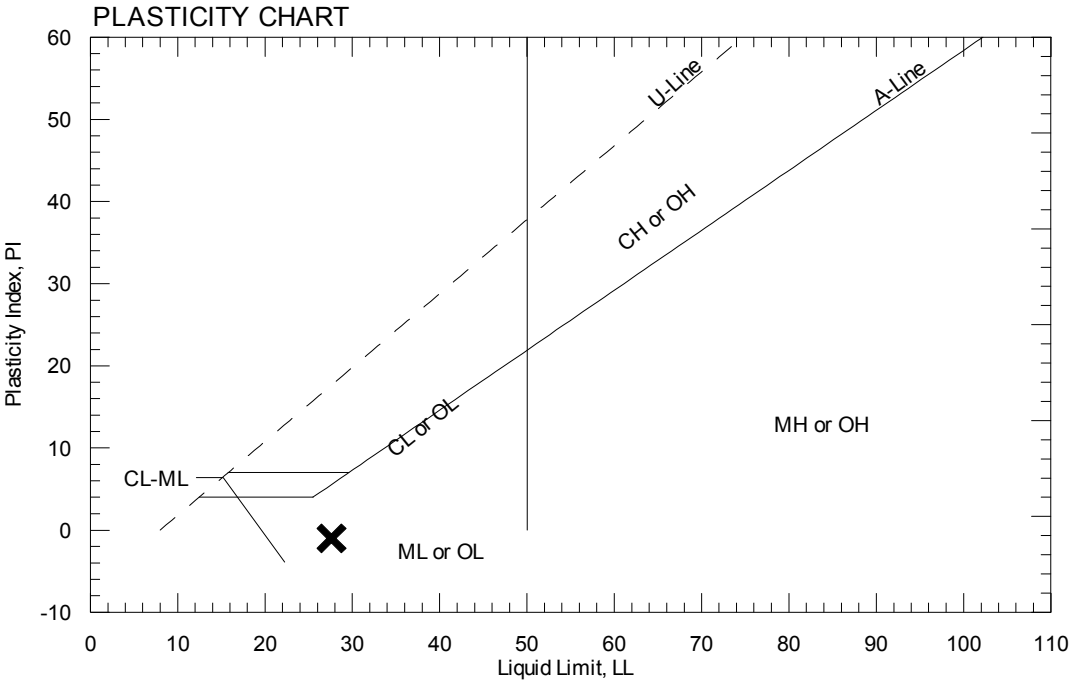
TOWN	Thomaston	Reference No.	240073
WIN	016755.00	Water Content, %	36.2
Sampled	11/1/2010	Plastic Limit	29
Boring No./Sample No.	BB-TSGR-103/7D	Liquid Limit	28
Station	27+54.3	Plasticity Index	NP
Depth	30.0-32.0	Tested By	BBURR





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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
240074	**BB-TSGR-103/9D**	**GEOTECHNICAL (DISTURBED)**	**11/1/2010**	**11/30/2010**

Sample Type: **GEOTECHNICAL** Location: Station: **27+54.3** Offset, ft: **43.0** **RT** Dbfg, ft: **40.0-42.0**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	**100.0**
¾ in. [19.0 mm]	**96.4**
½ in. [12.5 mm]	**78.0**
⅜ in. [9.5 mm]	**72.7**
¼ in. [6.3 mm]	**68.0**
No. 4 [4.75 mm]	**65.7**
No. 10 [2.00 mm]	**59.1**
No. 20 [0.850 mm]	**52.0**
No. 40 [0.425 mm]	**44.7**
No. 60 [0.250 mm]	**39.4**
No. 100 [0.150 mm]	**30.7**
No. 200 [0.075 mm]	**15.3**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %
Plastic Limit (T 90), %
Plasticity Index (T 90), %
Specific Gravity, Corrected to 20°C (T 100)

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %

29.5

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

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Reported by: **FOGG, BRIAN** Date Reported: **12/17/2010**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
261759	BB-TSGR-201/1D	GEOTECHNICAL (DISTURBED)	9/29/2011	10/6/2011

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **24+91.7** Offset, ft: **13.3 LT** Dbfg, ft: **0.0-2.0**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	100.0
¾ in. [19.0 mm]	80.9
½ in. [12.5 mm]	73.4
⅜ in. [9.5 mm]	64.6
¼ in. [6.3 mm]	55.2
No. 4 [4.75 mm]	50.4
No. 10 [2.00 mm]	39.5
No. 20 [0.850 mm]	31.4
No. 40 [0.425 mm]	24.6
No. 60 [0.250 mm]	19.9
No. 100 [0.150 mm]	15.4
No. 200 [0.075 mm]	10.0

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %

Plastic Limit (T 90), %

Plasticity Index (T 90), %

Specific Gravity, Corrected to 20°C (T 100)

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %

17.6

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **10/7/2011**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
261760	**BB-TSGR-201/2D**	**GEOTECHNICAL (DISTURBED)**	**9/27/2011**	**10/6/2011**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **24+91.7** Offset, ft: **13.3 LT** Dbfg, ft: **3.5-4.8**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	**100.0**
⅜ in. [9.5 mm]	**98.0**
¼ in. [6.3 mm]	**96.0**
No. 4 [4.75 mm]	**94.0**
No. 10 [2.00 mm]	**84.8**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**73.1**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**49.0**
[0.0321 mm]	**34.1**
[0.0122 mm]	**20.9**
[0.0088 mm]	**15.8**
[0.0062 mm]	**28.8**
[0.0063 mm]	**13.1**
[0.0031 mm]	**10.5**
[0.0013 mm]	**5.3**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %

Plastic Limit (T 90), %

Plasticity Index (T 90), %

Specific Gravity, Corrected to 20°C (T 100): **2.75**

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %: **10.1**

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **10/17/2011**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
261761	**BB-TSGR-202/1D**	**GEOTECHNICAL (DISTURBED)**	**9/27/2011**	**10/6/2011**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **25+61.5** Offset, ft: **6.2** **RT** Dbfg, ft: **0.0-2.0**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	**100.0**
1 in. [25.0 mm]	**72.9**
¾ in. [19.0 mm]	**72.9**
½ in. [12.5 mm]	**70.6**
⅜ in. [9.5 mm]	**67.9**
¼ in. [6.3 mm]	**64.4**
No. 4 [4.75 mm]	**61.8**
No. 10 [2.00 mm]	**49.0**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**30.2**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**19.0**
[0.0340 mm]	**18.5**
[0.0219 mm]	**15.4**
[0.0129 mm]	**10.8**
[0.0093 mm]	**9.2**
[0.0066 mm]	**7.7**
[0.0033 mm]	**6.2**
[0.0014 mm]	**4.6**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %

Plastic Limit (T 90), %

Plasticity Index (T 90), %

Specific Gravity, Corrected to 20°C (T 100): **2.60**

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %: **22.1**

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **10/14/2011**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
261762	**BB-TSGR-203/1D**	**GEOTECHNICAL (DISTURBED)**	**9/30/2011**	**10/6/2011**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **26+28.9** Offset, ft: **0.36** **RT** Dbfg, ft: **7.0-8.3**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	**100.0**
⅜ in. [9.5 mm]	**97.9**
¼ in. [6.3 mm]	**92.7**
No. 4 [4.75 mm]	**81.6**
No. 10 [2.00 mm]	**73.9**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**44.9**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**18.7**
[0.0349 mm]	**18.4**
[0.0220 mm]	**18.4**
[0.0130 mm]	**13.8**
[0.0092 mm]	**13.8**
[0.0065 mm]	**11.5**
[0.0032 mm]	**9.2**
[0.0014 mm]	**4.6**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %

Plastic Limit (T 90), %

Plasticity Index (T 90), %

Specific Gravity, Corrected to 20°C (T 100): **2.64**

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %: **24.7**

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **10/14/2011**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
261763	**BB-TSGR-203/2D**	**GEOTECHNICAL (DISTURBED)**	**9/30/2011**	**10/6/2011**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **26+28.9** Offset, ft: **0.36** **RT** Dbfg, ft: **20.0-21.4**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	**100.0**
¾ in. [19.0 mm]	**94.1**
½ in. [12.5 mm]	**90.9**
⅜ in. [9.5 mm]	**86.6**
¼ in. [6.3 mm]	**82.6**
No. 4 [4.75 mm]	**80.7**
No. 10 [2.00 mm]	**73.9**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**59.7**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**40.3**
[0.0329 mm]	**23.9**
[0.0207 mm]	**21.7**
[0.0122 mm]	**17.4**
[0.0089 mm]	**13.1**
[0.0063 mm]	**10.9**
[0.0031 mm]	**8.7**
[0.0013 mm]	**6.5**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %

Plastic Limit (T 90), %

Plasticity Index (T 90), %

Specific Gravity, Corrected to 20°C (T 100): **2.75**

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %: **9.1**

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **10/14/2011**

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GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
261839	**BB-TSGR-204/1D**	**GEOTECHNICAL (DISTURBED)**	**10/3/2011**	**10/19/2011**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **27+00.9** Offset, ft: **3.9** **RT** Dbfg, ft: **11.0-13.0**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 27, T 11)

Wash Method

Procedure A

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	**100.0**
¼ in. [6.3 mm]	**99.2**
No. 4 [4.75 mm]	**98.8**
No. 10 [2.00 mm]	**88.8**
No. 20 [0.850 mm]	**57.7**
No. 40 [0.425 mm]	**23.6**
No. 60 [0.250 mm]	**13.6**
No. 100 [0.150 mm]	**8.0**
No. 200 [0.075 mm]	**4.8**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %	

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %
Plastic Limit (T 90), %
Plasticity Index (T 90), %
Specific Gravity, Corrected to 20°C (T 100)

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %

28.3

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **10/27/2011**

Paper Copy: Lab File; Project File; Geotech File

Maine DOT, Materials Testing & Exploration, 219 Hogan Road, Bangor, Maine 04401

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
261764	**BB-TSGR-205/1D**	**GEOTECHNICAL (DISTURBED)**	**9/29/2011**	**10/6/2011**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **27+70.1** Offset, ft: **13.3 LT** Dbfg, ft: **0.0-2.0**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	**100.0**
¼ in. [6.3 mm]	**99.1**
No. 4 [4.75 mm]	**98.8**
No. 10 [2.00 mm]	**96.2**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**83.7**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**71.8**
[0.0297 mm]	**61.6**
[0.0192 mm]	**55.4**
[0.0116 mm]	**43.1**
[0.0085 mm]	**36.9**
[0.0061 mm]	**33.9**
[0.0031 mm]	**24.6**
[0.0013 mm]	**18.5**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %

Plastic Limit (T 90), %

Plasticity Index (T 90), %

Specific Gravity, Corrected to 20°C (T 100): **2.70**

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %: **75.6**

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **10/14/2011**

Paper Copy: Lab File; Project File; Geotech File

Maine DOT, Materials Testing & Exploration, 219 Hogan Road, Bangor, Maine 04401

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
261765	**BB-TSGR-205/2D**	**GEOTECHNICAL (DISTURBED)**	**9/29/2011**	**10/6/2011**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **27+70.1** Offset, ft: **13.3 LT** Dbfg, ft: **8.0-10.0**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	
½ in. [12.5 mm]	
⅜ in. [9.5 mm]	
¼ in. [6.3 mm]	
No. 4 [4.75 mm]	**100.0**
No. 10 [2.00 mm]	**99.8**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**98.4**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**57.9**
[0.0334 mm]	**34.9**
[0.0213 mm]	**31.7**
[0.0126 mm]	**28.5**
[0.0090 mm]	**25.3**
[0.0064 mm]	**22.2**
[0.0032 mm]	**19.1**
[0.0014 mm]	**12.7**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %

Plastic Limit (T 90), %

Plasticity Index (T 90), %

Specific Gravity, Corrected to 20°C (T 100): **2.64**

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %: **36.8**

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **10/14/2011**

Paper Copy: Lab File; Project File; Geotech File

Maine DOT, Materials Testing & Exploration, 219 Hogan Road, Bangor, Maine 04401

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
261766	**BB-TSGR-205/3D**	**GEOTECHNICAL (DISTURBED)**	**9/29/2011**	**10/6/2011**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **27+70.1** Offset, ft: **13.3 LT** Dbfg, ft: **15.0-17.0**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	**100.0**
½ in. [12.5 mm]	**99.1**
⅜ in. [9.5 mm]	**99.1**
¼ in. [6.3 mm]	**99.1**
No. 4 [4.75 mm]	**99.1**
No. 10 [2.00 mm]	**98.9**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**91.2**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**72.1**
[0.0341 mm]	**14.5**
[0.0218 mm]	**12.9**
[0.0130 mm]	**9.7**
[0.0092 mm]	**8.0**
[0.0066 mm]	**6.4**
[0.0033 mm]	**4.8**
[0.0014 mm]	**3.3**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %

Plastic Limit (T 90), %

Plasticity Index (T 90), %

Specific Gravity, Corrected to 20°C (T 100): **2.63**

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %: **35.5**

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **10/14/2011**

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Maine DOT, Materials Testing & Exploration, 219 Hogan Road, Bangor, Maine 04401

GEOTECHNICAL TEST REPORT
Central Laboratory

SAMPLE INFORMATION

Reference No.	Boring No./Sample No.	Sample Description	Sampled	Received
261767	**BB-TSGR-205/4D**	**GEOTECHNICAL (DISTURBED)**	**9/29/2011**	**10/6/2011**

Sample Type: **GEOTECHNICAL** Location: **OTHER** Station: **27+70.1** Offset, ft: **13.3 LT** Dbfg, ft: **19.5-21.3**

WIN **016755.00** Town: **Thomaston** Sampler: **WILDER, BRUCE H**

TEST RESULTS

Sieve Analysis (T 88)

Wash Method

SIEVE SIZE U.S. [SI]	% Passing
3 in. [75.0 mm]	
1 in. [25.0 mm]	
¾ in. [19.0 mm]	**100.0**
½ in. [12.5 mm]	**93.6**
⅜ in. [9.5 mm]	**89.2**
¼ in. [6.3 mm]	**82.7**
No. 4 [4.75 mm]	**81.5**
No. 10 [2.00 mm]	**75.4**
No. 20 [0.850 mm]	
No. 40 [0.425 mm]	**71.1**
No. 60 [0.250 mm]	
No. 100 [0.150 mm]	
No. 200 [0.075 mm]	**62.8**
[0.0289 mm]	**49.1**
[0.0196 mm]	**37.4**
[0.0119 mm]	**28.0**
[0.0086 mm]	**23.4**
[0.0061 mm]	**21.0**
[0.0031 mm]	**16.4**
[0.0013 mm]	**11.7**

Direct Shear (T 236)

Shear Angle, °			
Initial Water Content, %			
Normal Stress, psi			
Wet Density, lbs/ft³			
Dry Density, lbs/ft³			
Specimen Thickness, in			

Consolidation (T 216)

Trimmings, Water Content, %

	Initial	Final		Void Ratio	% Strain
Water Content, %			Pmin		
Dry Density, lbs/ft³			Pp		
Void Ratio			Pmax		
Saturation, %			Cc/C'c		

Miscellaneous Tests

Liquid Limit @ 25 blows (T 89), %

Plastic Limit (T 90), %

Plasticity Index (T 90), %

Specific Gravity, Corrected to 20°C (T 100): **2.75**

Loss on Ignition (T 267)

Loss, %	H2O, %

Water Content (T 265), %: **13.3**

Vane Shear Test on Shelby Tubes (Maine DOT)

Depth taken in tube, ft	3 In.		6 In.		Water Content, %	Description of Material Sampled at the Various Tube Depths
	U. Shear tons/ft²	Remold tons/ft²	U. Shear tons/ft²	Remold tons/ft²		

Comments:

AUTHORIZATION AND DISTRIBUTION

Reported by: **FOGG, BRIAN** Date Reported: **10/14/2011**

Paper Copy: Lab File; Project File; Geotech File

LABORATORY TESTING DATA SHEET

Project Name: Wadsworth Street Bridge Replacement
Location: Thomaston, ME
Reviewed By: Matthew Pelletier
Project No.: 09.0025781.00
Assigned By: Andrew Blaisdell
Project Manager: Andrew Blaisdell
Report Date: 5/26/2013
Date Reviewed: 5/26/2013

Boring No.	Sample No.	Depth Ft.	Lab No.	Sample Data: Water Content %	Do in.	L in.	(1) Unit Wt. PCF	(2) Wet Density PCF	Bulk Gs.	Compression Tests: (3) Other Tests	(4) Strength PSI	(5) Strain %	(6) Conf. Stress	(7) E sec PSI EE+06	(8) Poisson's Ratio	στ PSI	Is_{50} PSI	Rock Formation or Description or Remarks
BB-TSGR-202	R-1	5.8-6.2	1		1.934	4.547	170.6			U	15,761	0.21		6.46	0.21			Metapelite
BB-TSGR-203	R-1	25.2-25.6	2		1.987	4.535	171.8			U	8,411	0.16		4.99	0.15			Metapelite
BB-TSGR-301	R-1	11.5-11.9	3		1.982	4.552	168.8			U	21,463	0.28		6.25	0.07			Metapelite
BB-TSGR-303	R-2	29.0-29.4	4		1.983	4.628	178.7			U	13,897	0.15		7.44	0.14			Metasandstone

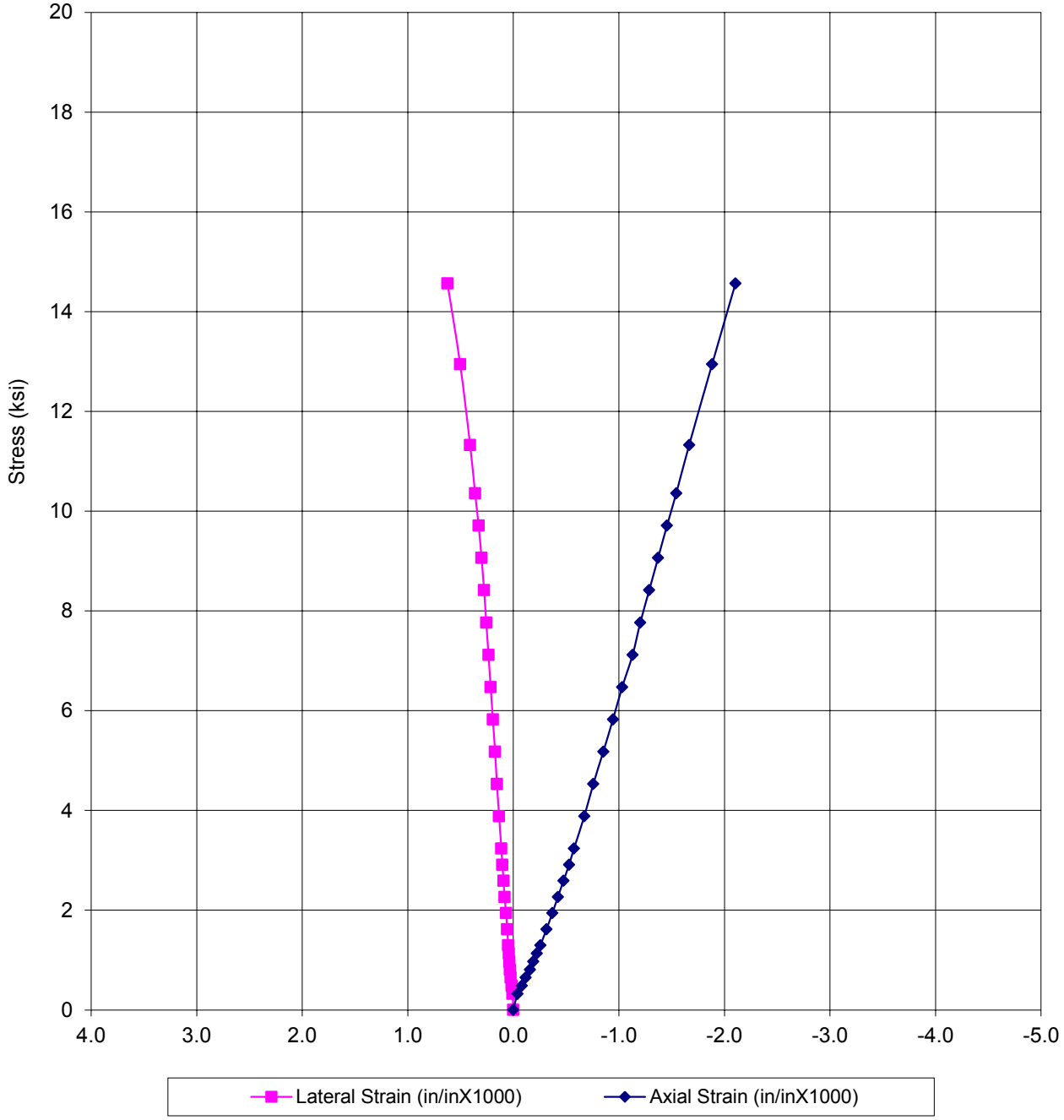
(1) Volume Determined By Measuring Dimensions
(2) Determined by Measuring Dimensions and Weight of Saturated Sample

(3) P=Petrographic PLD=Point Load (diametrical
PLA= Point Load (Axial) RST= Splitting Tensile
U= Unconfined Compressive Strength
(4) Taken at Peak Deviator Stress

(5) Strain at Peak Deviator Stress
(6) Represents Confining Stress on Triaxial Tests
(7) Represents Secant Modulus at 50% of Total Failure Stress
(8) Represents Secant Poisson's Ratio at 50% of Total Failure Stress

THIELSCH ENGINEERING
195 Frances Ave.
Cranston, RI 02910
401-467-6454

Wadsworth Street Bridge Replacement
Thomaston, ME



Rock Unconfined Compression Testing

Boring No.	BB-TSGR-202	File No.	09.0025781.00
Sample No.	R-1	Date:	05/22/13
Depth:	5.9-6.3'	Test No.	U 1

Wadsworth Street Bridge Replacement
Thomaston, ME

GZA Project # 09.0025781.00
CTS-74-13-0003.15

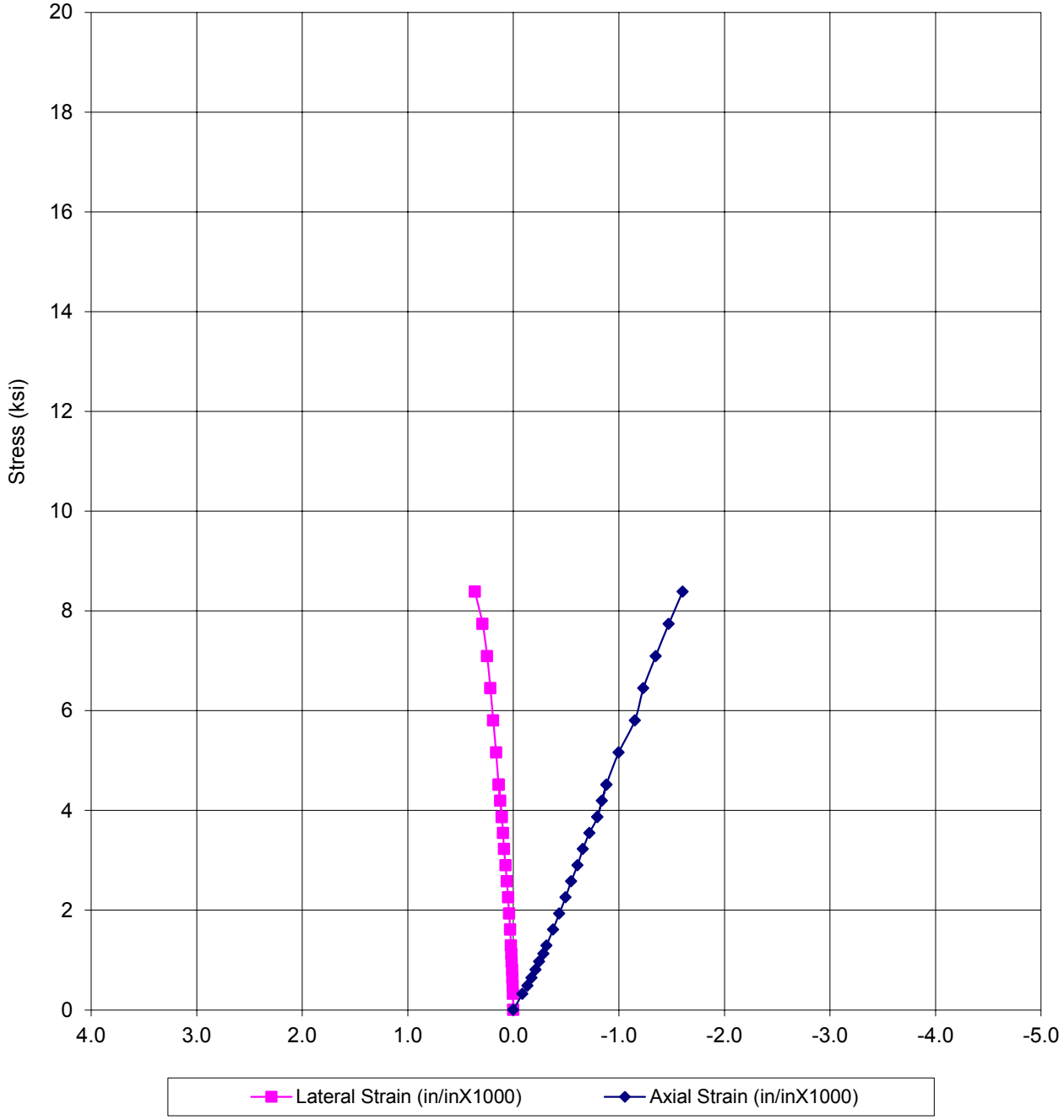
BORING NO.	SAMPLE NO.	DEPTH
BB-TSGR-202	R-1	5.8 – 6.2'

Wadsworth Street Bridge Replacement
Thomaston, ME

GZA Project # 09.0025781.00
CTS-74-13-0003.15

BORING NO.	SAMPLE NO.	DEPTH
BB-TSGR-202	R-1	5.8 – 6.2'

Wadsworth Street Bridge Replacement
Thomaston, ME



Rock Unconfined Compression Testing

Boring No.	BB-TSGR-203	File No.	09.0025781.00
Sample No.	R-1	Date:	05/21/13
Depth:	25.2-25.6'	Test No.	U 2

Wadsworth Street Bridge Replacement
Thomaston, ME

GZA Project # 09.0025781.00
CTS-74-13-0003.15

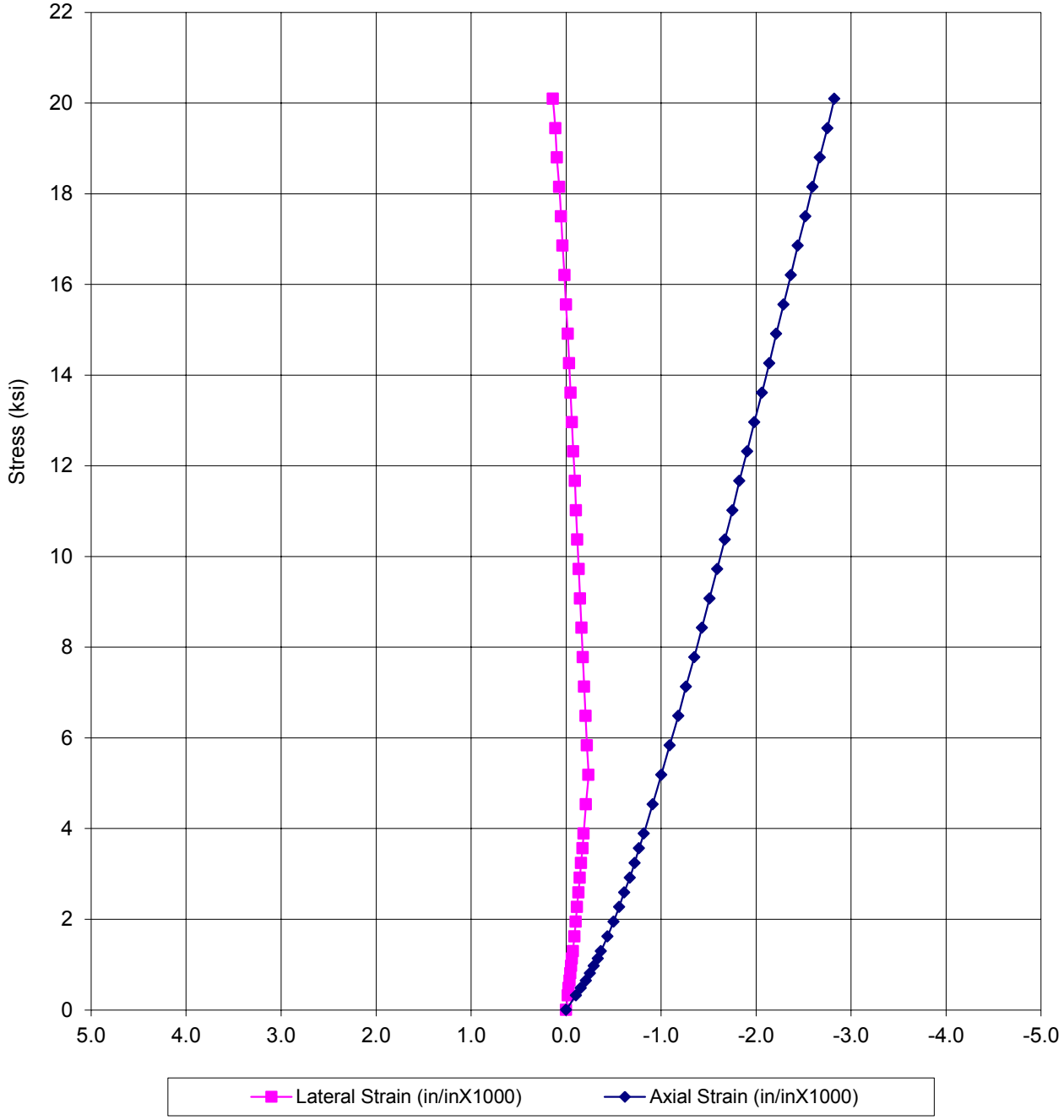
BORING NO.	SAMPLE NO.	DEPTH
BB-TSGR-203	R-1	25.2–25.6'

Wadsworth Street Bridge Replacement
Thomaston, ME

GZA Project # 09.0025781.00
CTS-74-13-0003.15

BORING NO.	SAMPLE NO.	DEPTH
BB-TSGR-203	R-1	25.2–25.6'

Wadsworth Street Bridge Replacement
Thomaston, ME



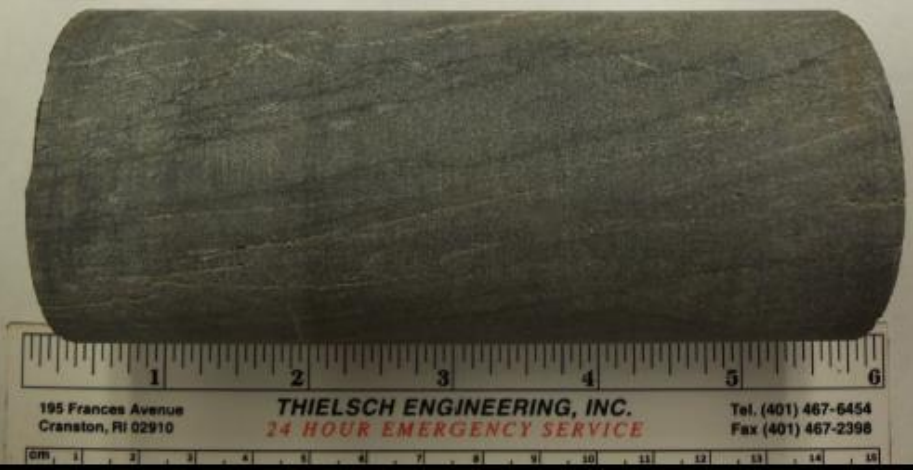
Rock Unconfined Compression Testing

Boring No.	BB-TSGR-301	File No.	09.0025781.00
Sample No.	R-1	Date:	05/22/13
Depth:	11.5-11.9'	Test No.	U 3

Wadsworth Street Bridge Replacement
Thomaston, ME

GZA Project # 09.0025781.00
CTS-74-13-0003.15

BORING NO.	SAMPLE NO.	DEPTH
BB-TSGR-301	R-1	11.5–11.9'

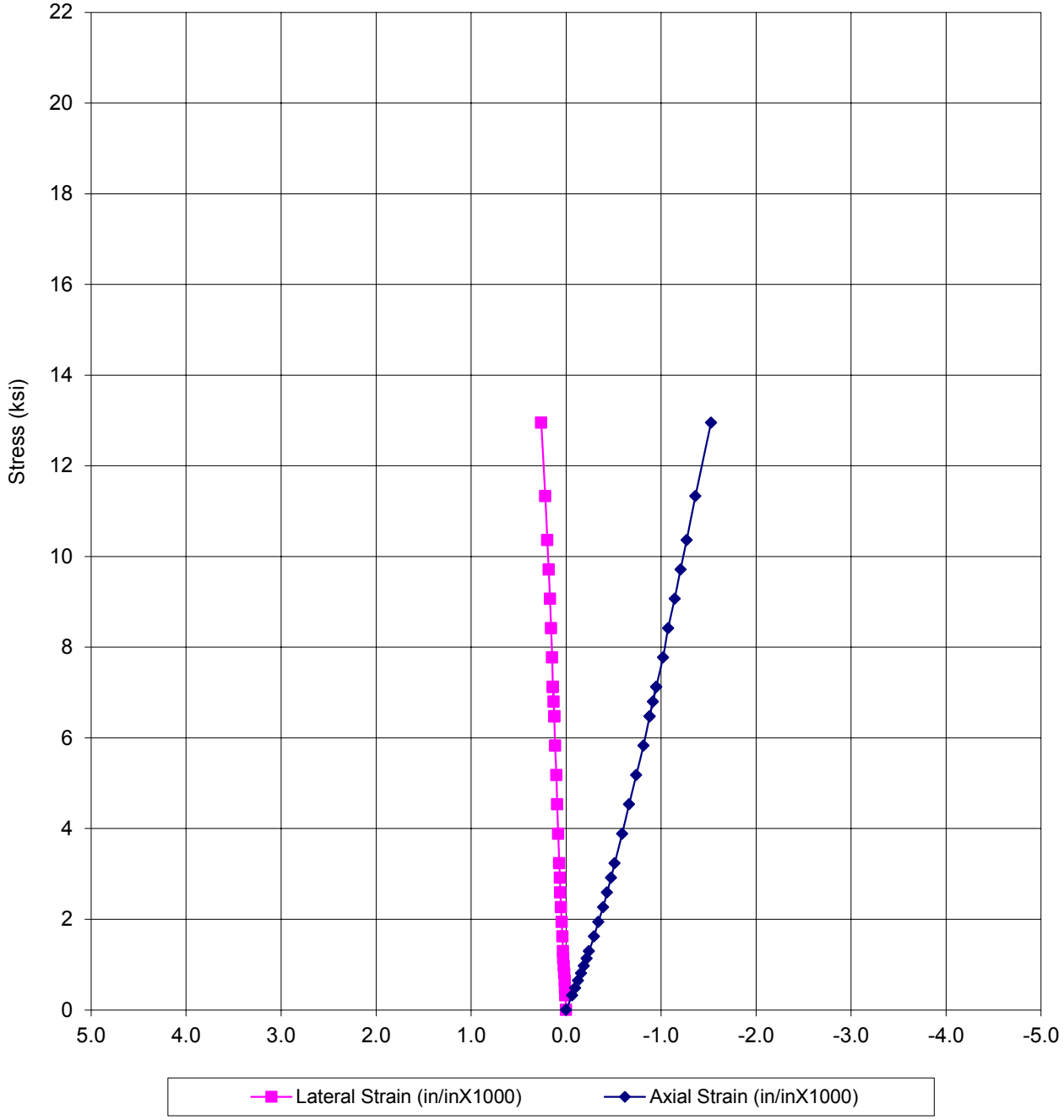


Wadsworth Street Bridge Replacement
Thomaston, ME

GZA Project # 09.0025781.00
CTS-74-13-0003.15

BORING NO.	SAMPLE NO.	DEPTH
BB-TSGR-301	R-1	11.5–11.9'

Wadsworth Street Bridge Replacement
Thomaston, ME



Rock Unconfined Compression Testing

Boring No.	BB-TSGR-303	File No.	09.0025781.00
Sample No.	R-2	Date:	05/22/13
Depth:	29.0-29.4'	Test No.	U 4

Wadsworth Street Bridge Replacement
Thomaston, ME

GZA Project # 09.0025781.00
CTS-74-13-0003.15

BORING NO.	SAMPLE NO.	DEPTH
BB-TSGR-303	R-2	29.0–29.4'

Wadsworth Street Bridge Replacement
Thomaston, ME

GZA Project # 09.0025781.00
CTS-74-13-0003.15

BORING NO.	SAMPLE NO.	DEPTH
BB-TSGR-303	R-2	29.0–29.4'

APPENDIX E

GEOTECHNICAL ENGINEERING CALCULATIONS

GZA
GeoEnvironmental, Inc.
477 Congress Street
Suite 700
Portland, Maine 04101
207-879-9190
Fax 207-879-0099
http://www.gza.com

Engineers and Scientists

Wadsworth Street Bridge
JOB: 09.0025781.00
SUBJECT: Downdrag, Abutments 1 and 2
SHEET: 1 of 9
CALCULATED BY: A. Blaisdell, 9/22/14
CHECKED BY: C. Snow, 9/22/14

OBJECTIVE: Evaluate downdrag on HP14x117 piles supporting Abutments 1 and 2.

EVALUATION: The settlement evaluation indicates that up to 5 inches of total settlement may occur adjacent to Abutment 1 and up to 1.5 inches of total settlement may occur adjacent to Abutment 2. Based on the sandy nature of the soils, we anticipate that at least half of the settlement will occur during fill placement, likely before the piles are installed. However, rate of settlement data is not available, and the sequence of fill placement and pile installation is unknown. Therefore, the piles must be designed to resist downdrag loading.

LRFD Section C3.11.8 indicates that downdrag loading should be calculated using either the alpha, lambda, or beta method for clay and an effective stress method for sand. Based on historical Maine practice, the beta method was used for sand and clay. Beta values were selected for new fill, river bottom sand, and silt/clay equal to 0.35, 0.3 and 0.23, respectively. It was assumed that the portions of these layers that experience greater than 0.4 inches of total settlement will contribute to downdrag. The nominal downdrag load was calculated using effective stressescalculated using either 1-D stress increase (Abutment 1) or Bousinesq theory (Abutment 2) as described on the following sheets.

A load factor of 1.0 was applied to the nominal downdrag load in accordance with past practice on MaineDOT projects.

CONCLUSION: The nominal and factored downdrag load is estimated at 15 kips at Abutment 1 and 66 kips at Abutment 2. Factored pile loads and required driving resistance are summarized below.

	Abutment 1	Abutment 2
Maximum factored pile load (kips)	15	66
Factored Downdrag Load (kips)	162	162
Required Factored Driving Resistance (kips)	177	228
Required Nominal Driving Resistance (kips)	272	351

Calculation of Settlement based on the Hough Method (AASHTO LRFD Eq. 10.6.2.4.2-2 and -3) and Downdrag based on the Beta Method (AASHTO LRFD Eq. 10.7.3.8.6c-1)

Wadsworth Street Bridge Replacement, File No. 25781.00

Calc by ARB 9/22/2014

Review by

Location: South Approach (Abutment 1), immediately behind abutment

Hough Method Equations

$$Se = \sum_{i=1}^{n} \Delta H_i$$

(10.6.2.4.2-2)

$$\Delta H_i = H_c \left(\frac{1}{C'}\right) log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$$

(10.6.2.4.2-3)

				Settlement Evaluation					Downdrag Evaluation					
Layer	Hc (ft)	γ' pcf	σ_{vo}' psf	D to midpt ft	$\Delta\sigma v$ midpt psf	Assumed N / USCS	C' dim.	ΔHi sublayer in	ΔHi total in	Beta β	σ_{vf}' psf	Maximum Downdrag kips	Calculated Downdrag kips	
New Fill	4	62.6	--	--	1565	--	--	0	1.4	0.35	1565	10	10	
Silt/Clay	4	57.6	115	2	1565	10 bpf / ML	40	1.4	1.4	0.23	1680	7	5	**

Se (in) **1.4** **Downdrag (kips)** **15**

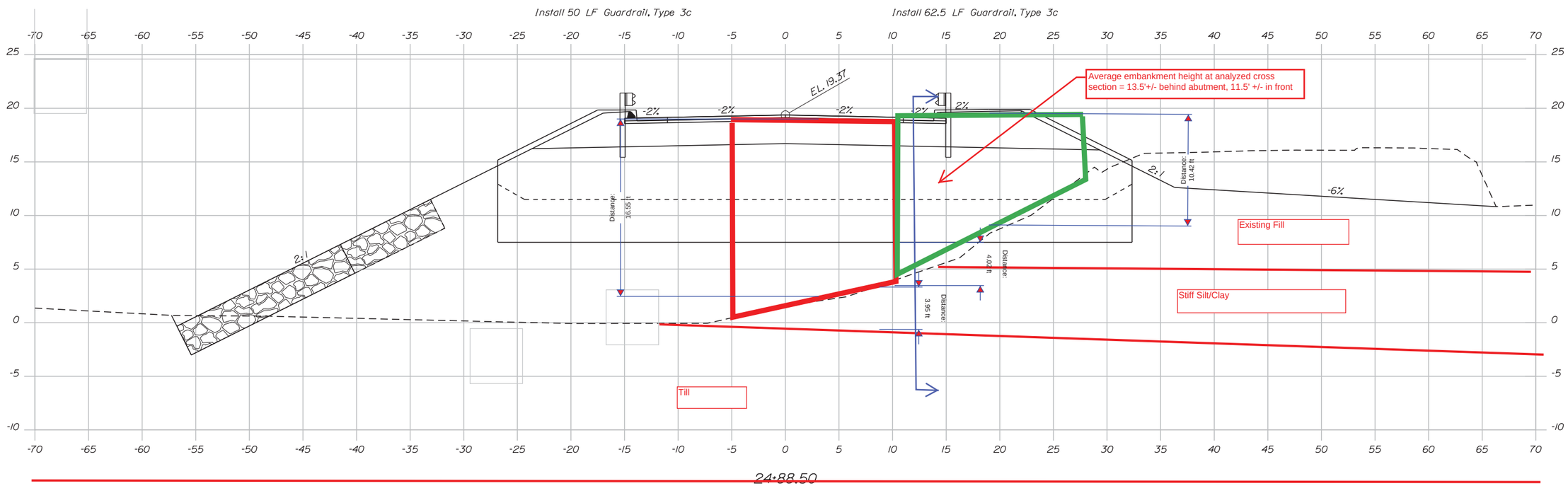
Basis of Assumptions and Evaluation

1. 13.5' high (avg.) embankment, all above water (q=1,690 psf) and 11.5' high slope in front of abutment (q=1,440 psf)
2. The soil layering is based on the soil types, strata thicknesses and average N-values in borings BB-TSGR-101 and -201.
3. Stress increase at the midpoint of each layer was calculated using 1-D load spread based on average fill height of 12.5' (q= 1,565 psf). Bousinesq elastic theory was not assumed due to location of compressible layer immediately beneath embankment.
4. The new fill will not settle, but it will move down and impose downdrag loading.
5. Total settlement, rather than post-construction settlement, is used for downdrag load calculation, due to lack of information on rate of settlement.
6. Beta values were selected based on historical Maine practice.
7. Check reasonableness of estimated Clay/Silt settlement by calculating corresponding equivalent Coefficient of Consolidation. Should at least be equal to RR.

***Check equivalent consolidation coefficient for Clay/Silt layer:*

$S=H*C*\log((\sigma_{vo}'+\Delta\sigma_v)/\sigma_{vo}')$

$C=S/(H*\log((\sigma_{vo}'+\Delta\sigma_v)/\sigma_{vo}')$ C= 0.025 ---For silt, corresponds to a moderate to high RR value, OK for high blow count material.



Sta. 25+00.00 to Sta. 27+80.00

PROJ. MANAGER		MAP	BY	DATE
DESIGN-DETAILED	ETC		BJN	OCT 2011
CHECKED-REVIEWED	----		----	----
DESIGN2-DETAILED2	----		----	----
DESIGN3-DETAILED3	----		----	----
REVISIONS 1	----		----	----
REVISIONS 2	----		----	----
REVISIONS 3	----		----	----
REVISIONS 4	----		----	----
FIELD CHANGES	----		----	----

SIGNATURE

P.E. NUMBER

DATE

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-1675(500)

PIN 16755.00

BRIDGE NO. 2904

HIGHWAY PLANS

WADSWORTH STREET BRIDGE
OVER
ST. GEORGE RIVER
WADSWORTH STREET
THOMASTON
KNOX COUNTY

CROSS SECTIONS

SHEET NUMBER
25
OF 63

Filename: ...\msta\025_Xsect_25+00_011.dgn Division: BRIDGE Username: Brian.J.Nichols Date:8/28/2014

Calculation of Settlement based on the Hough Method (AASHTO LRFD Eq. 10.6.2.4.2-2 and -3) and Downdrag based on the Beta Method (AASHTO LRFD Eq. 10.7.3.8.6c-1)

Wadsworth Street Bridge Replacement, File No. 25781.00

Calc by ARB 9/22/2014

Review by

Location: North Approach (Abutment 2), immediately behind abutment

Hough Method Equations

$$Se = \sum_{i=1}^{n} \Delta H_i$$

(10.6.2.4.2-2)

$$\Delta H_i = H_c \left(\frac{1}{C'}\right) log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right)$$

(10.6.2.4.2-3)

Location Abutment 2, Immediately behind abutment, center of abutment

				Settlement Evaluation					Downdrag Evaluation					
Layer	Hc (ft)	γ' pcf	σ_{vo}' psf	D to midpt ft	$\Delta\sigma v$ midpt psf	Assumed N / USCS	C' dim.	ΔHi sublayer in	ΔHi total in	Beta β	σ_{vf}' psf	Maximum Downdrag kips	Calculated Downdrag kips	
New Fill	4	62.6	--	--	1875	--	--	0	5.1	0.35	1875	12	12	
RB	11	57.6	317	7	1735	4 bpf / SM	35	3.1	5.1	0.30	2052	32	32	
Silt/Clay	7	55.6	828	16	1559	2 bpf / ML	25	1.5	2.1	0.23	2387	18	18	**
RB	5	57.6	1167	22	1442	8 bpf / SM	40	0.5	0.5	0.30	2609	18	4	
							Se (in)	**5.1**				**Downdrag (kips)**	**66**	

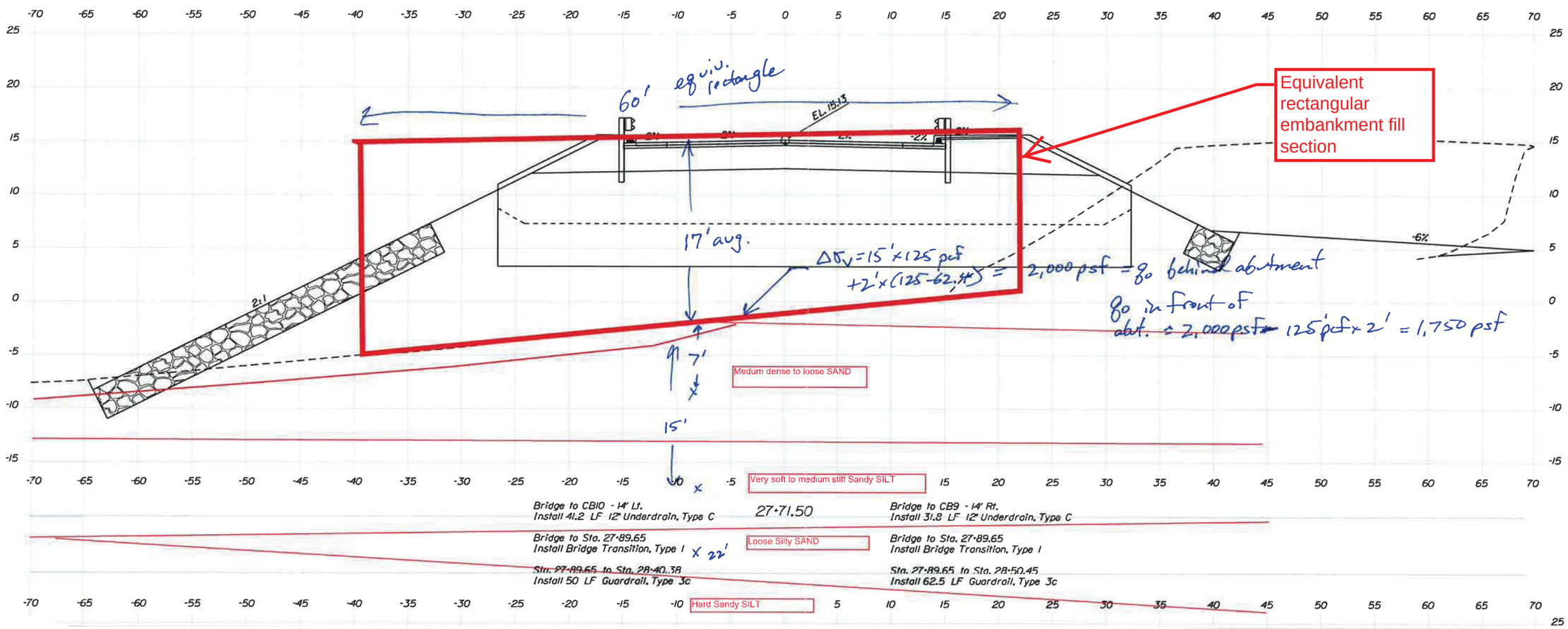
Basis of Assumptions and Evaluation

1. 17' high (avg.) embankment, 2' below water and 15' above (q=2,000 psf) and 15' high slope in front of abutment (q=1,750 psf)
2. The soil layering is based on the soil types, strata thicknesses and average N-values in borings BB-TSGR-103 and -205.
3. Stress increase at the midpoint of each layer was calculated using Bousinesq elastic theory at the location of the abutment in the center of the embankment.
4. The new fill will not settle, but it will move down and impose downdrag loading.
5. Total settlement, rather than post-construction settlement, is used for downdrag load calculation, due to lack of information on rate of settlement.
6. Beta values were selected based on historical Maine practice.
7. Check reasonableness of estimated Clay/Silt settlement by calculating corresponding equivalent Coefficient of Consolidation. Should at least be equal to RR.

***Check equivalent consolidation coefficient for Clay/Silt layer:*

$S=H*C*\log((\sigma_{vo}'+\Delta\sigma_v)/\sigma_{vo}')$

$C=S/(H*\log((\sigma_{vo}'+\Delta\sigma_v)/\sigma_{vo}')$ C= 0.040 ---For silt, corresponds to twice anticipated recompression index, RR, considered reasonable for settlement of Silt/Clay, allows for some virgin consolidation.



STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-1675(500)

PIN 16795.00

BRIDGE NO. 2804

HIGHWAY PLANS

	BY	DATE
PROJ. MANAGER		
DESIGN-DETAILED	ETC	OCT 2011
CHECKED-REVIEWED		
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE

P.E. NUMBER

DATE

WADSWORTH STREET BRIDGE
ST. GEORGE RIVER
WADSWORTH STREET
THOMASTON, KNOX COUNTY

CROSS SECTIONS

SHEET NUMBER
25
OF 63

Sta. 25+00.00 to Sta. 27+80.00

Filename: ...\msta\025_Xsect_25+00_011.dgn Division: BRIDGE Username: Brian.J.Nichols Date:8/28/2014

ABUTMENT 2
~~[illegible]~~ River Side

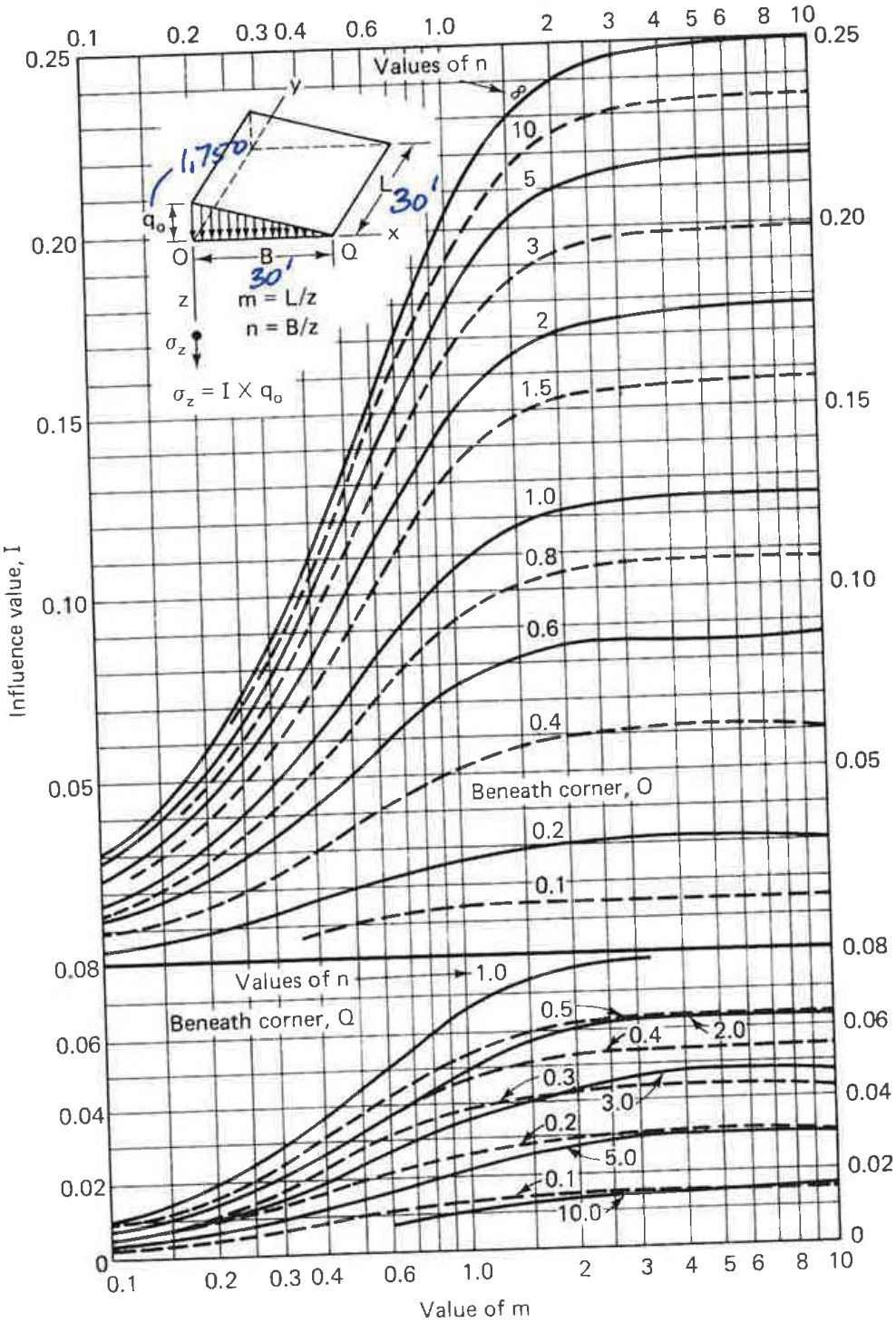


Fig. 8.24 Influence values for vertical stress under the corners of a triangular load of limited length (after U.S. Navy, 1971).

(L=60')

Layer	Z	m=L/z	I	$\sigma_z = q_o \times I \times 2$	m	I	σ_z
1	7'	4.3	0.21	735 psf	8.6	0.25	438 psf
2	15'	2	0.171	599 psf	4	0.176	308 psf
3	22'	1.4	0.149	522 psf	2.8	0.152	266 psf

Center

CORNER

ABUTMENT 2
Approach Side

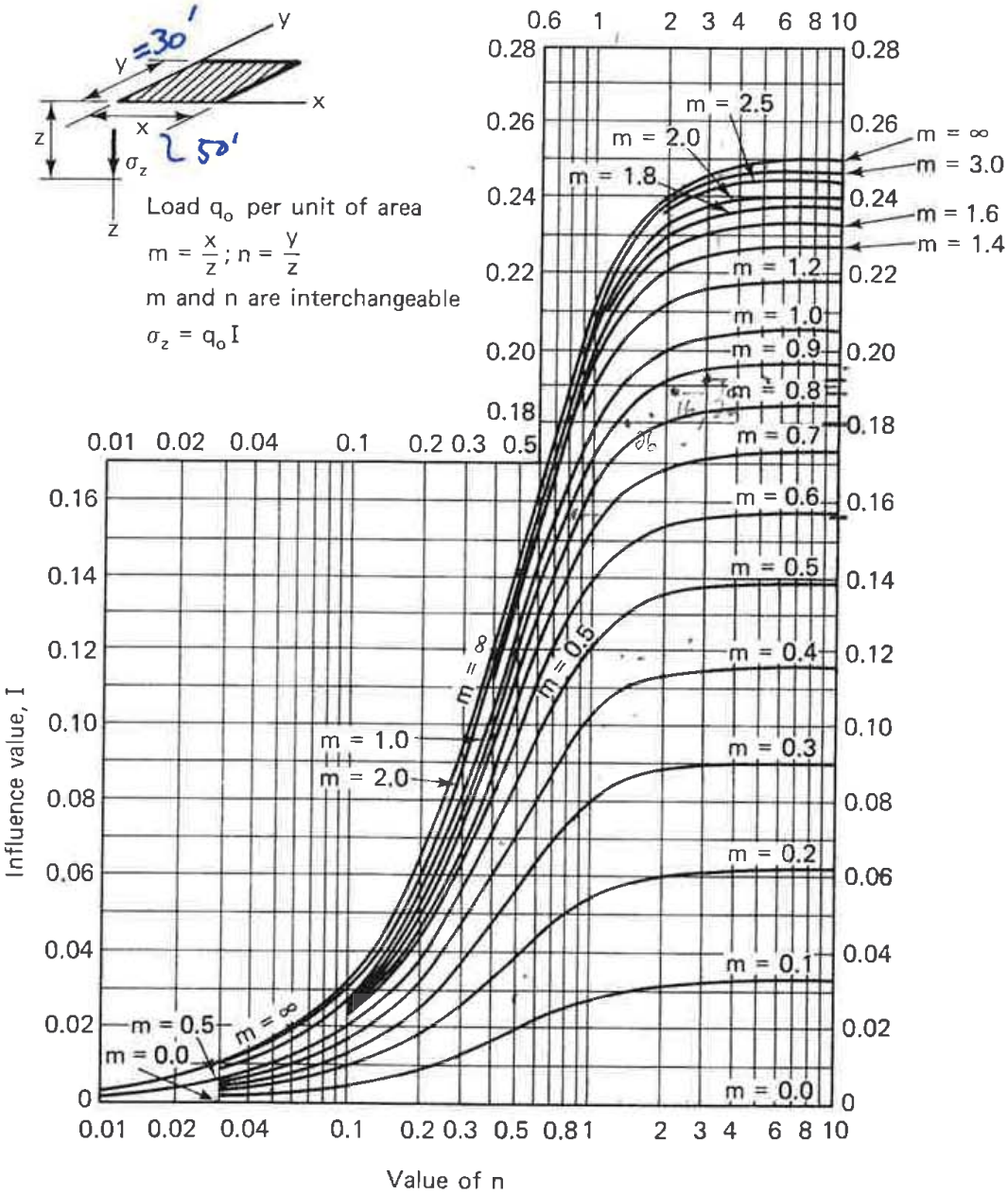


Fig. 8.21 Influence value for vertical stress under corner of a uniformly loaded rectangular area (after U.S. Navy, 1971).

	Layer	z	m	n	I	$\sigma_z = q_o \times I \times 2$ (psf)
CENTER	1	7'	7.1	4.3	0.25	1,000
	2	15'	3.3	2	0.24	960
	3	22'	2.3	1.4	0.23	920
						$\sigma_z = q_o \times I$
CORNER	1	7'	7.1	8.6	0.25	500
	2	15'	3.3	4	0.246	492
	3	22'	2.3	2.8	0.24	480

β_z = shape factor taken as specified in Table 10.6.2.4.2-1 (dim)

ν = Poisson's Ratio, taken as specified in Article 10.4.6.3 if direct measurements of ν are not available from the results of in situ or laboratory tests (dim)

Unless E_s varies significantly with depth, E_s should be determined at a depth of about 1/2 to 2/3 of B below the footing, where B is the footing width. If the soil modulus varies significantly with depth, a weighted average value of E_s should be used.

Table 10.6.2.4.2-1—Elastic Shape and Rigidity Factors, EPRI (1983)

L/B	Flexible, β_z (average)	β_z Rigid
Circular	1.04	1.13
1	1.06	1.08
2	1.09	1.10
3	1.13	1.15
5	1.22	1.24
10	1.41	1.41

only a single value of soil modulus, and Young's modulus varies with depth as a function of overburden stress. Therefore, in selecting an appropriate value for soil modulus, consideration should be given to the influence of soil layering, bedrock at a shallow depth, and adjacent footings.

For footings with eccentric loads, the area, A', should be computed based on reduced footing dimensions as specified in Article 10.6.1.3.

Estimation of spread footing settlement on cohesionless soils by the empirical Hough method shall be determined using Eqs. 10.6.2.4.2-2 and 10.6.2.4.2-3. *SPT* blow counts shall be corrected as specified in Article 10.4.6.2.4 for depth, i.e. overburden stress, before correlating the *SPT* blow counts to the bearing capacity index, C'.

$$S_e = \sum_{i=1}^{n} \Delta H_i \qquad (10.6.2.4.2\text{-}2)$$

in which:

$$\Delta H_i = H_c \frac{1}{C'} \log\left(\frac{\sigma'_o + \Delta\sigma_v}{\sigma'_o}\right) \qquad (10.6.2.4.2\text{-}3)$$

where:

n = number of soil layers within zone of stress influence of the footing

ΔH_i = elastic settlement of layer i (ft)

H_C = initial height of layer i (ft)

C' = bearing capacity index from Figure 10.6.2.4.2-1 (dim)

In Figure 10.5.2.4.2-1, N' shall be taken as $N1_{60}$, Standard Penetration Resistance, N (blows/ft), corrected for overburden pressure as specified in Article 10.4.6.2.4.

The Hough method was developed for normally consolidated cohesionless soils.

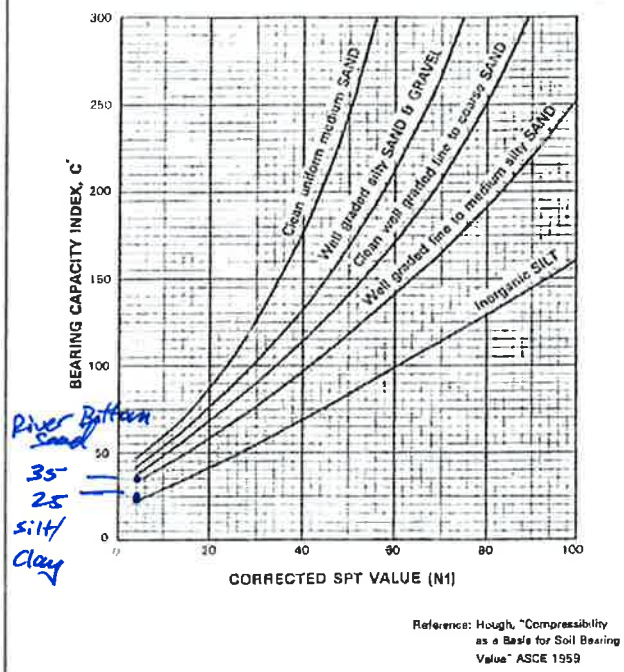
The Hough method has several advantages over other methods used to estimate settlement in cohesionless soil deposits, including express consideration of soil layering and the zone of stress influence beneath a footing of finite size.

The subsurface soil profile should be subdivided into layers based on stratigraphy to a depth of about three times the footing width. The maximum layer thickness should be about 10 ft.

While Cheney and Chassie (2000), and Hough (1959), did not specifically state that the SPT N values should be corrected for hammer energy in addition to overburden pressure, due to the vintage of the original work, hammers that typically have an efficiency of approximately 60 percent were in general used to develop the empirical correlations contained in the method. If using SPT hammers with efficiencies that differ significantly from this 60 percent value, the N values should also be corrected for hammer energy, in effect requiring that $N1_{60}$ be used.

σ'_o = initial vertical effective stress at the midpoint of layer i (ksf)

$\Delta\sigma_{vi}$ = increase in vertical stress at the midpoint of layer i (ksf)



Reference: Hough, "Compressibility as a Basis for Soil Bearing Value" ASCE 1959

Figure 10.6.2.4.2-1—Bearing Capacity Index versus Corrected SPT (modified from Cheney and Chassie, 2000, after Hough, 1959)

The Hough method is applicable to cohesionless soil deposits. The "Inorganic Silt" curve should generally not be applied to soils that exhibit plasticity. The settlement characteristics of cohesive soils that exhibit plasticity should be investigated using undisturbed samples and laboratory consolidation tests as prescribed in Article 10.6.2.4.3.

10.6.2.4.3—Settlement of Footings on Cohesive Soils

Spread footings in which cohesive soils are located within the zone of stress influence shall be investigated for consolidation settlement. Elastic and secondary settlement shall also be investigated in consideration of the timing and sequence of construction loading and the tolerance of the structure to total and differential movements.

Where laboratory test results are expressed in terms of void ratio, e, the consolidation settlement of footings shall be taken as:

- For overconsolidated soils where $\sigma'_p > \sigma'_o$, see Figure 10.6.2.4.3-1:

$$S_c = \left[\frac{H_c}{1+e_o}\right]\left[C_r \log\left(\frac{\sigma'_p}{\sigma'_o}\right) + C_c \log\left(\frac{\sigma'_f}{\sigma'_p}\right)\right] \quad (10.6.2.4.3\text{-}1)$$

- For normally consolidated soils where $\sigma'_p = \sigma'_o$:

C10.6.2.4.3

In practice, footings on cohesive soils are most likely founded on overconsolidated clays, and settlements can be estimated using elastic theory (Baguelin et al., 1978), or the tangent modulus method (Janbu, 1963, 1967). Settlements of footings on overconsolidated clay usually occur at approximately one order of magnitude faster than soils without preconsolidation, and it is reasonable to assume that they take place as rapidly as the loads are applied. Infrequently, a layer of cohesive soil may exhibit a preconsolidation stress less than the calculated existing overburden stress. The soil is then said to be underconsolidated because a state of equilibrium has not yet been reached under the applied overburden stress. Such a condition may have been caused by a recent lowering of the groundwater table. In this case, consolidation settlement will occur due to the additional load of the structure and the settlement that is occurring to reach a state of equilibrium. The total consolidation settlement due to these two components can be estimated by Eq. 10.6.2.4.3-3 or Eq. 10.6.2.4.3-6.

Slope W Output
Proposed Embankment
Thomaston Bridge Stability Evaluation
09.0025781.00
GZA GeoEnvironmental, Inc.

File Name: 25781 embankment.gsz
Title: Thomaston, Name: proposed embankment
Date: 9/8/2014
Created By: Jennifer Baron

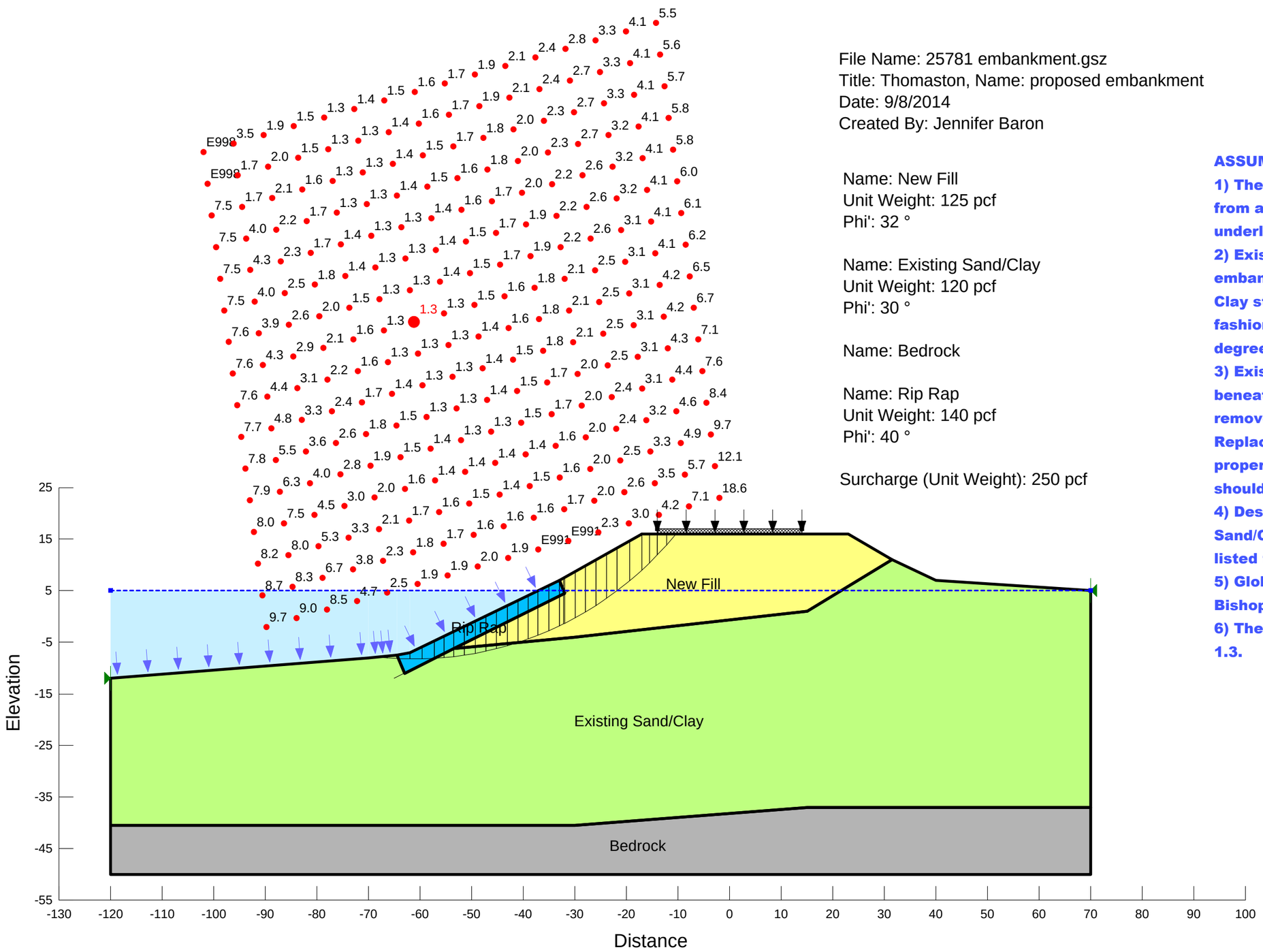
Name: New Fill
Unit Weight: 125 pcf
Phi': 32 °

Name: Existing Sand/Clay
Unit Weight: 120 pcf
Phi': 30 °

Name: Bedrock

Name: Rip Rap
Unit Weight: 140 pcf
Phi': 40 °

Surcharge (Unit Weight): 250 pcf



ASSUMPTIONS, INPUTS AND RESULTS:

1) The Abutment 2 (north) approach controls from a global stability standpoint due to weaker underlying materials than Abutment 1.

2) Existing soil profile beneath new embankment (labeled herein as Existing Sand/ Clay stratum) will primarily behave in drained fashion; use an effective friction angle of 30 degrees for all soil.

3) Existing very soft organic silt encountered beneath the proposed abutment footprint will be removed and replaced with granular fill. Replacement fill was assumed to have the same properties as the Existing Sand/Clay, which should be conservative.

4) Design parameters for New Fill and Existing Sand/Clay and design surcharge pressure are listed to the left.

5) Global stability was evaluated using the Bishop method.

6) The minimum calculated factor of safety is 1.3.

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Portland, Maine 04101
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http://www.gza.com

Engineers and Scientists

JOB: 09.0025781.00 Wadsworth Street
SUBJECT: Axial Capacity - Abutments
SHEET: 1 OF 11
CALCULATED BY J. Baron 9/3/14, A. Blaisdell, 9/22/14
CHECKED BY A. Blaisdell, 9/8/14

Objective

Evaluate the axial geotechnical resistance of the abutment piles for the Wadsworth Street Bridge Replacement in Thomaston, ME

Methodology

Evaluate proposed pile section for governing factored axial compression resistance as follows.

1. Nominal Compressive Resistance
2. Factored Structural Compressive Resistance - Strength Limit State
3. Factored Structural Compressive Resistance - Extreme/Service Limit State
4. Geotechnical Resistance (Static Analysis)
5. Geotechnical Resistance (Drivability Analysis)
6. Factored Geotechnical Resistance - Strength Limit State
7. Factored Geotechnical Resistance - Extreme/Service Limit State

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 6th edition, 2012. (AASHTO LRFD)

Soil Properties

Consider Wadsworth Street Bridge Interpretive Subsurface Profile (see Figure 3), subsurface layering and properties defined below:

DESIGN SUBSURFACE PROFILE – ABUTMENT 1 (Borings BB-TSGR-101 and -201)			
Strata Designation	**Thickness (feet)**	**Total Unit Weight (pcf)**	**Representative Φ' (°) / Su (psf) for layer**
New Fill	6	125	34°
Silt/Clay	4	118	1,000 psf
Glacial Till	11	130	38°
Depth to Bedrock	21	--	--

DESIGN SUBSURFACE PROFILE – ABUTMENT 2 (Borings BB-TSGR-103 and -205)			
Strata Designation	**Thickness (feet)**	**Total Unit Weight (pcf)**	**Representative Φ' (°) / Su (psf) for layer**
New Fill	4	125	34°
River Bottom Deposit	11	120	30°
Silt/Clay	7	118	1,000 psf
River Bottom Deposit	7	120	30°
Glacial Till	12	130	38°
Bedrock	41	--	--

Structural Properties

HP14x117

Yield Strength of Steel $F_y := 50ksi$

Area of section $A_s := 34.4in^2$

Young's Modulus of Steel $E_s := 30000 \cdot ksi$

Section Properties $I_x := 1220 \cdot in^4$

1. Nominal Structural Compressive Resistance P_n

Nominal Compressive Resistance: $P_n := 0.66^{\lambda} \cdot F_y \cdot A_s$ AASHTO Eq. 6.9.5.1-1

Determine normalized column slenderness factor λ

$$\lambda := \left(\frac{K \cdot l}{r_s \cdot \pi}\right)^2 \cdot \frac{F_y}{E}$$ AASHTO Eq. 6.9.4.1-3 pg. 6-74

$\lambda := 0$ Where the pile is fully embedded, AASHTO 10.7.3.13.1.
NOTE: Assumption may not be applicable for integral abutment pile, to be assessed by bridge designer.

Giving: $P_n := 0.66^{\lambda} \cdot F_y \cdot A_s$ $P_n = 1720 \cdot kip$

2. Factored Structural Compressive Resistance - Strength Limit State:

Factor for piles in compression under hard driving conditions:

From Article 6.5.4.2 $\phi_c := 0.5$

Factored Compressive Resistance for Strength Limit State:

$P_r := \phi_c \cdot P_n$ AASHTO Eq. 6.9.2.1-1 pg. 6-71

$P_r = 860 \cdot kip$

3. Factored Structural Compressive Resistance - Service/Extreme Limit State:

Resistance Factors for Extreme Limit States:

From Article 10.5.5.1 and 10.5.5.3 $\phi := 1$

Factored Compressive Resistance for Service/Extreme Limit State:

$P_r := \phi \cdot P_n$ AASHTO Eq. 6.9.2.1-1 pg. 6-71

$P_r = 1720 \cdot kip$

4. Geotechnical Axial Resistance - Static Analysis

AASHTO Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock is controlled by the structural limit state.

Used SPILE to estimate side friction resistance during driving using the Nordlund Thurman method, estimates 69 kips of side resistance for Abutment 1 piles and 140 kips of side resistance for Abutment 2 piles. SPILE output is attached on Sheets 6 and 7.

Required nominal resistance of 272 kips for design of Abutment 1 based on a maximum factored load of 162 kips with a downdrag load of 15 kips and a 0.65 resistance factor. Required nominal resistance of 351 kips for Abutment 2 based on a maximum factored load of 162 kips with a downdrag load of 66 kips and a 0.65 resistance factor.

Estimated friction distribution and % shaft resistance using GRLWEAP based on design loads.

5. Geotechnical Axial Resistance - Drivability Analysis

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ AASHTO Eq. 10.7.8.1

$f_y := 50ksi$ yield Strength of steel

$\phi_{da} := 1.0$ AASHTO Table 10.5.5.2.3-1 Refers to Article 6.5.4.2, Pg. 6-28

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ $\sigma_{dr} = 45 \cdot ksi$ Driving Stress in pile cannot exceed 45 ksi

Evaluate nominal geotechnical capacity using GRLWEAP.

Abutment 1 - Drive pile through 21 feet of soil to rock with toe quake representative of very hard driving conditions (0.04in). Pile length is modeled as 21 feet with 14 feet of stickup.

Estimated skin friction is 28% of required nominal resistance. For short pile; driving stress is anticipated to control resistance.

Abutment 2 - Drive pile through 41 feet of soil to rock with toe quake representative of moderate driving conditions (0.1 in). Pile length is modeled as 41 feet with 14 feet of stickup.

Estimated skin friction is 42% of required nominal resistance. For long pile, higher final blow count is anticipated to control

Drive piles with a APE D19-42 open-ended diesel hammer with a rated energy of 47,200 ft-lb. The proposed hammer is larger than necessary to achieve the required nominal resistance for abutment piles, but is sized to achieve the required nominal resistance for pier piles. A reduced fuel setting is considered due to the lower demand; chamber pressure of 1247 psi (72%).

Allowable stress in piles during driving is 0.9 fy=45 ksi

GRLWEAP Output is attached on Sheets 8 through 11.

Abutment 1:

$R_{ndr1} := 272kip$ Required nominal geotechnical resistance, pile driving stress=18 ksi, final penetration resistance=4 bpi.

Abutment 2:

$R_{ndr2} := 351kip$ Required nominal geotechnical resistance, pile driving stress=19 ksi, final penetration resistance=5 bpi.

6. Factored Drivability Resistance - Strength Limit State:

Strength Limit State Factored Drivability Resistance:

Abutment 1:

$R_{ndr1_factored} := R_{ndr1} \cdot \phi_{dyn}$

PDA, WEAP and CAPWAP used to establishing driving criteria

$\phi_{dyn} := 0.65$ AASHTO Table 10.5.5.2.3-1

$R_{ndr1_factored} := R_{ndr1} \cdot \phi_{dyn}$

$R_{ndr1_factored} = 177 \cdot kip$

Abutment 2:

$R_{ndr2_factored} := R_{ndr2} \cdot \phi_{dyn}$

$R_{ndr2_factored} = 228 \cdot kip$

7. Factored Drivability Resistance - Service/Extreme Limit States:

Service and Extreme Limit State Factored Drivability Resistance:

Resistance Factors for Extreme Limit States: $\phi_{serv_ext} := 1$

From Article 10.5.5.1 and 10.5.5.3

$R_{ndr1_serv_ext} := R_{ndr1} \cdot \phi_{serv_ext}$

$R_{ndr1_serv_ext} = 272 \cdot kip$

$R_{ndr2_serv_ext} := R_{ndr2} \cdot \phi_{serv_ext}$

$R_{ndr2_serv_ext} = 351 \cdot kip$

Summary of Results - Axial Loading:

ASTM A572, HP 14x117	Structural Resistance (kips)	Geotechnical Static Resistance (kips)	Required Resistance (kips)	Governing Resistance (kips)
Abutment 1: Strength Limit State, Design	860	n/a	177	177
Abutment 1: Service/Extreme Limit State, Design	1720	n/a	272	272
Abutment 2: Strength Limit State, Design	860	n/a	228	228
Abutment 2: Service/Extreme Limit State, Design	1720	n/a	351	351

Results indicate drivability controls, APE D19-42 is suitable to penetrate overburden and achieve strength limit state resistance. The results indicate a smaller hammer could be used at the abutments to achieve the required geotechnical capacity, but use of the D19-42 is feasible and appropriate for the piers.

ABUT1.TXT

```
ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿
³           Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods           ³
³                                                                              ³
³ Project Name   : Wadsworth St. Bridge Client          : MaineDOT            ³
³ File Name      : Thomaston,ME         Project Manager : Blaisdell            ³
³ Date           : 05/09/13             Computed by     : ETL                  ³
³                                                                              ³
³ Depth of Top of Pile =   0.00 ft.         Pile length          =  21.00 ft.  ³
³ Depth to Water Table =   2.00 ft.                                            ³
³ Type of Pile         = H Pile                                                ³
³   HP 14x117                                                                  ³
³                                                                              ³
³                            SKIN FRICTION CONTRIBUTION                        ³
³                                                                              ³
³ Layer    Soil        Thickness    Effective   Internal   N-SPT      Pile    ³
³          Type                       Stress    Friction             Perimeter ³
³                         (ft)        (psf)      Angle                (ft)     ³
³                                                                              ³
³   1   Cohesionless       6.00        312.60     34.00      --        4.85    ³
³   2   Cohesive           4.00        611.60       ---      --        4.85    ³
³   3   Cohesionless      11.00       1094.60     38.00      --        4.85    ³
³                                                                              ³
³                                                                              ³
³ Layer    Soil       Undrained Shear  Adhesion  Pile    Sliding       Skin    ³
³          Type           Strength               Taper   Friction   Resistance ³
³                          (psf)                           Angle      (Kips)   ³
³                                                                              ³
³   1   Cohesionless         --        --------   ----    28.88        5.34   ³
³   2   Cohesive         1000.00        500.00    ----    -----        9.70   ³
³   3   Cohesionless         --        --------   ----    32.28       53.81   ³
³                                                                              ³
³                                                                              ³
³                                          Total Side Friction  :     68.85    ³
³                                                                              ³
³                          POINT RESISTANCE CONTRIBUTION                       ³
³                                                                              ³
³ Effective     Internal      SPT      Pile End     Bearing        End Bearing ³
³ Stress at     Friction     Value       Area       Capacity       Resistance  ³
³ pile Tip       Angle                              Factor                     ³
³   (psf)                              (ft*ft)        Nq             (Kips)    ³
³                                                                              ³
³  1466.40       38.00       -----       0.24       110.40            27.92    ³
³                                                                              ³
³                                 Limiting End Bearing Resistance :    64.17   ³
³                                                                              ³
³                                                                              ³
³                                 Ultimate Static Pile Capacity   :    96.77   ³
³                                                                              ³
ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ
```

69 kips/249 nominal=28% side resistance

ABUT2.TXT

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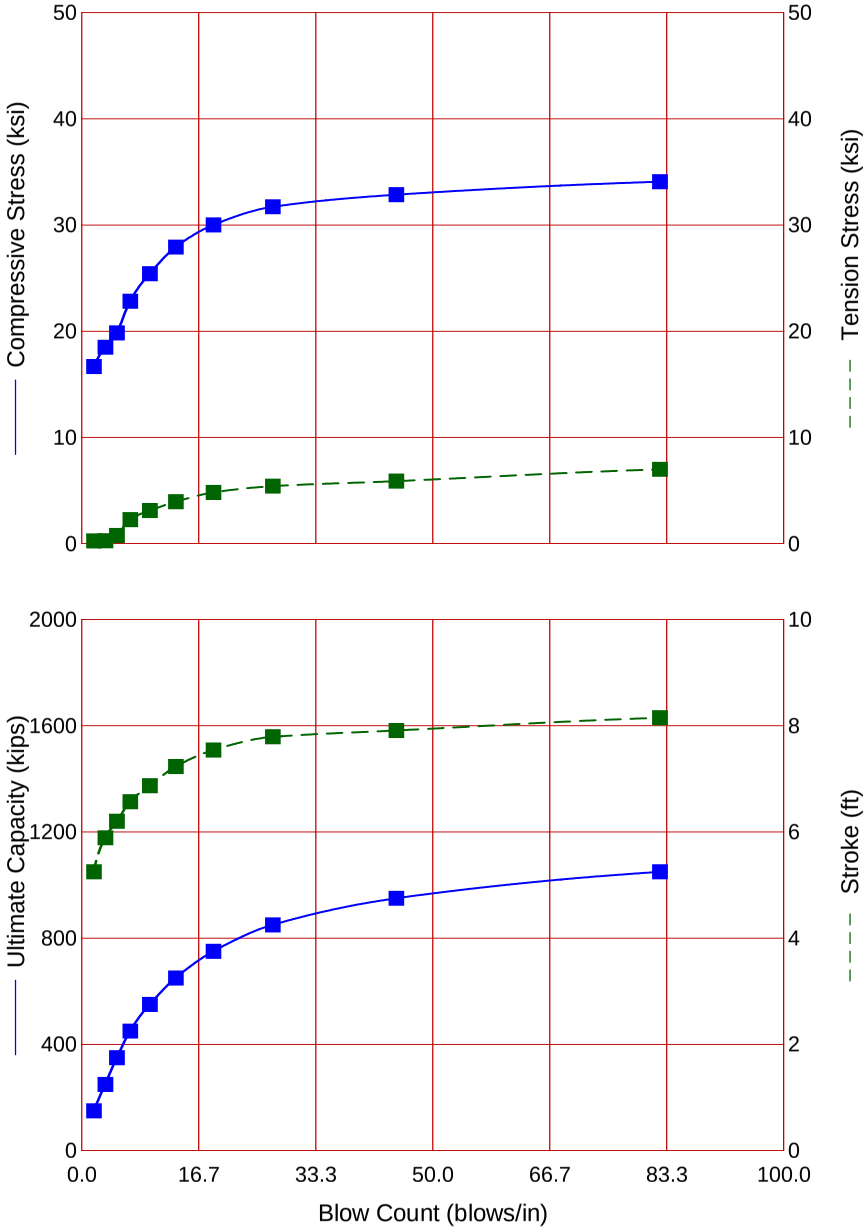
ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿
³           Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods           ³
³                                                                              ³
³ Project Name    : Wadsworth St. Bridge Client          : MaineDOT           ³
³ File Name       : Thomaston,ME         Project Manager : Blaisdell          ³
³ Date            : 05/09/13             Computed by     : ETL                ³
³                                                                              ³
³ Depth of Top of Pile =   0.00 ft.         Pile length          =  41.00 ft. ³
³ Depth to Water Table =   0.00 ft.                                            ³
³ Type of Pile         = H Pile                                                ³
³   HP 14x117                                                                  ³
³                                                                              ³
³                            SKIN FRICTION CONTRIBUTION                        ³
³                                                                              ³
³ Layer    Soil        Thickness    Effective   Internal   N-SPT      Pile    ³
³          Type                       Stress    Friction             Perimeter ³
³                         (ft)         (psf)      Angle                (ft)    ³
³                                                                              ³
³   1   Cohesionless       4.00        125.20     34.00      --        4.85   ³
³   2   Cohesionless      11.00        567.20     30.00      --        4.85   ³
³   3   Cohesive           7.00       1078.60       ---      --        4.85   ³
³   4   Cohesionless       7.00       1474.80     30.00      --        4.85   ³
³   5   Cohesionless      12.00       2082.00     38.00      --        4.85   ³
³                                                                              ³
³                                                                              ³
³ Layer    Soil       Undrained Shear  Adhesion  Pile    Sliding      Skin    ³
³          Type           Strength               Taper   Friction   Resistance ³
³                          (psf)                           Angle      (Kips)   ³
³                                                                              ³
³   1   Cohesionless         --        --------   ----    28.88        1.43   ³
³   2   Cohesionless         --        --------   ----    25.49       11.98   ³
³   3   Cohesive         1000.00        500.00    ----    -----       16.97   ³
³   4   Cohesionless         --        --------   ----    25.49       19.82   ³
³   5   Cohesionless         --        --------   ----    32.28      111.65   ³
³                                                                              ³
³                                                                              ³
³                                  Total Side Friction  :    161.84           ³
³                                                                              ³
³                            POINT RESISTANCE CONTRIBUTION                     ³
³                                                                              ³
³ Effective     Internal      SPT      Pile End     Bearing        End Bearing ³
³ Stress at     Friction     Value       Area       Capacity       Resistance  ³
³ pile Tip       Angle                              Factor                     ³
³   (psf)                              (ft*ft)        Nq             (Kips)    ³
³                                                                              ³
³  2487.60       38.00       -----       0.24       110.40           47.37    ³
³                                                                              ³
³                                Limiting End Bearing Resistance :    64.17    ³
³                                                                              ³
³                                                                              ³
³                                Ultimate Static Pile Capacity   :   209.21   ³
³                                                                              ³
ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ
```

Layer 5=2 ksf side resistance, seems too high
Meyerhof SPT Method qs=N/50 (ksf), say N=80
qs=1.6 ksf, Skin resistance=89.6 kips

89.6

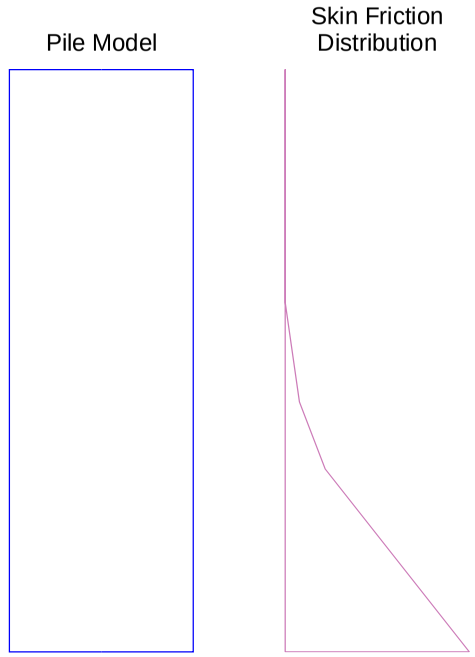
140 kips

140 kips/331 nominal = 42% side resistance



APE D 19-42

Ram Weight	4.19	kips
Efficiency	0.800	
Pressure	1247 (72%)	psi
Helmet Weight	3.00	kips
Hammer Cushion	34825	kips/in
COR of H.C.	0.920	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	35.00	ft
Pile Penetration	21.00	ft
Pile Top Area	34.40	in2

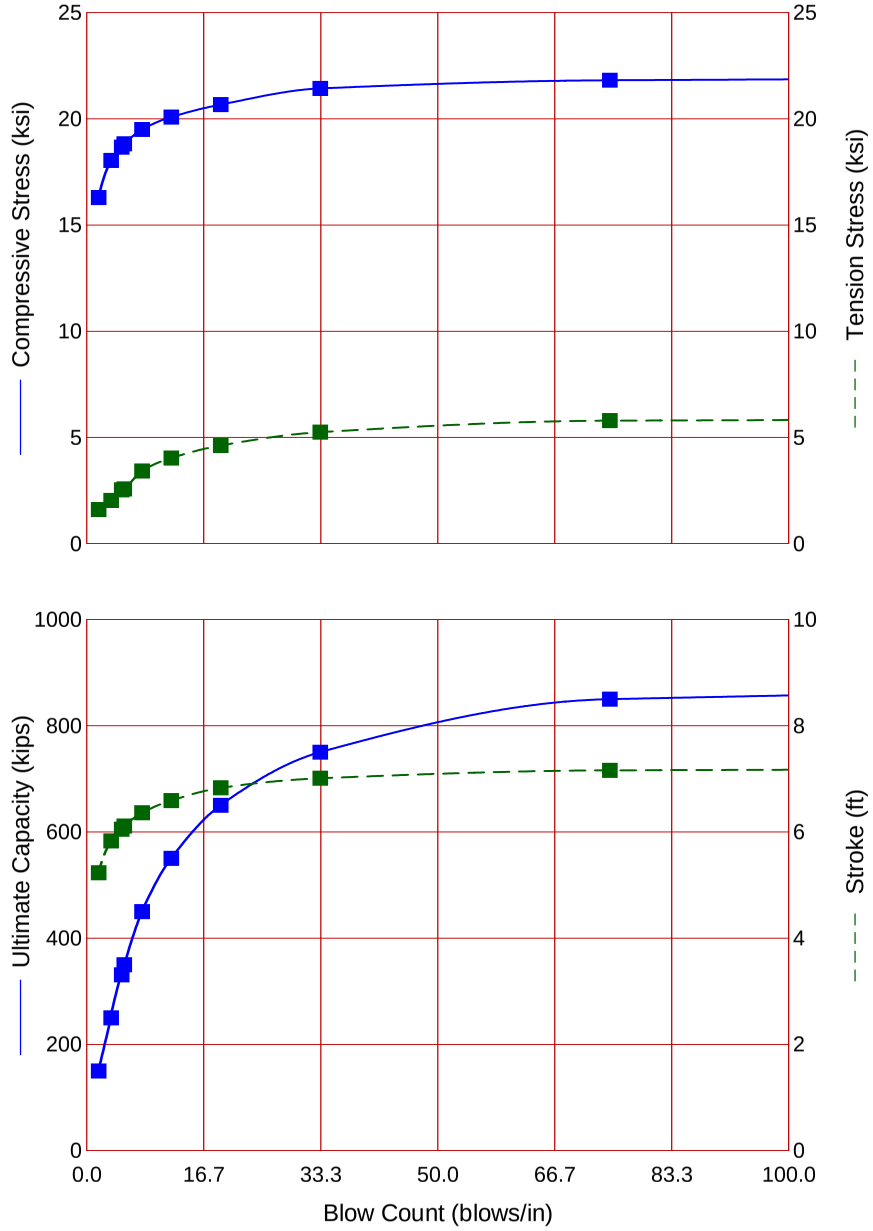


Res. Shaft = 28 %
(Proportional)

GZA Geo Environmental, Inc.
Wadsworth Street Bridge Replacement A1

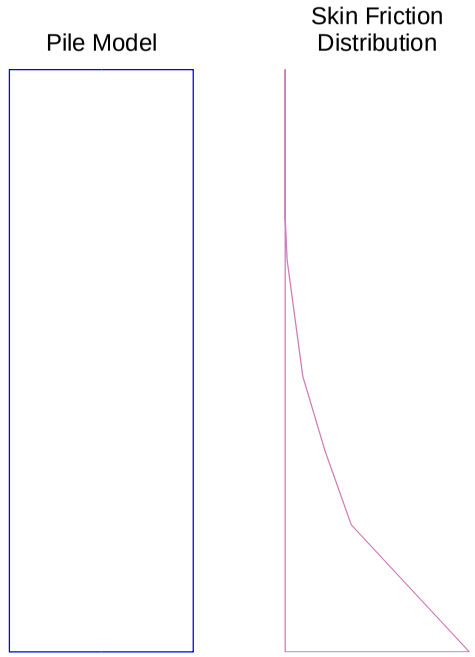
11-Sep-2014
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
150.0	16.67	0.26	1.8	5.25	12.98
249.0	18.49	0.28	3.4	5.89	12.86
350.0	19.83	0.77	5.0	6.20	12.90
450.0	22.82	2.26	6.9	6.57	13.46
550.0	25.41	3.11	9.7	6.87	14.07
650.0	27.92	3.95	13.4	7.23	14.99
750.0	30.00	4.82	18.8	7.54	15.78
850.0	31.71	5.42	27.2	7.79	16.39
950.0	32.85	5.89	44.8	7.91	16.67
1050.0	34.08	7.00	82.4	8.15	17.28



APE D 19-42

Ram Weight	4.19	kips
Efficiency	0.800	
Pressure	1247 (72%)	psi
Helmet Weight	3.00	kips
Hammer Cushion	34825	kips/in
COR of H.C.	0.920	
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	55.00	ft
Pile Penetration	41.00	ft
Pile Top Area	34.40	in2



Res. Shaft = 42 %
(Proportional)

GZA Geo Environmental, Inc.
Wadsworth Street Bridge Replacement A2

11-Sep-2014
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
150.0	16.29	1.61	1.7	5.23	13.14
250.0	18.04	2.03	3.5	5.83	13.22
331.0	18.66	2.53	5.0	6.05	13.14
350.0	18.82	2.58	5.4	6.11	13.25
450.0	19.49	3.42	7.9	6.36	13.84
550.0	20.08	4.03	12.1	6.59	14.38
650.0	20.66	4.62	19.1	6.83	14.93
750.0	21.43	5.25	33.3	7.01	15.35
850.0	21.81	5.79	74.5	7.16	15.73
950.0	22.42	6.21	601.2	7.32	16.10

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Engineers and Scientists

JOB: 09.0025781.00 Wadsworth Street
SUBJECT: Axial Capacity - Piers
SHEET: 1 OF 11
CALCULATED BY J. Baron 9/3/14
CHECKED BY A. Blaisdell, 9/8/14
REVIEWED BY

Objective

Evaluate the axial geotechnical resistance of the pier piles for the Wadsworth Street Bridge Replacement in Thomaston, ME

Methodology

Evaluate proposed pile section for governing factored axial compression resistance as follows.

1. Nominal Compressive Resistance
2. Factored Structural Compressive Resistance - Strength Limit State
3. Factored Structural Compressive Resistance - Extreme/Service Limit State
4. Geotechnical Resistance (Static Analysis)
5. Geotechnical Resistance (Drivability Analysis)
6. Factored Geotechnical Resistance - Strength Limit State
7. Factored Geotechnical Resistance - Extreme/Service Limit State

References

1. American Association of State Highway and Transportation Officials, AASHTO LRFD Bridge Design Specifications: Customary U.S. Units, 6th edition, 2012. (AASHTO LRFD)

Soil Properties

Consider Wadsworth Street Bridge Interpretive Subsurface Profile (see Figure 3). Piers 1 and 3 represent the shortest and longest pier piles, respectively. Therefore those profiles are suitable to cover the full range of site conditions; Pier 2 will not be analyzed. Subsurface layering and properties defined below. Total distance from bottom of cap to top of rock is approximately 30' at Pier 1 and 50' at Pier 3.

DESIGN SUBSURFACE PROFILE – PIER 1 (Boring BB-TSGR-301)			
Strata Designation	Thickness (feet)	Total Unit Weight (pcf)	Representative Φ' (°) / Su (psf) for layer
River Bottom	2	120	30º
Glacial Till	3	130	38º
Depth to Bedrock	5	--	--

DESIGN SUBSURFACE PROFILE – PIER 3 (Boring BB-TSGR-303)			
Strata Designation	Thickness (feet)	Total Unit Weight (pcf)	Representative Φ' (°) / Su (psf) for layer
River Bottom Deposit	6	120	30º
Silt/Clay	9	115	300 psf
River Bottom Deposit	4	120	30º
Glacial Till	7	130	38º
Bedrock	26	--	--

Structural Properties

PP24x0.625

Yield Strength of Steel $F_y := 50ksi$

Area of section (with 1/8" corrosion loss, outside of pipe) $A_s := 36.5in^2$

Young's Modulus of Steel $E_s := 30000 \cdot ksi$

Section Properties:

Moment of Intertia $I_x := 3143.5 \cdot in^4$

Section Modulus $S_x := 264.7in^3$

Radius of Gyration $r_s := 9.3in$

1. Nominal Structural Compressive Resistance P_n

The structural engineer will calculate nominal structural compressive resistance in accordance with AASHTO Section 4, 6, and 10, using the results of structural evaluations and the properties provided above.

2. Factored Structural Compressive Resistance Pr - Strength Limit State:

Factor for piles in compression under hard driving conditions:

From Article 6.5.4.2 $\phi_c := 0.6$

Factored Compressive Resistance for Strength Limit State:

$P_r := \phi_c \cdot P_n$ AASHTO Eq. 6.9.2.1-1 pg. 6-71

The structural engineer will calculate factored structural compressive resistance in accordance with AASHTO Section 4, 6, and 10, using the results of structural evaluations and the properties provided above.

3. Factored Structural Compressive Resistance - Service/Extreme Limit State:

Resistance Factors for Extreme Limit States:

From Article 10.5.5.1 and 10.5.5.3 $\phi := 1$

Factored Compressive Resistance for Service/Extreme Limit State:

$P_r := \phi \cdot P_n$ AASHTO Eq. 6.9.2.1-1 pg. 6-71

4. Geotechnical Axial Resistance - Static Analysis

AASHTO Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock is controlled by the structural limit state.

Used SPILE to estimate side friction resistance during driving using the Nordlund Thurman method, estimates 6 kips of side resistance for Pier 1 piles and 100 kips of side resistance for Pier 3 piles. SPILE output is attached on Sheets 6 and 7.

Required nominal resistance of 680 kips for design of Piers 1 and 3 based on a maximum factored load of 442 kips and a 0.65 resistance factor.

Estimated friction distribution and % shaft resistance using GRLWEAP based on design loads.

5. Geotechnical Axial Resistance - Drivability Analysis

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ AASHTO Eq. 10.7.8.1

$f_y := 50ksi$ yield Strength of steel (A252, Grade 3 Modified)

$\phi_{da} := 1.0$ AASHTO Table 10.5.5.2.3-1 Refers to Article 6.5.4.2, Pg. 6-28

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ $\sigma_{dr} = 45 \cdot ksi$ Driving Stress in pile cannot exceed 45 ksi

For drivability analysis, the uncorroded pile section area, As=45.9 in^2, is used.

Evaluate nominal geotechnical capacity using GRLWEAP.

Pier 1 - Drive pile through 5 feet of soil to rock with toe quake representative of very hard driving conditions (0.04in). Pile length is modeled as 40 feet with 35 feet of stickup.

Estimated skin friction is 1% of required nominal resistance. For short pile; driving stress is anticipated to control resistance.

Pier 3 - Drive pile through 26 feet of soil to rock with toe quake representative of moderate driving conditions (0.1 in). Pile length is modeled as 55 feet with 29 feet of stickup.

Estimated skin friction is 15% of required nominal resistance. For long pile, higher final blow count is anticipated to control

Drive piles with an APE D19-42 open-ended diesel hammer with a rated energy of 47,100 ft-lb. The maximum fuel setting is considered ; chamber pressure of 1710 psi (100%).

Allowable stress in piles during driving is 0.9 fy=45 ksi

GRLWEAP Output is attached on Sheets 8 through 11.

Pier 1:

$R_{ndr1} := 680kip$ Required nominal geotechnical resistance, pile driving stress=32ksi, final penetration resistance=7 bpi.

Pier 3:

$R_{ndr2} := 680kip$ Required nominal geotechnical resistance, pile driving stress=28 ksi, final penetration resistance=9 bpi.

6. Factored Drivability Resistance - Strength Limit State:

Strength Limit State Factored Drivability Resistance:

Pier 1:

$R_{ndr1_factored} := R_{ndr1} \cdot \phi_{dyn}$

$\phi_{dyn} := 0.65$ AASHTO Table 10.5.5.2.3-1 PDA, WEAP and CAPWAP used to establishing driving criteria

$R_{ndr1_factored} := R_{ndr1} \cdot \phi_{dyn}$

$R_{ndr1_factored} = 442 \cdot kip$

Pier 3:

$R_{ndr2_factored} := R_{ndr2} \cdot \phi_{dyn}$

$R_{ndr2_factored} = 442 \cdot kip$

7. Factored Drivability Resistance - Service/Extreme Limit States:

Service and Extreme Limit State Factored Drivability Resistance:

Resistance Factors for Extreme Limit States:

From Article 10.5.5.1 and 10.5.5.3 $\phi_{serv_ext} := 1$

$R_{ndr1_serv_ext} := R_{ndr1} \cdot \phi_{serv_ext}$

$R_{ndr1_serv_ext} = 680 \cdot kip$

$R_{ndr2_serv_ext} := R_{ndr2} \cdot \phi_{serv_ext}$

$R_{ndr2_serv_ext} = 680 \cdot kip$

Summary of Results - Axial Loading:

ASTM A252 Grade 3, PP24x0.625	Geotechnical Static Resistance (kips)	Drivability Resistance (kips)
Pier 1: Strength Limit State, Design	n/a	442
Pier 1: Service/Extreme Limit State, Design	n/a	680
Pier 3: Strength Limit State, Design	n/a	442
Pier 3: Service/Extreme Limit State, Design	n/a	680

Results indicate APE D19-42 is suitable to penetrate overburden and achieve geotechnical strength limit state resistance.

Structural engineer will assess nominal compressive resistance to assess whether it controls.

PIER1.TXT

```
ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿
³           Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods           ³
³                                                                              ³
³ Project Name   : Wadsworth St. Bridge Client          : maineDOT             ³
³ File Name      : Thomaston,ME         Project Manager : Blaisdell            ³
³ Date           : 05/09/13             Computed by     : ETL                  ³
³                                                                              ³
³ Depth of Top of Pile =   0.00 ft.         Pile length          =   5.00 ft.  ³
³ Depth to Water Table =   0.00 ft.                                            ³
³ Diameter of pile     =  24.00 in.                                            ³
³ Type of Pile         = Pipe Pile                                             ³
³                                                                              ³
³                                                                              ³
³                                                                              ³
³                            SKIN FRICTION CONTRIBUTION                        ³
³                                                                              ³
³ Layer    Soil        Thickness    Effective   Internal   N-SPT      Pile    ³
³          Type                       Stress    Friction             Perimeter ³
³                         (ft)        (psf)      Angle                (ft)     ³
³                                                                              ³
³   1   Cohesionless       2.00        57.60     30.00      --        6.28    ³
³   2   Cohesionless       3.00       216.60     38.00      --        6.28    ³
³                                                                              ³
³                                                                              ³
³ Layer    Soil       Undrained Shear  Adhesion  Pile    Sliding       Skin   ³
³          Type           Strength               Taper   Friction   Resistance ³
³                          (psf)                           Angle       (Kips)  ³
³                                                                              ³
³   1   Cohesionless         --        --------   ----    23.94         0.36  ³
³   2   Cohesionless         --        --------   ----    30.32         5.57  ³
³                                                                              ³
³                                                                              ³
³                                          Total Side Friction  :      5.93    ³
³                                                                              ³
³                         POINT RESISTANCE CONTRIBUTION                        ³
³                                                                              ³
³ Effective     Internal      SPT      Pile End     Bearing        End Bearing ³
³ Stress at     Friction     Value       Area       Capacity       Resistance  ³
³ pile Tip       Angle                              Factor                     ³
³   (psf)                              (ft*ft)        Nq             (Kips)    ³
³                                                                              ³
³   318.00       38.00       -----       3.14       110.40            79.63   ³
³                                                                              ³
³                                Limiting End Bearing Resistance :   843.83    ³
³                                                                              ³
³                                                                              ³
³                                Ultimate Static Pile Capacity   :    85.56    ³
³                                                                              ³
ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ
```

6 kips/680 kips nominal=1% friction

PIER3.TXT

```
ÚÄÄÄÄÄÄÄÄ ULTIMATE STATIC PILE CAPACITY/Federal Highway Administration ÄÄÄÄÄÄÄÄ¿
³           Nordlund (1963, 1979) and Tomlinson (1979, 1980) methods           ³
³                                                                              ³
³ Project Name   : Wadsworth St. Bridge Client          : maineDOT             ³
³ File Name      : Thomaston,ME         Project Manager : Blaisdell            ³
³ Date           : 05/09/13             Computed by     : ETL                  ³
³                                                                              ³
³ Depth of Top of Pile =   0.00 ft.         Pile length          =  26.00 ft.  ³
³ Depth to Water Table =   0.00 ft.                                            ³
³ Diameter of pile     =  24.00 in.                                            ³
³ Type of Pile         = Pipe Pile                                             ³
³                                                                              ³
³                                                                              ³
³                                                                              ³
³                            SKIN FRICTION CONTRIBUTION                        ³
³                                                                              ³
³ Layer    Soil        Thickness    Effective   Internal   N-SPT      Pile    ³
³          Type                       Stress    Friction             Perimeter ³
³                         (ft)         (psf)      Angle                (ft)    ³
³                                                                              ³
³   1   Cohesionless       6.00        172.80     30.00      --        6.28   ³
³   2   Cohesive           9.00        582.30       ---      --        6.28   ³
³   3   Cohesionless       4.00        934.20     30.00      --        6.28   ³
³   4   Cohesionless       7.00       1286.00     38.00      --        6.28   ³
³                                                                              ³
³                                                                              ³
³ Layer    Soil       Undrained Shear  Adhesion  Pile    Sliding       Skin   ³
³          Type           Strength               Taper   Friction   Resistance ³
³                          (psf)                           Angle       (Kips)  ³
³                                                                              ³
³   1   Cohesionless         --        --------   ----    23.94         3.22  ³
³   2   Cohesive          300.00        150.00    ----    -----         8.48  ³
³   3   Cohesionless         --        --------   ----    23.94        11.62  ³
³   4   Cohesionless         --        --------   ----    30.32        77.13  ³
³                                                                              ³
³                                                                              ³
³                                          Total Side Friction  :    100.46   ³
³                                                                              ³
³                          POINT RESISTANCE CONTRIBUTION                       ³
³                                                                              ³
³ Effective     Internal      SPT      Pile End     Bearing        End Bearing ³
³ Stress at     Friction     Value       Area       Capacity       Resistance  ³
³ pile Tip       Angle                              Factor                     ³
³   (psf)                              (ft*ft)        Nq             (Kips)    ³
³                                                                              ³
³  1522.60       38.00       -----       3.14       110.40           381.28   ³
³                                                                              ³
³                                 Limiting End Bearing Resistance :   843.83   ³
³                                                                              ³
³                                                                              ³
³                                 Ultimate Static Pile Capacity   :   481.74   ³
³                                                                              ³
ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ
```

100 kips/680 kips nominal=15% friction

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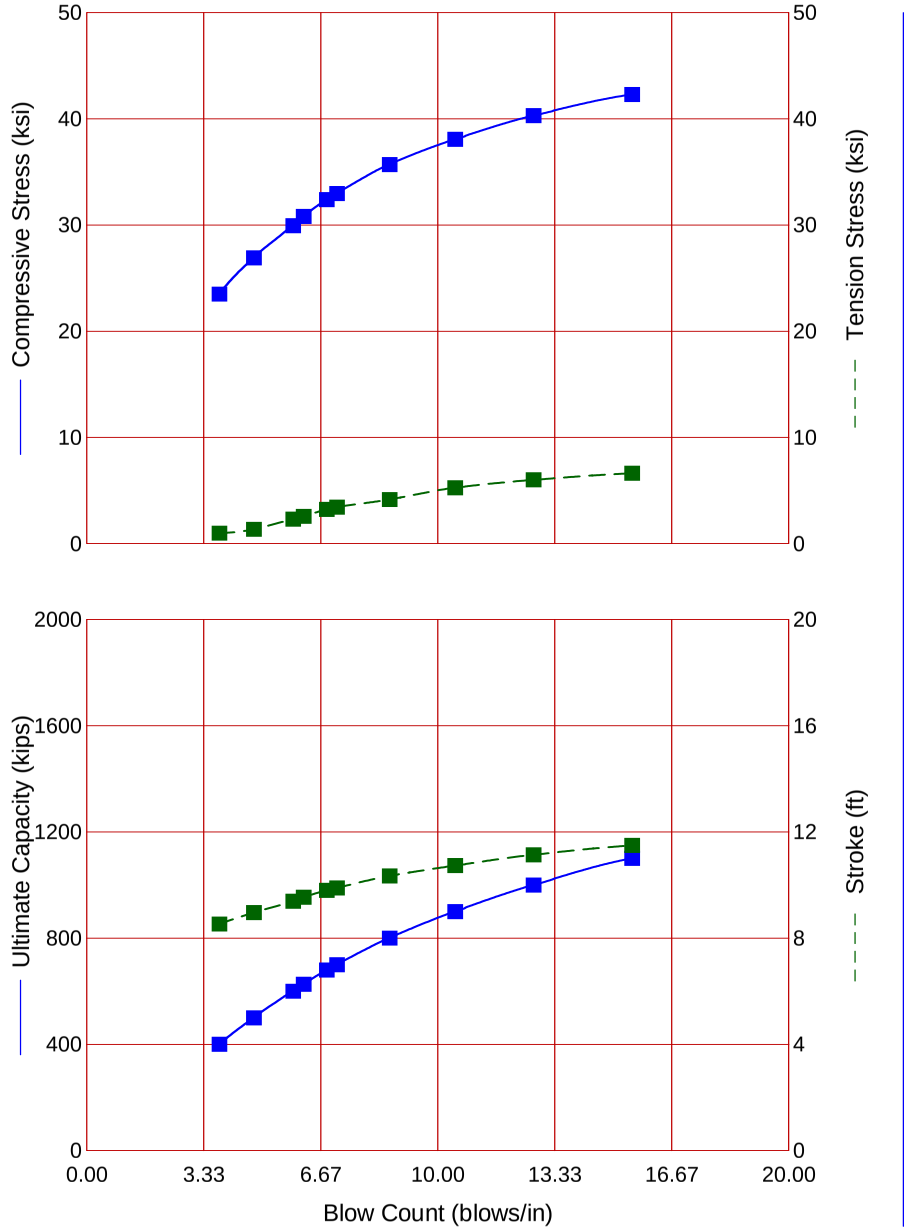
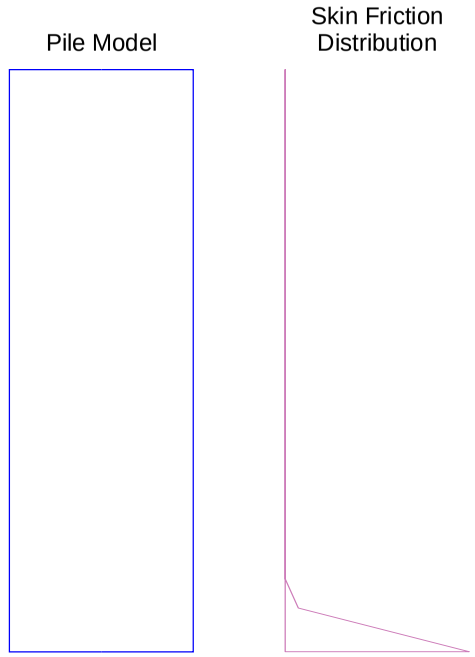
Thomaston, ME Pier 1

11-Sep-2014
GRLWEAP Version 2010



APE D 19-42

Ram Weight	4.19	kips
Efficiency	0.800	
Pressure	1710 (100%)	psi
Helmet Weight	4.00	kips
Hammer Cushion	42875	kips/in
COR of H.C.	0.920	
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	40.00	ft
Pile Penetration	5.00	ft
Pile Top Area	45.89	in2



Res. Shaft = 1 %
(Proportional)

GZA Geo Environmental, Inc.
Thomaston, ME Pier 1

11-Sep-2014
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	23.49	0.99	3.8	8.53	20.46
500.0	26.90	1.35	4.8	8.96	21.17
600.0	29.93	2.31	5.9	9.39	22.10
626.0	30.80	2.57	6.2	9.54	22.49
680.0	32.39	3.23	6.8	9.80	23.22
700.0	32.95	3.44	7.1	9.89	23.43
800.0	35.68	4.15	8.6	10.34	24.67
900.0	38.05	5.26	10.5	10.73	25.73
1000.0	40.29	6.00	12.7	11.14	26.84
1100.0	42.28	6.63	15.5	11.49	27.81

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Thomaston, ME

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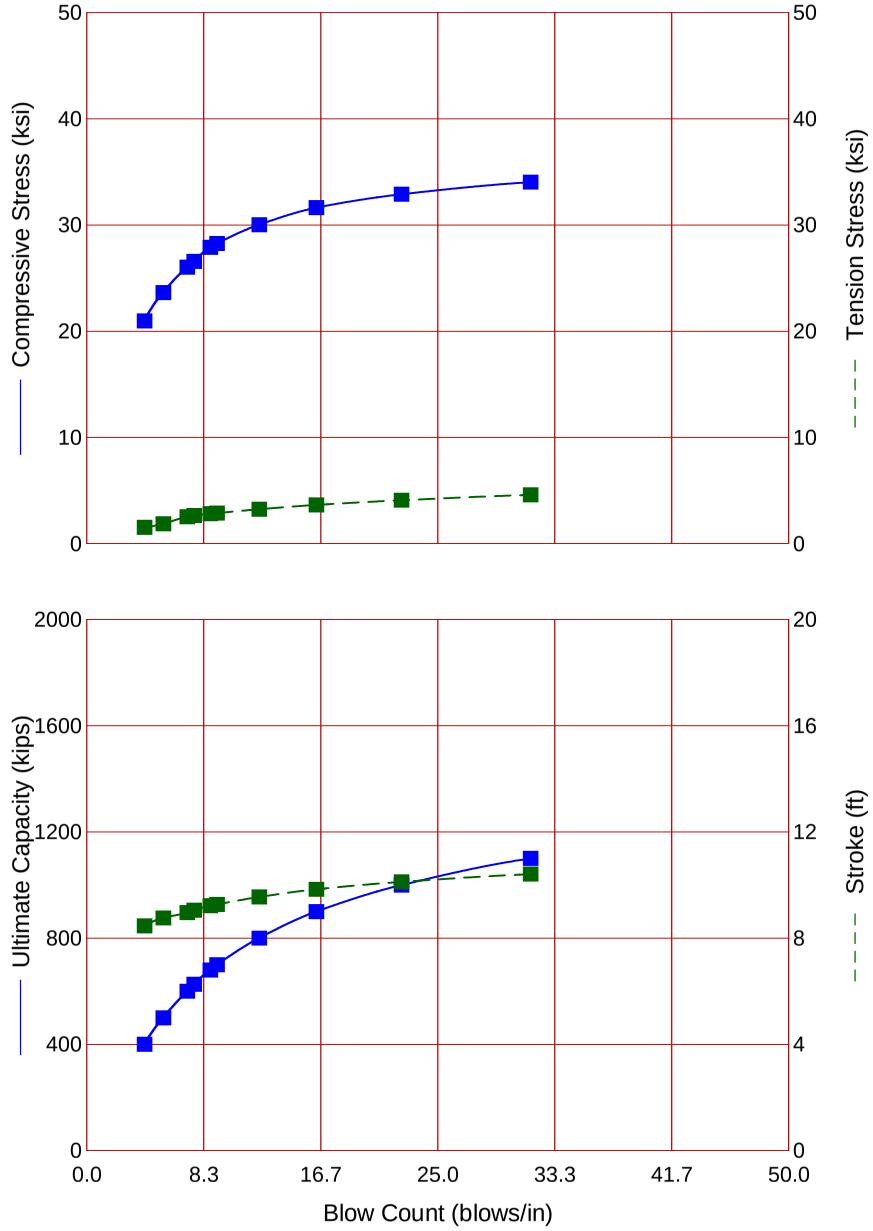
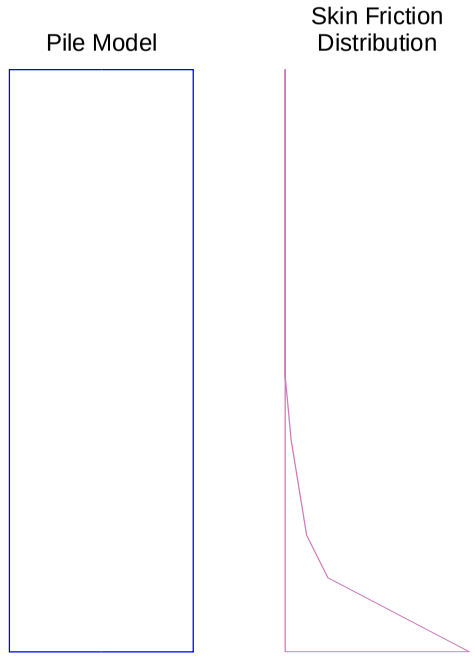
GRLWEAP Version 2010



APE D 19-42

Ram Weight	4.19	kips
Efficiency	0.800	
Pressure	1710 (100%)	psi
Helmet Weight	4.00	kips
Hammer Cushion	42875	kips/in
COR of H.C.	0.920	
Skin Quake	0.100	in
Toe Quake	0.100	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	55.00	ft
Pile Penetration	26.00	ft
Pile Top Area	45.89	in2



Res. Shaft = 15 %
(Proportional)

GZA Geo Environmental, Inc.
Thomaston, ME

11-Sep-2014
GRLWEAP Version 2010

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke ft	Energy kips-ft
400.0	20.97	1.53	4.1	8.46	21.00
500.0	23.64	1.87	5.5	8.76	21.48
600.0	26.03	2.54	7.2	8.96	22.02
626.0	26.58	2.65	7.7	9.05	22.25
680.0	27.89	2.82	8.8	9.22	22.72
700.0	28.26	2.88	9.3	9.27	22.87
800.0	30.02	3.24	12.3	9.55	23.58
900.0	31.62	3.65	16.4	9.84	24.43
1000.0	32.89	4.09	22.4	10.12	25.19
1100.0	34.04	4.59	31.6	10.41	25.98

Objective

Evaluate the performance of the proposed bent pier foundations under design axial and lateral loads. Assess effective point of fixed/pinned condition for use in structural analyses. The piers are assumed to be supported using five, PP24x0.625 pipe piles, including two plumb and three battered piles, driven to practical refusal on bedrock.

Results Summary

The piles were found to achieve a pinned condition under Extreme II loading near the bottom of the pile. Therefore, it is recommended that a pinned condition be assumed at the pile tip (El. -38 at Pier 2 and El. -40 at Pier 3).

Approach and Input Parameters

1. Soil and Rock Parameters- L-Pile Typ.

Using data from test borings and laboratory testing, GZA developed representative subsurface profiles and engineering properties for soil and bedrock. Subsurface profiles are presented for each on Sheet 6. Soil engineering properties, soil models and LPile parameters, presented below, were selected to model the shafts in GROUP.

Pier 2 (Borings: BB-TSGR-102, BB-TSGR-203 and BB-TSGR-302)					
Stratum	**Soil Model**	**Layer Thickness (ft)**	**k (pci)**	**ϕ' (deg)**	**γ_e (pci)**
River Bottom	Reese Sand	14	20	30	0.033
Till	Reese Sand	5	90	38	0.036
Notes: Depth to mudline from top of pile = 28 ft.					

Pier 3 (Borings: BB-TSGR-204 and BB-TSGR-303)					
Stratum	**Soil Model**	**Layer Thickness (ft)**	**k (pci) / E50**	**ϕ' (deg) / Su (psf)**	**γ_e (pci)**
River Bottom Deposit	Reese Sand	6	20	30	0.033
Silt/Clay	Soft Clay	9	E_{50} = 0.02	300	0.03
River Bottom Deposit	Reese Sand	4	20	30	0.033
Till	Reese Sand	7	90	38	0.036
Notes: Depth to mudline from top of pile = 23 ft.					

2. Basis of Evaluation

GZA modeled the bent pier performance with the intent of assessing pile fixity. The evaluation was not intended to calculate actual pile bent deflection and stress, as several considerations were neglected, including concrete fill in the piles and beneficial resistance of the bridge deck to lateral deflection. These assumptions lead to a conservative model of the actual condition.

The analytical software GROUP was used to model the bent piers under design loading. Piers 2 and 3 were selected for evaluation, as Pier 1 was judged to require rock anchors to fix the pile toe due to very thin overburden.

3. Pile Bent Properties

GZA modeled the PP24x0.625 piles as hollow members (not concrete filled), which is conservative given that the piles will be filled with unreinforced concrete. We modeled two conditions: an uncorroded pile section, and a section including 1/8" of section loss around the outsider perimeter of the piles, extending from the bottom of pile cap to 5' below mudline. A cross section showing bent pier geometry is shown on Sheet 4.

4. Design Loads

Bent pier loads were taken from load combination summary sheets provided by Calderwood Engineering, dated 3/4/14. Based on our review of the load combinations and discussion with Calderwood, two load cases were selected for analysis: Service I and Extreme II (Ice loading). The Extreme II loads were judged to control for the factored load cases. Pier 3 reportedly had slightly larger loads the Pier 2, so Pier 3 loads were assumed for both cases. The design load diagrams are presented in Sheets 4 and 5.

Analysis

1. Uncorroded Section

GROUP analyses were initially conducted for Piers 2 and 3, Extreme II and Service I, for the full, uncorroded pile section. GROUP output files and plots of deflection, shear and moment vs. depth are presented on Sheets 6 through 22 (Pier 2, Extreme and Service) and 23 through 35 (Pier 3, Exterme only). The results indicated the following:

a. The toe of the piles did not gain "fixity", defined as two points of inflection in the pile deflection curve.
b. The toe of the piles did achieve a "pinned" condition, defined as one point of inflection (i.e., deflection crosses the y-axis from positive to negative once.
c. Pier 2 exhibited more negative tip deflection than Pier 2, therefore, Pier 3 was closer to fixed and Pier 2 controls lateral pile performance.
d. Pier 2, Extreme II load case resulted in the largest deflection (1.2") and maximum pile stress (21 ksi).
e. Pier 2, Service I load case resulted in much smaller maximum deflection (0.4") and maximum pile stress (10 ksi).

2. Corroded Section

Subsequent GROUP analyses were conducted for Pier 2, Extreme II and Service I, for the corroded pile section described above. GROUP output files and plots of deflection, shear and moment vs. depth are presented on Sheets 36 through 55. The results indicated similar results as the uncorroded section, summarized below:

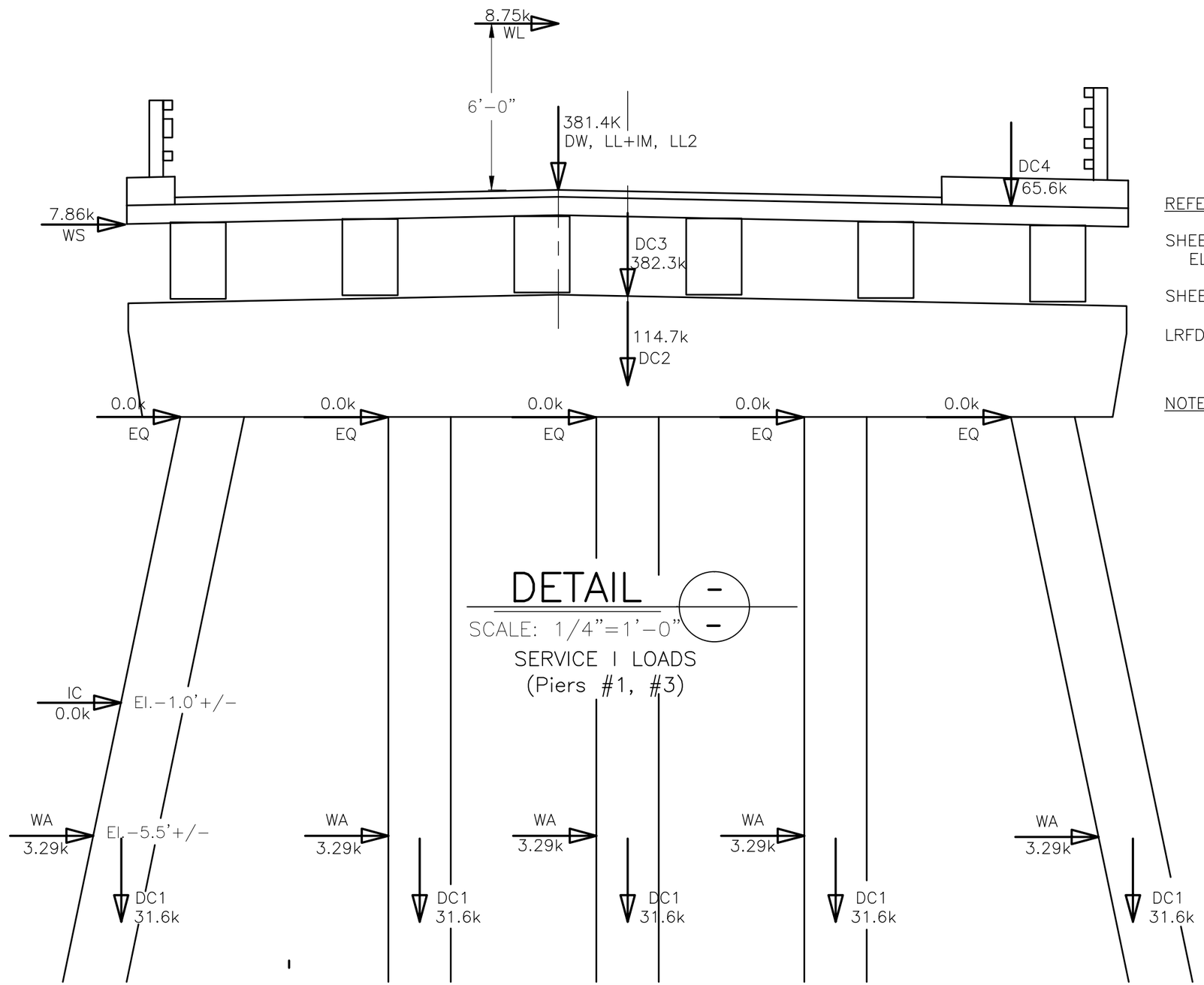
a. The toe of the piles achieved a "pinned" condition, defined as one point of inflection (i.e., deflection crosses the y-axis from positive to negative once, at a similar depth as the uncorroded case, a few feet above the bottom of the pile.
d. Pier 2, Extreme II load case maximum deflection and pile stress increased to 1.4" and 26 ksi, respectively.
e. Pier 2, Service I load case maximum deflection and pile stress were very similar to the uncorroded section, 0.4" and 12 ksi, respectively.

Conclusion

Based on the results, a pinned pile condition (pinned by the soil reaction) should be assumed to occur at the pile tip, corresponding to El. -38 at Pier 2 and El. -40 at Pier 3.

PROJECT: Wadsworth St Bridge, Thomaston
SUBJECT: Pier Loads
ITEM: SERVICE I LOADS

DATE: 03/04/14
BY: RG
SHEET: 4 OF 9999
PROJECT NO. 083-br-12



DETAIL
SCALE: 1/4"=1'-0"
SERVICE I LOADS
(Piers #1, #3)

REFERENCES:

SHEET 1: PIER GEOMETRY, DIMENSIONS, ELEVATIONS

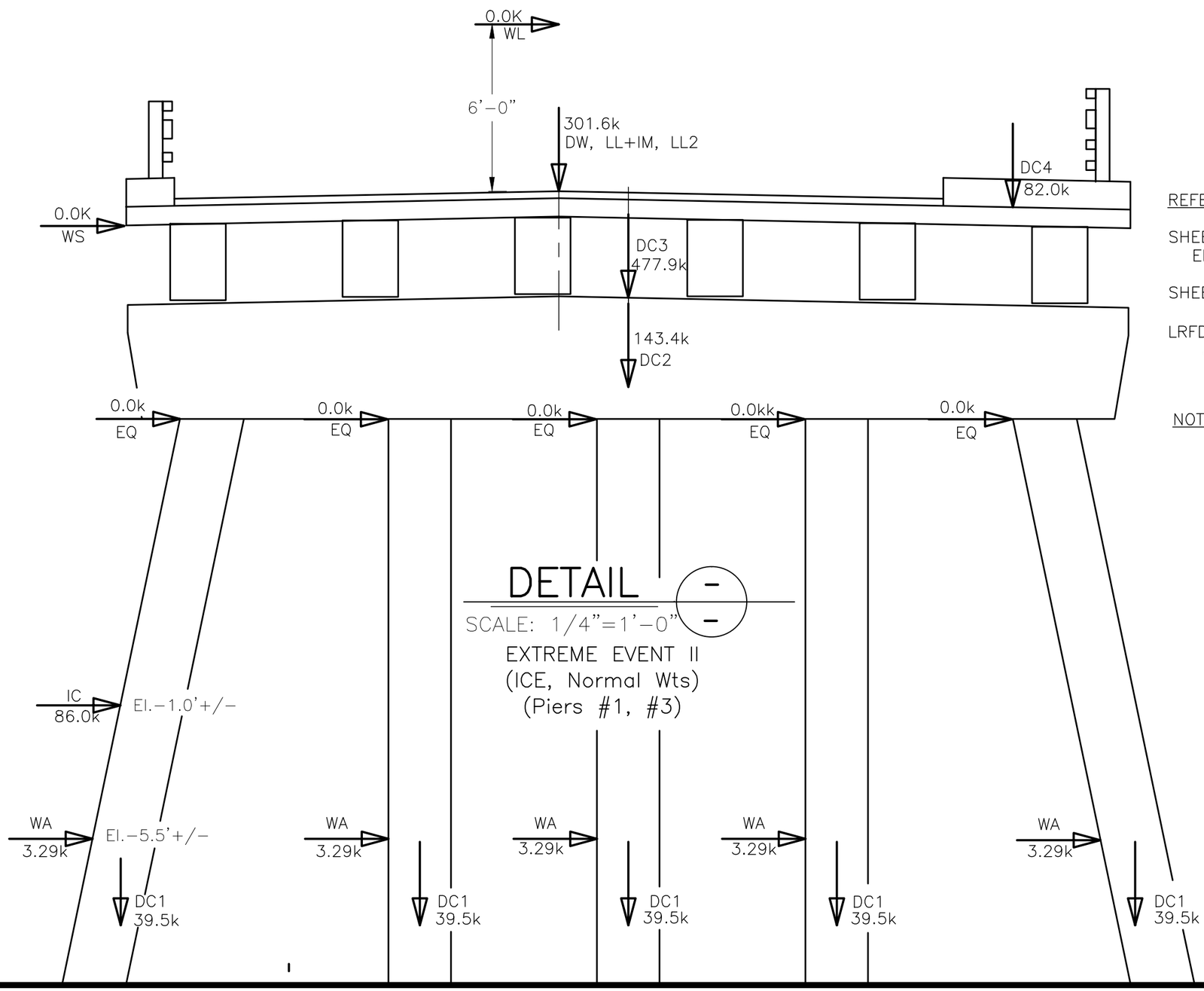
SHEET 2: PIIER LOADS LEGEND

LRFD/TABLE 3.4.1-1: LOAD COMBINATIONS & FACTORS

NOTE: Loads for Pier #2 are lower than for Piers #1, #3

PROJECT: Wadsworth St Bridge, Thomaston
SUBJECT: Pier Loads
ITEM: EXTREME EVENT II (ICE) LOADS

DATE: 03/04/14
BY: RG
SHEET: 7 OF 9999
PROJECT NO. 083-br-12



DETAIL
SCALE: 1/4"=1'-0"
EXTREME EVENT II
(ICE, Normal Wts)
(Piers #1, #3)

REFERENCES:

SHEET 1: PIER GEOMETRY, DIMENSIONS, ELEVATIONS

SHEET 2: PIIER LOADS LEGEND

LRFD/TABLE 3.4.1-1: LOAD COMBINATIONS & FACTORS

NOTE: Loads for Pier #2 are lower than for Piers #1, #3

```
==========================================================================

                 GROUP for Windows, Version 8.0.15

                  Serial Number :  364300562

                  Analysis of A Group of Piles
               Subjected to Axial and Lateral Loading

                (c) Copyright ENSOFT, Inc., 1987-2010
                      All Rights Reserved

==========================================================================
```

Pier 2
PP24x0.625 piles
Uncorroded Section
No concrete fill included
Extreme II and Service I Load Cases

```
This program is licensed to :

gza
GZA GeoEnvironmental, Inc.

Path to file locations      : P:\09 Jobs\0025700s\09.0025781.00 Thomaston Bridge MaineDOT\Work\Calcs\GROUP\
Name of input data file     : 25781 5 pile Bent Pier 2.gp8d
Name of output file         : 25781 5 pile Bent Pier 2.gp8o
Name of output summary file : 25781 5 pile Bent Pier 2.gp8t
Name of plot output file    : 25781 5 pile Bent Pier 2.gp8p
Name of runtime file        : 25781 5 pile Bent Pier 2.gp8r


--------------------------------------------------------------------------
                     Time and Date of Analysis
--------------------------------------------------------------------------

          Date:  March 25, 2014     Time:  11:51:34




           *****     INPUT INFORMATION     *****


Thomaston

ANALYSIS TYPE = 3D ANALYSIS

UNITS SYSTEM  = ENGL


* TABLE C *  LOAD AND CONTROL PARAMETERS


** LOAD CASES **
```

NUMBER OF LOAD CASES : 2

LOAD CASE : 1
CASE NAME : extreme II
LOAD TYPE : Dead, DL
SCALE FACTOR : 1.0000

* CONCENTRATED LOADS *

NL	VERT.LOAD LBS	HR.LOAD Y LBS	HR.LOAD Z LBS	MOMENT X LBS-IN	MOMENT Y LBS-IN	MOMENT Z LBS-IN	COORD X IN	COORD Y IN	COORD Z IN
1	39500.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-180.000	0.00000
2	39500.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-90.0000	0.00000
3	39500.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
4	39500.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	90.0000	0.00000
5	39500.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	180.000	0.00000
6	6.21300E+05	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
7	3.01600E+05	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-30.0000	0.00000
8	82000.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	165.000	0.00000
9	0.00000	86000.0	0.00000	0.00000	0.00000	0.00000	120.000	-212.000	0.00000
10	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	-215.000	0.00000
11	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	-90.0000	0.00000
12	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	0.00000	0.00000
13	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	90.0000	0.00000
14	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	215.000	0.00000

* EQUIVALENT CONC. LOAD AT ORIGIN *

VER.LOAD X,LBS	HOR.LOAD Y,LBS	HOR.LOAD Z,LBS
1.20240E+06	1.02450E+05	0.00000

MOMENT X,LBS-IN	MOMENT Y,LBS-IN	MOMENT Z,LBS-IN
0.00000	0.00000	8.79900E+06

* THE LOADING IS STATIC *

* CONTROL PARAMETERS *
TOLERANCE ON CONVERGENCE OF FOUNDATION REACTION, = 1.00000E-04
TOLERANCE ON DETERMINATION OF DEFLECTIONS = 1.00000E-04 IN
MAX NO OF ITERATIONS ALLOWED FOR FOUNDATION ANALYSIS = 100
MAXIMUM NO OF ITERATIONS ALLOWED FOR PILE ANALYSIS = 100
FACTOR TO APPLY THE LOAD IN INCREMENTS = 1.0000
MINIMUM FACTOR FOR LOAD INCREMENTS = 1.0000
PRINT RESULTS ONLY AT PILE CAP

LOAD CASE : 2
CASE NAME : Service I
LOAD TYPE : Dead, DL
SCALE FACTOR : 1.0000

* CONCENTRATED LOADS *

NL	VERT.LOAD LBS	HR.LOAD Y LBS	HR.LOAD Z LBS	MOMENT X LBS-IN	MOMENT Y LBS-IN	MOMENT Z LBS-IN	COORD X IN	COORD Y IN	COORD Z IN
1	31600.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-180.000	0.00000
2	31600.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-90.0000	0.00000
3	31600.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
4	31600.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	90.0000	0.00000
5	31600.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	180.000	0.00000
6	4.97000E+05	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
7	3.81400E+05	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-30.0000	0.00000
8	65600.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	165.000	0.00000
9	0.00000	8750.00	0.00000	0.00000	0.00000	0.00000	-168.000	-30.0000	0.00000
10	0.00000	7860.00	0.00000	0.00000	0.00000	0.00000	-84.0000	-225.000	0.00000
11	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	-225.000	0.00000
12	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	-90.0000	0.00000
13	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	0.00000	0.00000
14	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	90.0000	0.00000
15	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	225.000	0.00000

```
* EQUIVALENT CONC. LOAD AT ORIGIN *

      VER.LOAD X,LBS     HOR.LOAD Y,LBS     HOR.LOAD Z,LBS
         1.10200E+06          33060.0           0.00000

      MOMENT X,LBS-IN    MOMENT Y,LBS-IN    MOMENT Z,LBS-IN
             0.00000          0.00000       1.44876E+06


* THE LOADING IS STATIC *


* CONTROL PARAMETERS *
    TOLERANCE ON CONVERGENCE OF FOUNDATION REACTION,     =  1.00000E-04
    TOLERANCE ON DETERMINATION OF DEFLECTIONS            =  1.00000E-04 IN
    MAX NO OF ITERATIONS ALLOWED FOR FOUNDATION ANALYSIS =      100
    MAXIMUM NO OF ITERATIONS ALLOWED FOR PILE ANALYSIS   =      100
    FACTOR TO APPLY THE LOAD IN INCREMENTS               =   1.0000
    MINIMUM FACTOR FOR LOAD INCREMENTS                   =   1.0000
    PRINT RESULTS ONLY AT PILE CAP


* TABLE D *   ARRANGEMENT OF PILE GROUPS
```

GROUP	CONNECTY	CONNECTZ	PILE PROP	P-Y CURVE	L-S CURVE	T-R CURVE
1	FIX	FIX	1	0	0	0
2	FIX	FIX	1	0	0	0
3	FIX	FIX	1	0	0	0
4	FIX	FIX	1	0	0	0
5	FIX	FIX	1	0	0	0

GROUP	CorX,IN	CorY,IN	CorZ,IN	ALPHA,DEG	BETA,DEG	GROUND,IN	SPR.y,LBS-IN	SPR.z,LBS-IN
1	0.00000	-180.000	0.00000	0.00000	101.770	336.000	0.00000	0.00000
2	0.00000	-90.0000	0.00000	0.00000	90.0000	336.000	0.00000	0.00000
3	0.00000	0.00000	0.00000	0.00000	90.0000	336.000	0.00000	0.00000

```
   4        0.00000      90.0000      0.00000      0.00000      90.0000      336.000      0.00000      0.00000
   5        0.00000      180.000      0.00000      0.00000      78.2300      336.000      0.00000      0.00000


* TABLE E *   PILE GEOMETRY AND PROPERTIES
             PILE TYPE = 1 - DRIVEN PILE
                       = 2 - DRILLED SHAFT

  PROP   SECTS   INC     PILE TYPE      LENGTH, IN        E,LBS/IN**2
    1      1     100          1            564.00        2.9000E+07

* PILE SECTIONS *

  PROP    SECT      FROM,IN         TO,IN    CROSS SECT
    1       1        0.00000        564.000       1

* PILE CROSS SECTIONS *

 CROSS SECTION       :     1
 SECTION NAME        : Section
 CROSS SECTION TYPE  : CIRCULAR PIPE
 EXTERNAL DIAMETER   :       24.0000 IN
 INTERNAL DIAMETER   :       22.7500 IN
 SHEAR MODULUS       :    1.0000     LBS/IN**2
 HEAR MODULUS       : ,      1.00000 LBS/IN**2

* PILE CROSS SECTIONS PROPERTIES *

  SECT      DIAM,IN     AREA,IN**2     Iz,IN**4      Iy,IN**4      GJ,LBS-IN**2     Mn,LBS-IN      Vn,LBS
    1        24.0000       45.8967       3136.93       3136.93     7.27768E+10       0.00000       0.00000


* TABLE F *   SOIL DATA


SOILS INFORMATION

    GROUND SURFACE          =      336.000  IN

    3 LAYER(S) OF SOIL

    LAYER  1
    THE SOIL IS A SAND
```

River Bottom

```
                                                   TOP OF LAYER    BOTTOM OF LAYER
    X COORDINATE                          (IN)        336.000          504.000
    EFFECTIVE UNIT WEIGHT         (LBS/IN**3)      0.0333000        0.0333000
    FRICTION ANGLE                  (DEGREES)        30.0000          30.0000
    P-Y SUBGRADE MODULUS          (LBS/IN**3)        20.0000          20.0000
    ULTIMATE UNIT SIDE FRICTION   (LBS/IN**2)        0.00000          0.00000
    ULTIMATE UNIT TIP RESISTANCE  (LBS/IN**2)        0.00000          0.00000

    LAYER  2
    THE SOIL IS A SAND
```

Till

```
                                                   TOP OF LAYER    BOTTOM OF LAYER
```

```
X COORDINATE                          (IN)        504.000          564.000
EFFECTIVE UNIT WEIGHT         (LBS/IN**3)      0.0360000        0.0360000
FRICTION ANGLE                  (DEGREES)        38.0000          38.0000
P-Y SUBGRADE MODULUS          (LBS/IN**3)        90.0000          90.0000
ULTIMATE UNIT SIDE FRICTION   (LBS/IN**2)        0.00000          0.00000
ULTIMATE UNIT TIP RESISTANCE  (LBS/IN**2)        0.00000          0.00000

LAYER  3
THE LAYER IS A VUGGY LIMESTONE
```

Rock (Tip of Pile at Top of Rock)

```
                                                TOP OF LAYER    BOTTOM OF LAYER
X COORDINATE                          (IN)        564.000          600.000
EFFECTIVE UNIT WEIGHT         (LBS/IN**3)      0.0560000        0.0560000
UNIAXIAL COMPRESSIVE STRENGTH (LBS/IN**2)        4000.00          4000.00
ULTIMATE UNIT SIDE FRICTION   (LBS/IN**2)        0.00000          0.00000
ULTIMATE UNIT TIP RESISTANCE  (LBS/IN**2)        17000.0          17000.0

Note :  Default values will be generated for
        ULTIMATE UNIT SIDE FRICTION  and ULTIMATE UNIT TIP RESISTANCE
        when input values are 0.


* TABLE H *   AXIAL LOAD VS SETTLEMENT

LOAD-SETTLEMENT CURVES GENERATED INTERNALLY

  NUM OF CURVES  1

   CURVE  1          NUM OF POINTS 19

       POINT        AXIAL LOAD,LBS       SETTLEMENT, IN
         1          -1.01646E+05             -2.03583
         2              -95708.3             -1.03374
         3              -92739.7             -0.53270
         4              -59145.2             -0.12084
         5              -41120.4           -0.0644811
         6              -12109.1           -0.0142584
         7              -6093.66         -7.14091E-03
         8              -1218.73         -1.42818E-03
         9              -121.873         -1.42818E-04
        10               0.00000              0.00000
        11               2609.58          1.17654E-03
        12               26095.8            0.0117654
        13            1.29603E+05            0.0585902
        14            2.55285E+05              0.11594
        15            6.17766E+05              0.30745
        16            8.49006E+05              0.45463
        17            8.73314E+05              0.86342
        18            8.76283E+05              1.36447
        19            8.82220E+05              2.36656


* TABLE I *   TORS. MOM. VS ANGLE ROT.

TORQUE-ROTATION CURVES GENERATED INTERNALLY
```

NUM OF CURVES 1

CURVE 1 NUM OF POINTS 19

POINT	TORS.MOMEN,LBS/IN	ROT. ANGLE,Rad.
1	-92.9727	-0.16667
2	-92.9718	-0.0833340
3	-92.9714	-0.0416673
4	-92.9625	-8.33398E-03
5	-92.9515	-4.16732E-03
6	-92.8680	-8.33983E-04
7	-46.4353	-4.16992E-04
8	-9.28706	-8.33983E-05
9	-0.92871	-8.33983E-06
10	0.00000	0.00000
11	0.92871	8.33983E-06
12	9.28706	8.33983E-05
13	46.4353	4.16992E-04
14	92.8680	8.33983E-04
15	92.9515	4.16732E-03
16	92.9625	8.33398E-03
17	92.9714	0.0416673
18	92.9718	0.0833340
19	92.9727	0.16667

***** COMPUTATION RESULTS *****

Thomaston

***** LOAD CASES RESULTS *****

LOAD CASE : 1
CASE NAME : extreme II
LOAD TYPE : Dead, DL

* TABLE L * COMPUTATION ON PILE CAP

* EQUIVALENT CONC. LOAD AT ORIGIN *

VERT. LOAD, LBS	HOR. LOAD Y, LBS	HOR. LOAD Z, LBS
1.20240E+06	1.02450E+05	0.00000

```
     MOMENT X ,IN-LBS    MOMENT Y,IN-LBS    MOMENT Z,IN-LBS
            0.00000            0.00000        8.79900E+06

* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *

       VERTICAL ,IN      HORIZONTAL Y,IN     HORIZONTAL Z,IN
            0.11495            1.13160            0.00000

     ANGLE ROT. X,RAD    ANGLE ROT. Y,RAD    ANGLE ROT. Z,RAD
            0.00000            0.00000        9.92887E-04

     NUMBER OF GLOBAL ITERATIONS =    4


LOAD CASE :     2
CASE NAME : Service I
LOAD TYPE : Dead, DL


* TABLE L *   COMPUTATION ON PILE CAP

* EQUIVALENT CONC. LOAD AT ORIGIN *

      VERT. LOAD, LBS    HOR. LOAD Y, LBS   HOR. LOAD Z, LBS
        1.10200E+06           33060.0            0.00000

     MOMENT X ,IN-LBS    MOMENT Y,IN-LBS    MOMENT Z,IN-LBS
            0.00000            0.00000        1.44876E+06

* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *

       VERTICAL ,IN      HORIZONTAL Y,IN     HORIZONTAL Z,IN
            0.10213            0.34488            0.00000

     ANGLE ROT. X,RAD    ANGLE ROT. Y,RAD    ANGLE ROT. Z,RAD
            0.00000            0.00000        2.97538E-04

     NUMBER OF GLOBAL ITERATIONS =    4
```

```
-----------------------------------------------------------------------
                    Time and Date of Analysis
-----------------------------------------------------------------------

          Date:  March 25, 2014     Time:  11:51:34


            *****     COMPUTATION RESULTS     *****


Thomaston


            *****     LOAD CASES RESULTS      *****


LOAD CASE :     1
CASE NAME : extreme II
LOAD TYPE : Dead, DL


* TABLE L *   COMPUTATION ON PILE CAP

* EQUIVALENT CONC. LOAD AT ORIGIN *

      VERT. LOAD, LBS    HOR. LOAD Y, LBS   HOR. LOAD Z, LBS
        1.20240E+06        1.02450E+05           0.00000

     MOMENT X ,IN-LBS    MOMENT Y,IN-LBS    MOMENT Z,IN-LBS
            0.00000            0.00000        8.79900E+06

* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *
```

VERTICAL ,IN	HORIZONTAL Y,IN	HORIZONTAL Z,IN
0.11495	1.13160	0.00000

ANGLE ROT. X,RAD	ANGLE ROT. Y,RAD	ANGLE ROT. Z,RAD
0.00000	0.00000	9.92887E-04

THE GLOBAL STRUCTURAL COORDINATE SYSTEM
---

* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. X,IN	DISP. Y,IN	DISP. Z,IN	ROT. X,RAD	ROT. Y,RAD	ROT. Z,RAD
1	0.2937	1.1316	0.0000	0.0000	0.0000	9.9289E-04
2	0.2043	1.1316	0.0000	0.0000	0.0000	9.9289E-04
3	0.1149	1.1316	0.0000	0.0000	0.0000	9.9289E-04
4	2.5595E-02	1.1316	0.0000	0.0000	0.0000	9.9289E-04
5	-6.3765E-02	1.1316	0.0000	0.0000	0.0000	9.9289E-04
MINIMUM	-6.3765E-02	1.1316	0.0000	0.0000	0.0000	9.9289E-04
Pile N.	5	1	1	1	1	1
MAXIMUM	0.2937	1.1316	0.0000	0.0000	0.0000	9.9289E-04
Pile N.	1	1	1	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	FOR. X,LBS	FOR. Y,LBS	FOR. Z,LBS	MOM X,LBS-IN	MOM Y,LBS-IN	MOM Z,LBS-IN	STRESS,LBS/IN**2
1	1.2513E+05	-1.4035E+04	0.0000	0.0000	0.0000	3.1685E+06	1.4852E+04
2	4.2256E+05	1.0710E+04	0.0000	0.0000	0.0000	3.0457E+06	2.0858E+04
3	2.5312E+05	1.1166E+04	0.0000	0.0000	0.0000	3.0744E+06	1.7276E+04
4	5.6666E+04	1.1693E+04	0.0000	0.0000	0.0000	3.1073E+06	1.3121E+04
5	3.4492E+05	8.2917E+04	0.0000	0.0000	0.0000	3.0356E+06	1.9338E+04
MINIMUM	5.6666E+04	-1.4035E+04	0.0000	0.0000	0.0000	3.0356E+06	1.3121E+04
Pile N.	4	1	1	1	1	5	4
MAXIMUM	4.2256E+05	8.2917E+04	0.0000	0.0000	0.0000	3.1685E+06	2.0858E+04
Pile N.	2	5	1	1	1	1	2

THE PILE COORDINATE SYSTEM (LOCAL AXES)
---

* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. x,IN	DISP. y,IN	DISP. z,IN	ROT. x,RAD	ROT. y,RAD	ROT. z,RAD
1	5.6672E-02	1.1677	0.0000	0.0000	0.0000	9.9289E-04
2	0.2043	1.1316	0.0000	0.0000	0.0000	9.9289E-04
3	0.1149	1.1316	0.0000	0.0000	0.0000	9.9289E-04
4	2.5595E-02	1.1316	0.0000	0.0000	0.0000	9.9289E-04
5	0.1684	1.1208	0.0000	0.0000	0.0000	9.9289E-04
MINIMUM	2.5595E-02	1.1208	0.0000	0.0000	0.0000	9.9289E-04
Pile N.	4	5	1	1	1	1
MAXIMUM	0.2043	1.1677	0.0000	0.0000	0.0000	9.9289E-04
Pile N.	2	1	1	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	AXIAL,LBS	LAT. y,LBS	LAT. z,LBS	MOM x,LBS-IN	MOM y,LBS-IN	MOM z,LBS-IN	STRESS,LBS/IN**2
1	1.2536E+05	1.1785E+04	0.0000	0.0000	0.0000	3.1685E+06	1.4852E+04
2	4.2256E+05	1.0710E+04	0.0000	0.0000	0.0000	3.0457E+06	2.0858E+04
3	2.5312E+05	1.1166E+04	0.0000	0.0000	0.0000	3.0744E+06	1.7276E+04
4	5.6666E+04	1.1693E+04	0.0000	0.0000	0.0000	3.1073E+06	1.3121E+04
5	3.5458E+05	1.0815E+04	0.0000	0.0000	0.0000	3.0356E+06	1.9338E+04
MINIMUM	5.6666E+04	1.0710E+04	0.0000	0.0000	0.0000	3.0356E+06	1.3121E+04
Pile N.	4	2	1	1	1	5	4
MAXIMUM	4.2256E+05	1.1785E+04	0.0000	0.0000	0.0000	3.1685E+06	2.0858E+04
Pile N.	2	1	1	1	1	1	2

* EFFECTS FOR LATERALLY LOADED PILE *

* MINIMUM VALUES AND LOCATIONS *

PILE	DEFLECTION		BENDING MOMENT		SHEAR FORCE		SOIL REACTION		TOTAL	FLEXURAL RIGIDITY	
	y-DIR IN	z-DIR IN	z-DIR LBS-IN	y-DIR LBS-IN	y-DIR LBS	z-DIR LBS	y-DIR LBS/IN	z-DIR LBS/IN	STRESS LBS/IN**2	z-DIR LBS-IN**2	y-DIR LBS-IN**2
1	-4.1100E-02	0.0000	-3.1700E+06	0.0000	-1.5200E+04	0.0000	-659.00	0.0000	2730.0	9.1000E+10	9.1000E+10
x(IN)	564.00	0.0000	0.0000	0.0000	513.24	0.0000	564.00	0.0000	564.00	0.0000	0.0000
2	-4.0200E-02	0.0000	-3.0500E+06	0.0000	-1.5200E+04	0.0000	-650.00	0.0000	9210.0	9.1000E+10	9.1000E+10
x(IN)	564.00	0.0000	0.0000	0.0000	513.24	0.0000	564.00	0.0000	564.00	0.0000	0.0000
3	-3.9900E-02	0.0000	-3.0700E+06	0.0000	-1.5000E+04	0.0000	-647.00	0.0000	5520.0	9.1000E+10	9.1000E+10
x(IN)	564.00	0.0000	0.0000	0.0000	513.24	0.0000	564.00	0.0000	564.00	0.0000	0.0000

Max deflection=1.2"

25781 5 pile Bent Pier 2.gp8t.txt

4	-3.9600E-02	0.0000	-3.1100E+06	0.0000	-1.4700E+04	0.0000	-645.00	0.0000	1230.0	9.1000E+10	9.1000E+10
x(IN)	564.00	0.0000	0.0000	0.0000	518.88	0.0000	564.00	0.0000	564.00	0.0000	0.0000
5	-3.9700E-02	0.0000	-3.0400E+06	0.0000	-1.5000E+04	0.0000	-645.00	0.0000	7730.0	9.1000E+10	9.1000E+10
x(IN)	564.00	0.0000	0.0000	0.0000	513.24	0.0000	564.00	0.0000	564.00	0.0000	0.0000
Min.	-4.1100E-02	0.0000	-3.1700E+06	0.0000	-1.5200E+04	0.0000	-659.00	0.0000	1230.0	9.1000E+10	9.1000E+10
Pile N.	1	1	1	1	1	1	1	1	4	1	1

* MAXIMUM VALUES AND LOCATIONS *

PILE	DEFLECTION		BENDING MOMENT		SHEAR FORCE		SOIL REACTION		TOTAL	FLEXURAL RIGIDITY	
	y-DIR	z-DIR	z-DIR	y-DIR	y-DIR	z-DIR	y-DIR	z-DIR	STRESS	z-DIR	y-DIR
	IN	IN	LBS-IN	LBS-IN	LBS	LBS	LBS/IN	LBS/IN	LBS/IN**2	LBS-IN**2	LBS-IN**2
*****	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********
1	1.1700	0.0000	1.4800E+06	0.0000	1.1800E+04	0.0000	233.00	0.0000	1.4900E+04	9.1000E+10	9.1000E+10
x(IN)	28.200	0.0000	411.72	0.0000	0.0000	0.0000	406.08	0.0000	0.0000	0.0000	0.0000
2	1.1300	0.0000	1.4500E+06	0.0000	1.0700E+04	0.0000	224.00	0.0000	2.0900E+04	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	411.72	0.0000	0.0000	0.0000	406.08	0.0000	0.0000	0.0000	0.0000
3	1.1300	0.0000	1.4400E+06	0.0000	1.1200E+04	0.0000	225.00	0.0000	1.7300E+04	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	411.72	0.0000	0.0000	0.0000	406.08	0.0000	0.0000	0.0000	0.0000
4	1.1300	0.0000	1.4300E+06	0.0000	1.1700E+04	0.0000	227.00	0.0000	1.3100E+04	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	411.72	0.0000	0.0000	0.0000	406.08	0.0000	0.0000	0.0000	0.0000
5	1.1200	0.0000	1.4400E+06	0.0000	1.0800E+04	0.0000	223.00	0.0000	1.9300E+04	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	411.72	0.0000	0.0000	0.0000	406.08	0.0000	0.0000	0.0000	0.0000
Max.	1.1700	0.0000	1.4800E+06	0.0000	1.1800E+04	0.0000	233.00	0.0000	2.0900E+04	9.1000E+10	9.1000E+10
Pile N.	1	1	1	1	1	1	1	1	2	1	1

Max stress=21 ksi

LOAD CASE : 2
CASE NAME : Service I
LOAD TYPE : Dead, DL

* TABLE L * COMPUTATION ON PILE CAP

* EQUIVALENT CONC. LOAD AT ORIGIN *

VERT. LOAD, LBS	HOR. LOAD Y, LBS	HOR. LOAD Z, LBS
1.10200E+06	33060.0	0.00000

Page 4

MOMENT X ,IN-LBS	MOMENT Y,IN-LBS	MOMENT Z,IN-LBS
0.00000	0.00000	1.44876E+06

* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *

VERTICAL ,IN	HORIZONTAL Y,IN	HORIZONTAL Z,IN
0.10213	0.34488	0.00000

ANGLE ROT. X,RAD	ANGLE ROT. Y,RAD	ANGLE ROT. Z,RAD
0.00000	0.00000	2.97538E-04

THE GLOBAL STRUCTURAL COORDINATE SYSTEM
---

* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. X,IN	DISP. Y,IN	DISP. Z,IN	ROT. X,RAD	ROT. Y,RAD	ROT. Z,RAD
1	0.1557	0.3449	0.0000	0.0000	0.0000	2.9754E-04
2	0.1289	0.3449	0.0000	0.0000	0.0000	2.9754E-04
3	0.1021	0.3449	0.0000	0.0000	0.0000	2.9754E-04
4	7.5347E-02	0.3449	0.0000	0.0000	0.0000	2.9754E-04
5	4.8568E-02	0.3449	0.0000	0.0000	0.0000	2.9754E-04
MINIMUM	4.8568E-02	0.3449	0.0000	0.0000	0.0000	2.9754E-04
Pile N.	5	1	1	1	1	1
MAXIMUM	0.1557	0.3449	0.0000	0.0000	0.0000	2.9754E-04
Pile N.	1	1	1	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	FOR. X,LBS	FOR. Y,LBS	FOR. Z,LBS	MOM X,LBS-IN	MOM Y,LBS-IN	MOM Z,LBS-IN	STRESS,LBS/IN**2
1	1.7798E+05	-3.3321E+04	0.0000	0.0000	0.0000	9.9408E+05	7747.1
2	2.7982E+05	3401.0	0.0000	0.0000	0.0000	9.3696E+05	9680.9
3	2.2501E+05	3446.0	0.0000	0.0000	0.0000	9.3976E+05	8497.4
4	1.6632E+05	3494.1	0.0000	0.0000	0.0000	9.4275E+05	7230.3
5	2.5287E+05	5.6039E+04	0.0000	0.0000	0.0000	9.0167E+05	9092.1
MINIMUM	1.6632E+05	-3.3321E+04	0.0000	0.0000	0.0000	9.0167E+05	7230.3

Pile N.	4	1	1	1	1	5	4
MAXIMUM	2.7982E+05	5.6039E+04	0.0000	0.0000	0.0000	9.9408E+05	9680.9
Pile N.	2	5	1	1	1	1	2

THE PILE COORDINATE SYSTEM (LOCAL AXES)
---

* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. x,IN	DISP. y,IN	DISP. z,IN	ROT. x,RAD	ROT. y,RAD	ROT. z,RAD
1	8.2058E-02	0.3694	0.0000	0.0000	0.0000	2.9754E-04
2	0.1289	0.3449	0.0000	0.0000	0.0000	2.9754E-04
3	0.1021	0.3449	0.0000	0.0000	0.0000	2.9754E-04
4	7.5347E-02	0.3449	0.0000	0.0000	0.0000	2.9754E-04
5	0.1179	0.3277	0.0000	0.0000	0.0000	2.9754E-04
MINIMUM	7.5347E-02	0.3277	0.0000	0.0000	0.0000	2.9754E-04
Pile N.	4	5	1	1	1	1
MAXIMUM	0.1289	0.3694	0.0000	0.0000	0.0000	2.9754E-04
Pile N.	2	1	1	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	AXIAL,LBS	LAT. y,LBS	LAT. z,LBS	MOM x,LBS-IN	MOM y,LBS-IN	MOM z,LBS-IN	STRESS,LBS/IN**2
1	1.8103E+05	3684.6	0.0000	0.0000	0.0000	9.9408E+05	7747.1
2	2.7982E+05	3401.0	0.0000	0.0000	0.0000	9.3696E+05	9680.9
3	2.2501E+05	3446.0	0.0000	0.0000	0.0000	9.3976E+05	8497.4
4	1.6632E+05	3494.1	0.0000	0.0000	0.0000	9.4275E+05	7230.3
5	2.5899E+05	3279.4	0.0000	0.0000	0.0000	9.0167E+05	9092.1
MINIMUM	1.6632E+05	3279.4	0.0000	0.0000	0.0000	9.0167E+05	7230.3
Pile N.	4	5	1	1	1	5	4
MAXIMUM	2.7982E+05	3684.6	0.0000	0.0000	0.0000	9.9408E+05	9680.9
Pile N.	2	1	1	1	1	1	2

* EFFECTS FOR LATERALLY LOADED PILE *

* MINIMUM VALUES AND LOCATIONS *

PILE	DEFLECTION		BENDING MOMENT		SHEAR FORCE		SOIL REACTION		TOTAL	FLEXURAL RIGIDITY	
	y-DIR	z-DIR	z-DIR	y-DIR	y-DIR	z-DIR	y-DIR	z-DIR	STRESS	z-DIR	y-DIR

25781 5 pile Bent Pier 2.gp8t.txt

	IN	IN	LBS-IN	LBS-IN	LBS	LBS	LBS/IN	LBS/IN	LBS/IN**2	LBS-IN**2	LBS-IN**2
*****	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********
1	-1.2700E-02	0.0000	-9.9400E+05	0.0000	-4800.0	0.0000	-214.00	0.0000	3940.0	9.1000E+10	9.1000E+10
x(IN)	564.00	0.0000	0.0000	0.0000	513.24	0.0000	564.00	0.0000	564.00	0.0000	0.0000
2	-1.1900E-02	0.0000	-9.3700E+05	0.0000	-4550.0	0.0000	-202.00	0.0000	6100.0	9.1000E+10	9.1000E+10
x(IN)	564.00	0.0000	0.0000	0.0000	513.24	0.0000	564.00	0.0000	564.00	0.0000	0.0000
3	-1.1900E-02	0.0000	-9.4000E+05	0.0000	-4530.0	0.0000	-202.00	0.0000	4900.0	9.1000E+10	9.1000E+10
x(IN)	564.00	0.0000	0.0000	0.0000	513.24	0.0000	564.00	0.0000	564.00	0.0000	0.0000
4	-1.1900E-02	0.0000	-9.4300E+05	0.0000	-4510.0	0.0000	-201.00	0.0000	3620.0	9.1000E+10	9.1000E+10
x(IN)	564.00	0.0000	0.0000	0.0000	513.24	0.0000	564.00	0.0000	564.00	0.0000	0.0000
5	-1.1400E-02	0.0000	-9.0200E+05	0.0000	-4340.0	0.0000	-193.00	0.0000	5640.0	9.1000E+10	9.1000E+10
x(IN)	564.00	0.0000	0.0000	0.0000	513.24	0.0000	564.00	0.0000	564.00	0.0000	0.0000
Min.	-1.2700E-02	0.0000	-9.9400E+05	0.0000	-4800.0	0.0000	-214.00	0.0000	3620.0	9.1000E+10	9.1000E+10
Pile N.	1	1	1	1	1	1	1	1	4	1	1

* MAXIMUM VALUES AND LOCATIONS *

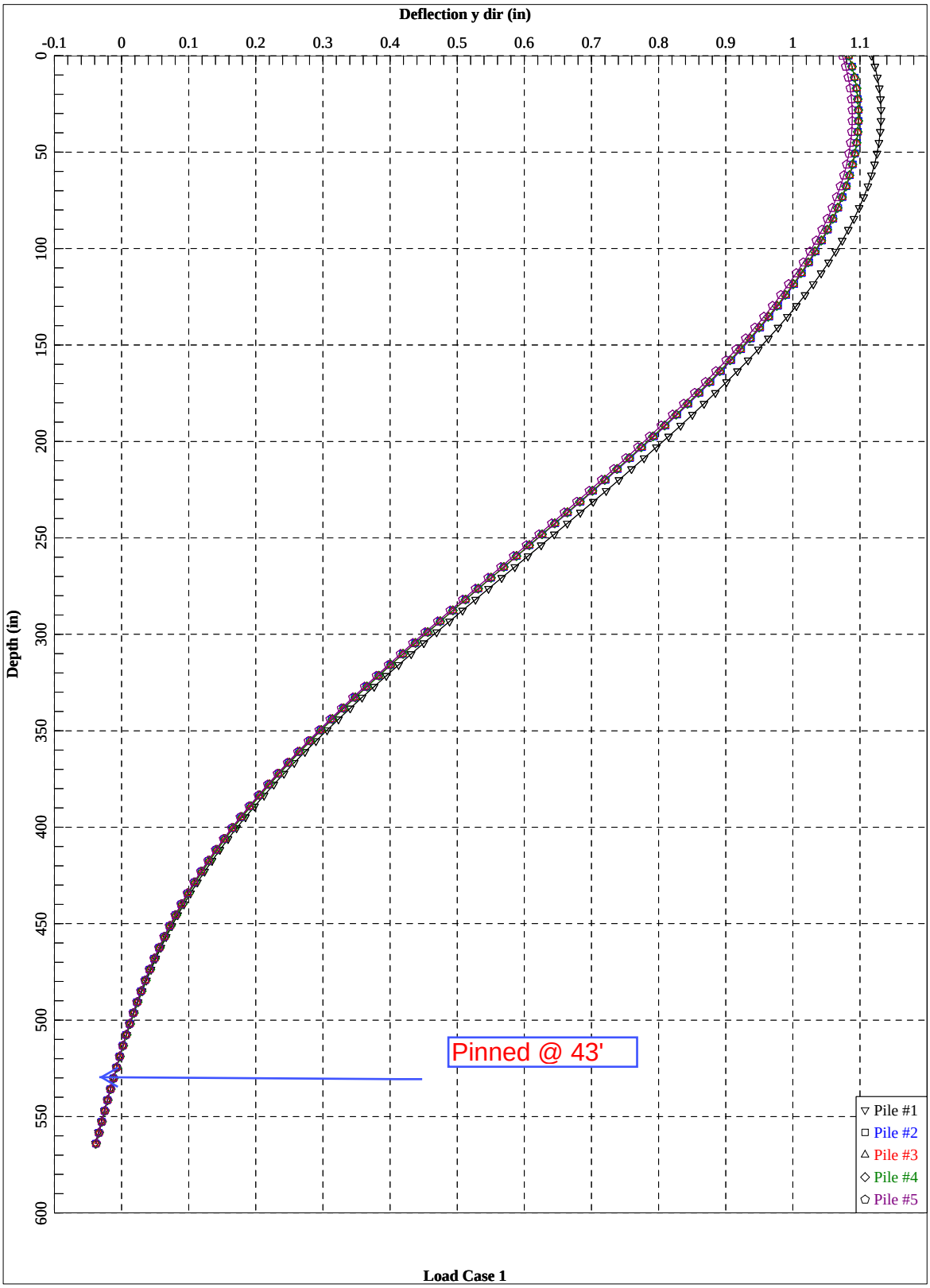
PILE	DEFLECTION		BENDING MOMENT		SHEAR FORCE		SOIL REACTION		TOTAL	FLEXURAL RIGIDITY	
	y-DIR	z-DIR	z-DIR	y-DIR	y-DIR	z-DIR	y-DIR	z-DIR	STRESS	z-DIR	y-DIR
	IN	IN	LBS-IN	LBS-IN	LBS	LBS	LBS/IN	LBS/IN	LBS/IN**2	LBS-IN**2	LBS-IN**2
*****	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********
1	0.3690	0.0000	4.6900E+05	0.0000	3690.0	0.0000	72.100	0.0000	7750.0	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	411.72	0.0000	0.0000	0.0000	406.08	0.0000	0.0000	0.0000	0.0000
2	0.3450	0.0000	4.4100E+05	0.0000	3410.0	0.0000	67.700	0.0000	9680.0	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	411.72	0.0000	0.0000	0.0000	406.08	0.0000	0.0000	0.0000	0.0000
3	0.3450	0.0000	4.4100E+05	0.0000	3450.0	0.0000	67.800	0.0000	8500.0	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	411.72	0.0000	0.0000	0.0000	406.08	0.0000	0.0000	0.0000	0.0000
4	0.3450	0.0000	4.4000E+05	0.0000	3500.0	0.0000	67.900	0.0000	7230.0	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	411.72	0.0000	0.0000	0.0000	406.08	0.0000	0.0000	0.0000	0.0000
5	0.3280	0.0000	4.2100E+05	0.0000	3290.0	0.0000	64.800	0.0000	9090.0	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	411.72	0.0000	0.0000	0.0000	406.08	0.0000	0.0000	0.0000	0.0000
Max.	0.3690	0.0000	4.6900E+05	0.0000	3690.0	0.0000	72.100	0.0000	9680.0	9.1000E+10	9.1000E+10
Pile N.	1	1	1	1	1	1	1	1	2	1	1

Max deflection=0.4"

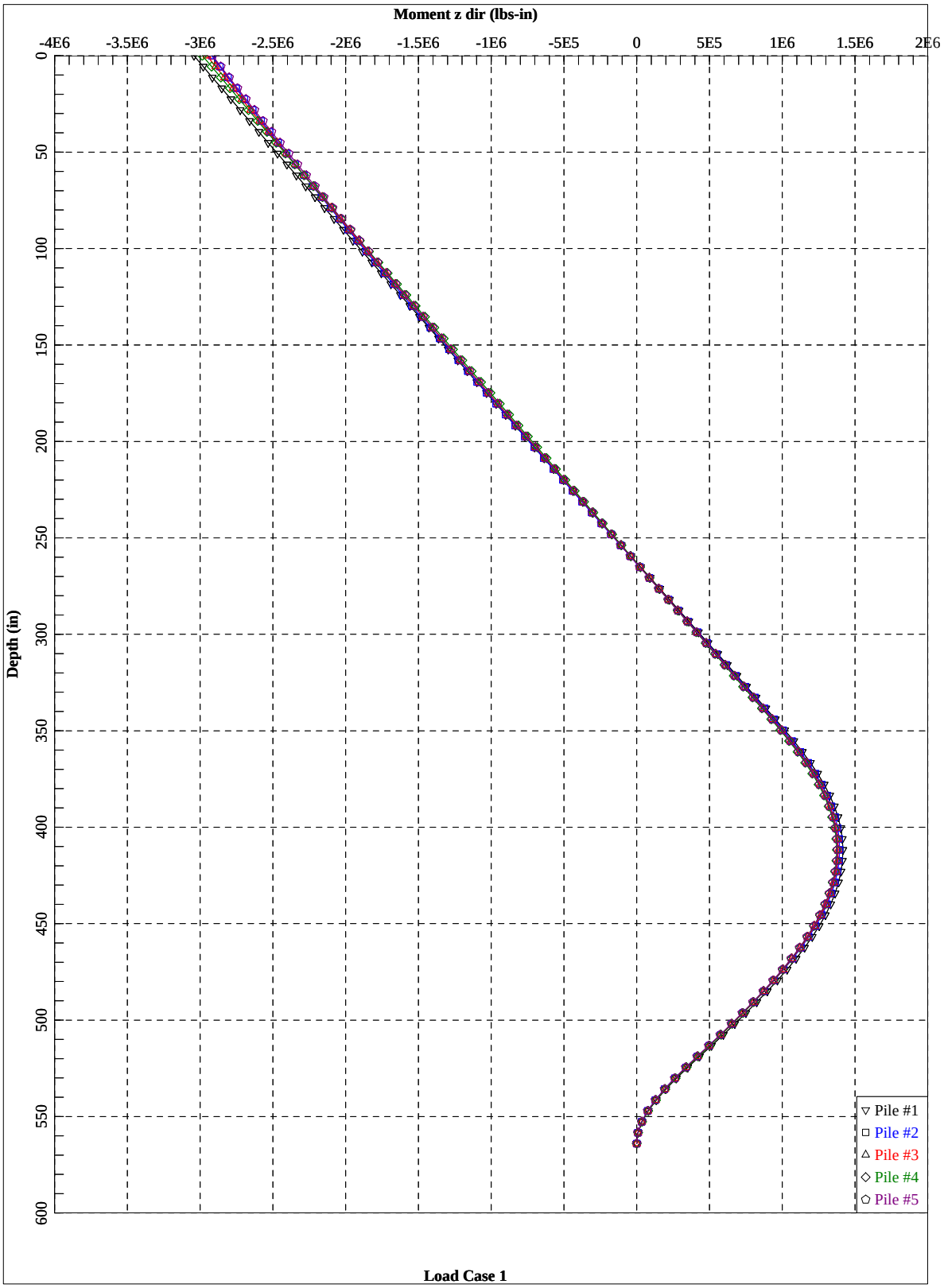
Max stress=10 ksi

Page 7

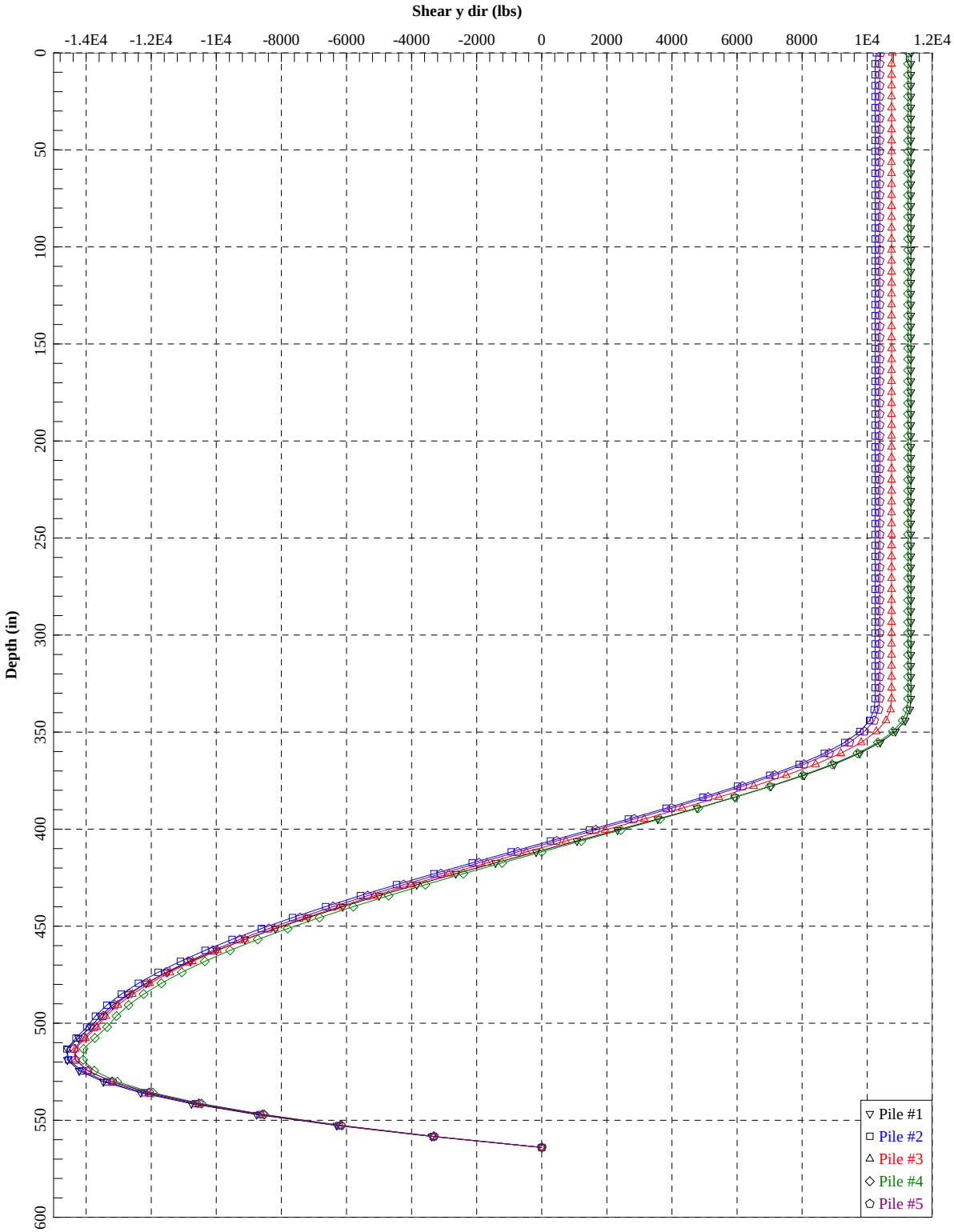
Extreme II Load Case



Extreme II Load Case

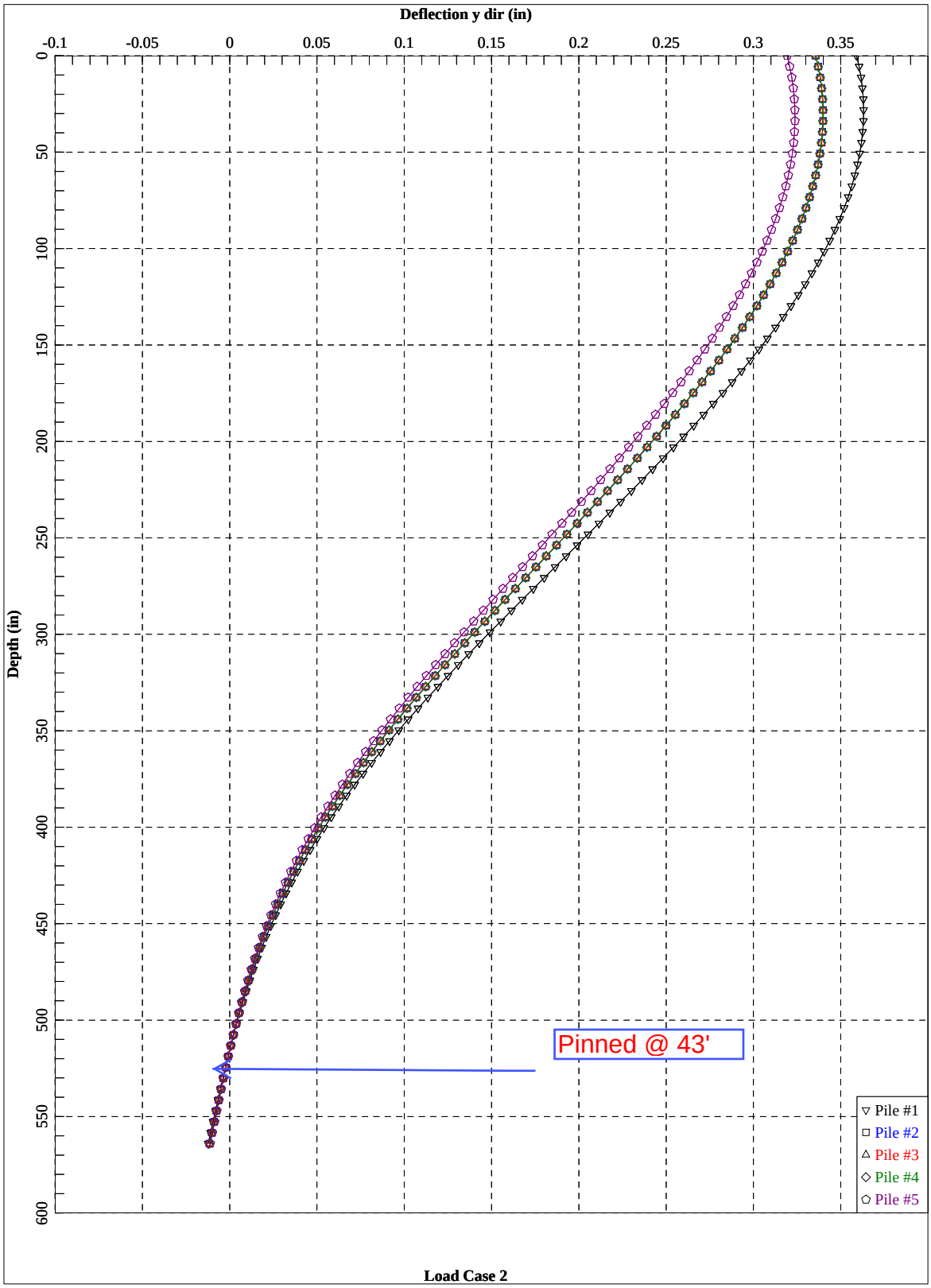


Extreme II Load Case

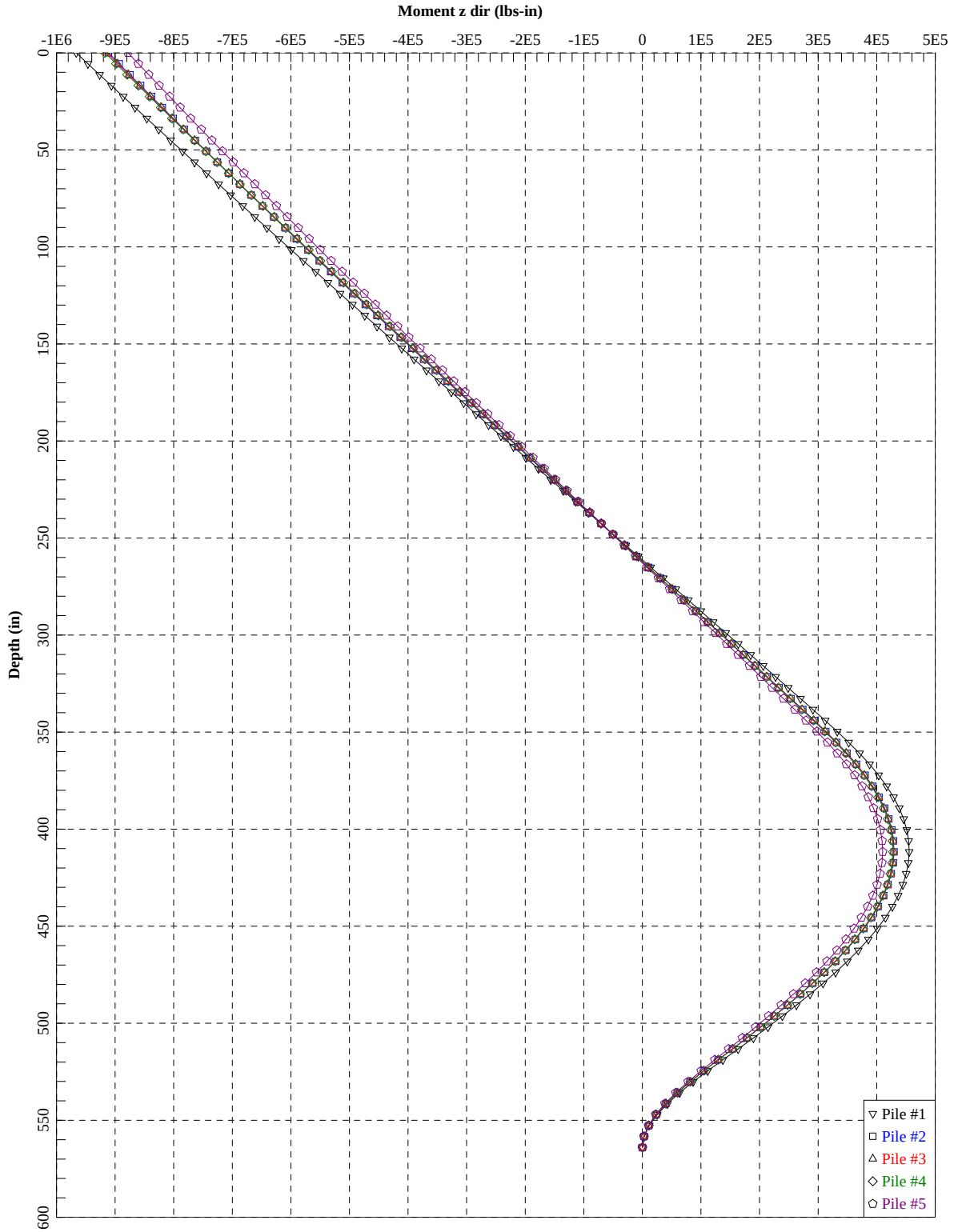


Load Case 1

Service I Load Case

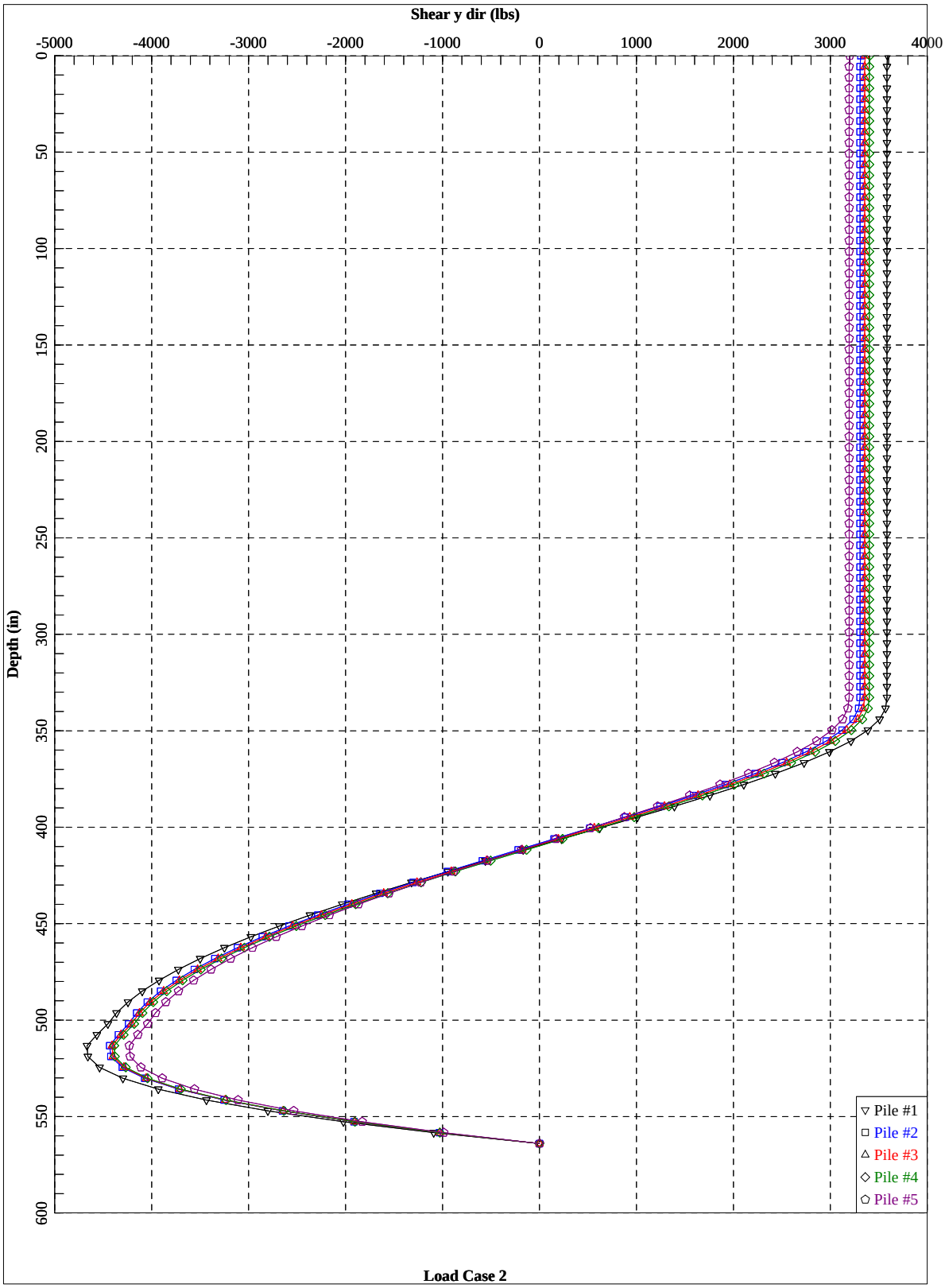


Service I Load Case



Load Case 2

Service I Load Case



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==========================================================================

                 GROUP for Windows, Version 8.0.15

                  Serial Number :  364300562

                  Analysis of A Group of Piles
               Subjected to Axial and Lateral Loading

                (c) Copyright ENSOFT, Inc., 1987-2010
                      All Rights Reserved
==========================================================================
```

Pier 3
PP24x0.625 piles
Uncorroded Section
No concrete fill included
Extreme II Load Case

```
This program is licensed to :

gza
GZA GeoEnvironmental, Inc.

Path to file locations      : P:\09 Jobs\0025700s\09.0025781.00 Thomaston Bridge MaineDOT\Work\Calcs\GROUP\
Name of input data file     : 25781 5 pile Bent Pier 3.gp8d
Name of output file         : 25781 5 pile Bent Pier 3.gp8o
Name of output summary file : 25781 5 pile Bent Pier 3.gp8t
Name of plot output file    : 25781 5 pile Bent Pier 3.gp8p
Name of runtime file        : 25781 5 pile Bent Pier 3.gp8r


--------------------------------------------------------------------------
                      Time and Date of Analysis
--------------------------------------------------------------------------

          Date:  March 26, 2014     Time:  10:31:49




           *****     INPUT INFORMATION     *****

Thomaston pier 3

ANALYSIS TYPE = 3D ANALYSIS

UNITS SYSTEM  = ENGL


* TABLE C *  LOAD AND CONTROL PARAMETERS


** LOAD CASES **
```

NUMBER OF LOAD CASES : 1

LOAD CASE : 1
CASE NAME : extreme II
LOAD TYPE : Dead, DL
SCALE FACTOR : 1.0000

* CONCENTRATED LOADS *

NL	VERT.LOAD LBS	HR.LOAD Y LBS	HR.LOAD Z LBS	MOMENT X LBS-IN	MOMENT Y LBS-IN	MOMENT Z LBS-IN	COORD X IN	COORD Y IN	COORD Z IN
1	39500.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-180.000	0.00000
2	39500.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-90.0000	0.00000
3	39500.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
4	39500.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	90.0000	0.00000
5	39500.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	180.000	0.00000
6	6.21300E+05	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
7	3.01600E+05	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-30.0000	0.00000
8	82000.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	165.000	0.00000
9	0.00000	86000.0	0.00000	0.00000	0.00000	0.00000	120.000	-215.000	0.00000
10	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	-230.000	0.00000
11	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	-90.0000	0.00000
12	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	0.00000	0.00000
13	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	90.0000	0.00000
14	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	230.000	0.00000

* EQUIVALENT CONC. LOAD AT ORIGIN *

VER.LOAD X,LBS	HOR.LOAD Y,LBS	HOR.LOAD Z,LBS
1.20240E+06	1.02450E+05	0.00000

MOMENT X,LBS-IN	MOMENT Y,LBS-IN	MOMENT Z,LBS-IN
0.00000	0.00000	8.79900E+06

* THE LOADING IS STATIC *

* CONTROL PARAMETERS *
TOLERANCE ON CONVERGENCE OF FOUNDATION REACTION, = 1.00000E-04
TOLERANCE ON DETERMINATION OF DEFLECTIONS = 1.00000E-04 IN
MAX NO OF ITERATIONS ALLOWED FOR FOUNDATION ANALYSIS = 100
MAXIMUM NO OF ITERATIONS ALLOWED FOR PILE ANALYSIS = 100
FACTOR TO APPLY THE LOAD IN INCREMENTS = 1.0000
MINIMUM FACTOR FOR LOAD INCREMENTS = 1.0000
PRINT RESULTS ONLY AT PILE CAP

* TABLE D * ARRANGEMENT OF PILE GROUPS

GROUP	CONNECTY	CONNECTZ	PILE PROP	P-Y CURVE	L-S CURVE	T-R CURVE
1	FIX	FIX	1	0	0	0
2	FIX	FIX	1	0	0	0

3	FIX	FIX	1	0	0	0
4	FIX	FIX	1	0	0	0
5	FIX	FIX	1	0	0	0

GROUP	CorX,IN	CorY,IN	CorZ,IN	ALPHA,DEG	BETA,DEG	GROUND,IN	SPR.y,LBS-IN	SPR.z,LBS-IN
1	0.00000	-180.000	0.00000	0.00000	101.770	276.000	0.00000	0.00000
2	0.00000	-90.0000	0.00000	0.00000	90.0000	276.000	0.00000	0.00000
3	0.00000	0.00000	0.00000	0.00000	90.0000	276.000	0.00000	0.00000
4	0.00000	90.0000	0.00000	0.00000	90.0000	276.000	0.00000	0.00000
5	0.00000	180.000	0.00000	0.00000	78.2300	276.000	0.00000	0.00000

```
* TABLE E *   PILE GEOMETRY AND PROPERTIES
              PILE TYPE = 1 - DRIVEN PILE
                        = 2 - DRILLED SHAFT
```

PROP	SECTS	INC	PILE TYPE	LENGTH, IN	E,LBS/IN**2
1	1	100	1	588.00	2.9000E+07

```
* PILE SECTIONS *
```

PROP	SECT	FROM,IN	TO,IN	CROSS SECT
1	1	0.00000	588.000	1

```
* PILE CROSS SECTIONS *

 CROSS SECTION       :     1
 SECTION NAME        : Section
 CROSS SECTION TYPE  : CIRCULAR PIPE
 EXTERNAL DIAMETER   :       24.0000 IN
 INTERNAL DIAMETER   :       22.7500 IN
 SHEAR MODULUS       :    1.0000     LBS/IN**2
 HEAR MODULUS       : ,      1.00000 LBS/IN**2
```

```
* PILE CROSS SECTIONS PROPERTIES *
```

SECT	DIAM,IN	AREA,IN**2	Iz,IN**4	Iy,IN**4	GJ,LBS-IN**2	Mn,LBS-IN	Vn,LBS
1	24.0000	45.8967	3136.93	3136.93	7.27768E+10	0.00000	0.00000

```
* TABLE F *   SOIL DATA

SOILS INFORMATION

    GROUND SURFACE          =      276.000  IN

    5 LAYER(S) OF SOIL

    LAYER  1
    THE SOIL IS A SAND
                                                TOP OF LAYER    BOTTOM OF LAYER
    X COORDINATE                          (IN)       276.000          348.000
    EFFECTIVE UNIT WEIGHT         (LBS/IN**3)      0.0333000        0.0333000
```

```
FRICTION ANGLE                  (DEGREES)        30.0000           30.0000
P-Y SUBGRADE MODULUS          (LBS/IN**3)        20.0000           20.0000
ULTIMATE UNIT SIDE FRICTION   (LBS/IN**2)        0.00000           0.00000
ULTIMATE UNIT TIP RESISTANCE  (LBS/IN**2)        0.00000           0.00000
```

River Bottom Sand

```
LAYER  2
THE SOIL IS A SOFT CLAY
                                            TOP OF LAYER    BOTTOM OF LAYER
X COORDINATE                          (IN)       348.000           456.000
EFFECTIVE UNIT WEIGHT         (LBS/IN**3)      0.0304000         0.0304000
UNDRAINED COHESION, C         (LBS/IN**2)        2.08000           2.08000
STRAIN AT 50% STRESS                          0.0200000         0.0200000
ULTIMATE UNIT SIDE FRICTION   (LBS/IN**2)        0.00000           0.00000
ULTIMATE UNIT TIP RESISTANCE  (LBS/IN**2)        0.00000           0.00000
```

Silt/Clay

```
LAYER  3
THE SOIL IS A SAND
                                            TOP OF LAYER    BOTTOM OF LAYER
X COORDINATE                          (IN)       456.000           504.000
EFFECTIVE UNIT WEIGHT         (LBS/IN**3)      0.0333000         0.0330000
FRICTION ANGLE                  (DEGREES)        30.0000           30.0000
P-Y SUBGRADE MODULUS          (LBS/IN**3)        20.0000           20.0000
ULTIMATE UNIT SIDE FRICTION   (LBS/IN**2)        0.00000           0.00000
ULTIMATE UNIT TIP RESISTANCE  (LBS/IN**2)        0.00000           0.00000
```

River Bottom Sand

```
LAYER  4
THE SOIL IS A SAND
                                            TOP OF LAYER    BOTTOM OF LAYER
X COORDINATE                          (IN)       504.000           588.000
EFFECTIVE UNIT WEIGHT         (LBS/IN**3)      0.0360000         0.0360000
FRICTION ANGLE                  (DEGREES)        38.0000           38.0000
P-Y SUBGRADE MODULUS          (LBS/IN**3)        90.0000           90.0000
ULTIMATE UNIT SIDE FRICTION   (LBS/IN**2)        0.00000           0.00000
ULTIMATE UNIT TIP RESISTANCE  (LBS/IN**2)        0.00000           0.00000
```

Till/Boulders

```
LAYER  5
THE LAYER IS A VUGGY LIMESTONE
                                            TOP OF LAYER    BOTTOM OF LAYER
X COORDINATE                          (IN)       588.000           600.000
EFFECTIVE UNIT WEIGHT         (LBS/IN**3)      0.0565000         0.0565000
UNIAXIAL COMPRESSIVE STRENGTH (LBS/IN**2)        4000.00           4000.00
ULTIMATE UNIT SIDE FRICTION   (LBS/IN**2)        0.00000           0.00000
ULTIMATE UNIT TIP RESISTANCE  (LBS/IN**2)        0.00000           0.00000

  Note :  Default values will be generated for
          ULTIMATE UNIT SIDE FRICTION  and ULTIMATE UNIT TIP RESISTANCE
          when input values are 0.


* TABLE H *   AXIAL LOAD VS SETTLEMENT

LOAD-SETTLEMENT CURVES GENERATED INTERNALLY

  NUM OF CURVES  1
```

CURVE 1 NUM OF POINTS 19

POINT	AXIAL LOAD,LBS	SETTLEMENT, IN
1	-1.13869E+05	-2.04023
2	-1.08728E+05	-1.03834
3	-1.06158E+05	-0.53740
4	-77363.8	-0.12673
5	-59238.5	-0.0701930
6	-16323.4	-0.0155872
7	-8212.41	-7.80571E-03
8	-1642.48	-1.56114E-03
9	-164.248	-1.56114E-04
10	0.00000	0.00000
11	5757.53	2.51866E-03
12	57644.3	0.0252232
13	2.81782E+05	0.12451
14	5.42150E+05	0.24276
15	1.27046E+06	0.60311
16	1.74193E+06	0.86075
17	1.75907E+06	1.26752
18	1.76164E+06	1.76847
19	1.76678E+06	2.77036

* TABLE I * TORS. MOM. VS ANGLE ROT.

TORQUE-ROTATION CURVES GENERATED INTERNALLY

NUM OF CURVES 1

CURVE 1 NUM OF POINTS 19

POINT	TORS.MOMEN,LBS/IN	ROT. ANGLE,Rad.
1	-136.698	-0.16667
2	-136.697	-0.0833343
3	-136.696	-0.0416676
4	-136.683	-8.33427E-03
5	-136.666	-4.16760E-03
6	-136.537	-8.34270E-04
7	-68.2739	-4.17135E-04
8	-13.6548	-8.34270E-05
9	-1.36548	-8.34270E-06
10	0.00000	0.00000
11	1.36548	8.34270E-06
12	13.6548	8.34270E-05
13	68.2739	4.17135E-04
14	136.537	8.34270E-04
15	136.666	4.16760E-03
16	136.683	8.33427E-03
17	136.696	0.0416676
18	136.697	0.0833343
19	136.698	0.16667

```
            *****     COMPUTATION RESULTS     *****


Thomaston pier 3



            *****     LOAD CASES RESULTS      *****


LOAD CASE :     1
CASE NAME : extreme II
LOAD TYPE : Dead, DL


* TABLE L *   COMPUTATION ON PILE CAP

* EQUIVALENT CONC. LOAD AT ORIGIN *

      VERT. LOAD, LBS    HOR. LOAD Y, LBS   HOR. LOAD Z, LBS
        1.20240E+06        1.02450E+05            0.00000

     MOMENT X ,IN-LBS    MOMENT Y,IN-LBS    MOMENT Z,IN-LBS
            0.00000            0.00000        8.79900E+06

* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *

       VERTICAL ,IN      HORIZONTAL Y,IN     HORIZONTAL Z,IN
            0.10839            0.88657            0.00000

     ANGLE ROT. X,RAD    ANGLE ROT. Y,RAD    ANGLE ROT. Z,RAD
            0.00000            0.00000        7.74613E-04

     NUMBER OF GLOBAL ITERATIONS =    4
```

```
----------------------------------------------------------------------
                    Time and Date of Analysis
----------------------------------------------------------------------

          Date:  March 26, 2014     Time:  10:31:49


            *****     COMPUTATION RESULTS     *****


Thomaston pier 3


            *****     LOAD CASES RESULTS      *****


LOAD CASE :     1
CASE NAME : extreme II
LOAD TYPE : Dead, DL


* TABLE L *   COMPUTATION ON PILE CAP

* EQUIVALENT CONC. LOAD AT ORIGIN *

      VERT. LOAD, LBS    HOR. LOAD Y, LBS   HOR. LOAD Z, LBS
        1.20240E+06        1.02450E+05           0.00000

     MOMENT X ,IN-LBS    MOMENT Y,IN-LBS    MOMENT Z,IN-LBS
            0.00000            0.00000        8.79900E+06

* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *
```

VERTICAL ,IN	HORIZONTAL Y,IN	HORIZONTAL Z,IN
0.10839	0.88657	0.00000

ANGLE ROT. X,RAD	ANGLE ROT. Y,RAD	ANGLE ROT. Z,RAD
0.00000	0.00000	7.74613E-04

THE GLOBAL STRUCTURAL COORDINATE SYSTEM
---

* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. X,IN	DISP. Y,IN	DISP. Z,IN	ROT. X,RAD	ROT. Y,RAD	ROT. Z,RAD
1	0.2478	0.8866	0.0000	0.0000	0.0000	7.7461E-04
2	0.1781	0.8866	0.0000	0.0000	0.0000	7.7461E-04
3	0.1084	0.8866	0.0000	0.0000	0.0000	7.7461E-04
4	3.8679E-02	0.8866	0.0000	0.0000	0.0000	7.7461E-04
5	-3.1036E-02	0.8866	0.0000	0.0000	0.0000	7.7461E-04
MINIMUM	-3.1036E-02	0.8866	0.0000	0.0000	0.0000	7.7461E-04
Pile N.	5	1	1	1	1	1
MAXIMUM	0.2478	0.8866	0.0000	0.0000	0.0000	7.7461E-04
Pile N.	1	1	1	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	FOR. X,LBS	FOR. Y,LBS	FOR. Z,LBS	MOM X,LBS-IN	MOM Y,LBS-IN	MOM Z,LBS-IN	STRESS,LBS/IN**2
1	1.3985E+05	-1.5877E+04	0.0000	0.0000	0.0000	3.0446E+06	1.4700E+04
2	3.9980E+05	1.2105E+04	0.0000	0.0000	0.0000	2.9448E+06	1.9976E+04
3	2.4541E+05	1.2442E+04	0.0000	0.0000	0.0000	2.9601E+06	1.6671E+04
4	8.8021E+04	1.2784E+04	0.0000	0.0000	0.0000	2.9755E+06	1.3300E+04
5	3.2933E+05	8.0996E+04	0.0000	0.0000	0.0000	2.9212E+06	1.8559E+04
MINIMUM	8.8021E+04	-1.5877E+04	0.0000	0.0000	0.0000	2.9212E+06	1.3300E+04
Pile N.	4	1	1	1	1	5	4
MAXIMUM	3.9980E+05	8.0996E+04	0.0000	0.0000	0.0000	3.0446E+06	1.9976E+04
Pile N.	2	5	1	1	1	1	2

THE PILE COORDINATE SYSTEM (LOCAL AXES)
---

* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. x,IN	DISP. y,IN	DISP. z,IN	ROT. x,RAD	ROT. y,RAD	ROT. z,RAD
1	6.1768E-02	0.9185	0.0000	0.0000	0.0000	7.7461E-04
2	0.1781	0.8866	0.0000	0.0000	0.0000	7.7461E-04
3	0.1084	0.8866	0.0000	0.0000	0.0000	7.7461E-04
4	3.8679E-02	0.8866	0.0000	0.0000	0.0000	7.7461E-04
5	0.1505	0.8743	0.0000	0.0000	0.0000	7.7461E-04
MINIMUM	3.8679E-02	0.8743	0.0000	0.0000	0.0000	7.7461E-04
Pile N.	4	5	1	1	1	1
MAXIMUM	0.1781	0.9185	0.0000	0.0000	0.0000	7.7461E-04
Pile N.	2	1	1	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	AXIAL,LBS	LAT. y,LBS	LAT. z,LBS	MOM x,LBS-IN	MOM y,LBS-IN	MOM z,LBS-IN	STRESS,LBS/IN**2
1	1.4014E+05	1.2983E+04	0.0000	0.0000	0.0000	3.0446E+06	1.4700E+04
2	3.9980E+05	1.2105E+04	0.0000	0.0000	0.0000	2.9448E+06	1.9976E+04
3	2.4541E+05	1.2442E+04	0.0000	0.0000	0.0000	2.9601E+06	1.6671E+04
4	8.8021E+04	1.2784E+04	0.0000	0.0000	0.0000	2.9755E+06	1.3300E+04
5	3.3893E+05	1.2115E+04	0.0000	0.0000	0.0000	2.9212E+06	1.8559E+04
MINIMUM	8.8021E+04	1.2105E+04	0.0000	0.0000	0.0000	2.9212E+06	1.3300E+04
Pile N.	4	2	1	1	1	5	4
MAXIMUM	3.9980E+05	1.2983E+04	0.0000	0.0000	0.0000	3.0446E+06	1.9976E+04
Pile N.	2	1	1	1	1	1	2

* EFFECTS FOR LATERALLY LOADED PILE *

* MINIMUM VALUES AND LOCATIONS *

PILE	DEFLECTION y-DIR IN	DEFLECTION z-DIR IN	BENDING MOMENT z-DIR LBS-IN	BENDING MOMENT y-DIR LBS-IN	SHEAR FORCE y-DIR LBS	SHEAR FORCE z-DIR LBS	SOIL REACTION y-DIR LBS/IN	SOIL REACTION z-DIR LBS/IN	TOTAL STRESS LBS/IN**2	FLEXURAL RIGIDITY z-DIR LBS-IN**2	FLEXURAL RIGIDITY y-DIR LBS-IN**2
1	-1.2700E-02	0.0000	-3.0400E+06	0.0000	-9300.0	0.0000	-245.00	0.0000	3050.0	9.1000E+10	9.1000E+10
x(IN)	588.00	0.0000	0.0000	0.0000	511.56	0.0000	588.00	0.0000	588.00	0.0000	0.0000
2	-1.1700E-02	0.0000	-2.9400E+06	0.0000	-9180.0	0.0000	-227.00	0.0000	8710.0	9.1000E+10	9.1000E+10
x(IN)	588.00	0.0000	0.0000	0.0000	505.68	0.0000	588.00	0.0000	588.00	0.0000	0.0000
3	-1.1900E-02	0.0000	-2.9600E+06	0.0000	-9070.0	0.0000	-231.00	0.0000	5350.0	9.1000E+10	9.1000E+10
x(IN)	588.00	0.0000	0.0000	0.0000	511.56	0.0000	588.00	0.0000	588.00	0.0000	0.0000

4	-1.2100E-02	0.0000	-2.9800E+06	0.0000	-8980.0	0.0000	-234.00	0.0000	1920.0	9.1000E+10	9.1000E+10
x(IN)	588.00	0.0000	0.0000	0.0000	511.56	0.0000	588.00	0.0000	588.00	0.0000	0.0000
5	-1.1600E-02	0.0000	-2.9200E+06	0.0000	-9020.0	0.0000	-224.00	0.0000	7380.0	9.1000E+10	9.1000E+10
x(IN)	588.00	0.0000	0.0000	0.0000	505.68	0.0000	588.00	0.0000	588.00	0.0000	0.0000
Min.	-1.2700E-02	0.0000	-3.0400E+06	0.0000	-9300.0	0.0000	-245.00	0.0000	1920.0	9.1000E+10	9.1000E+10
Pile N.	1	1	1	1	1	1	1	1	4	1	1

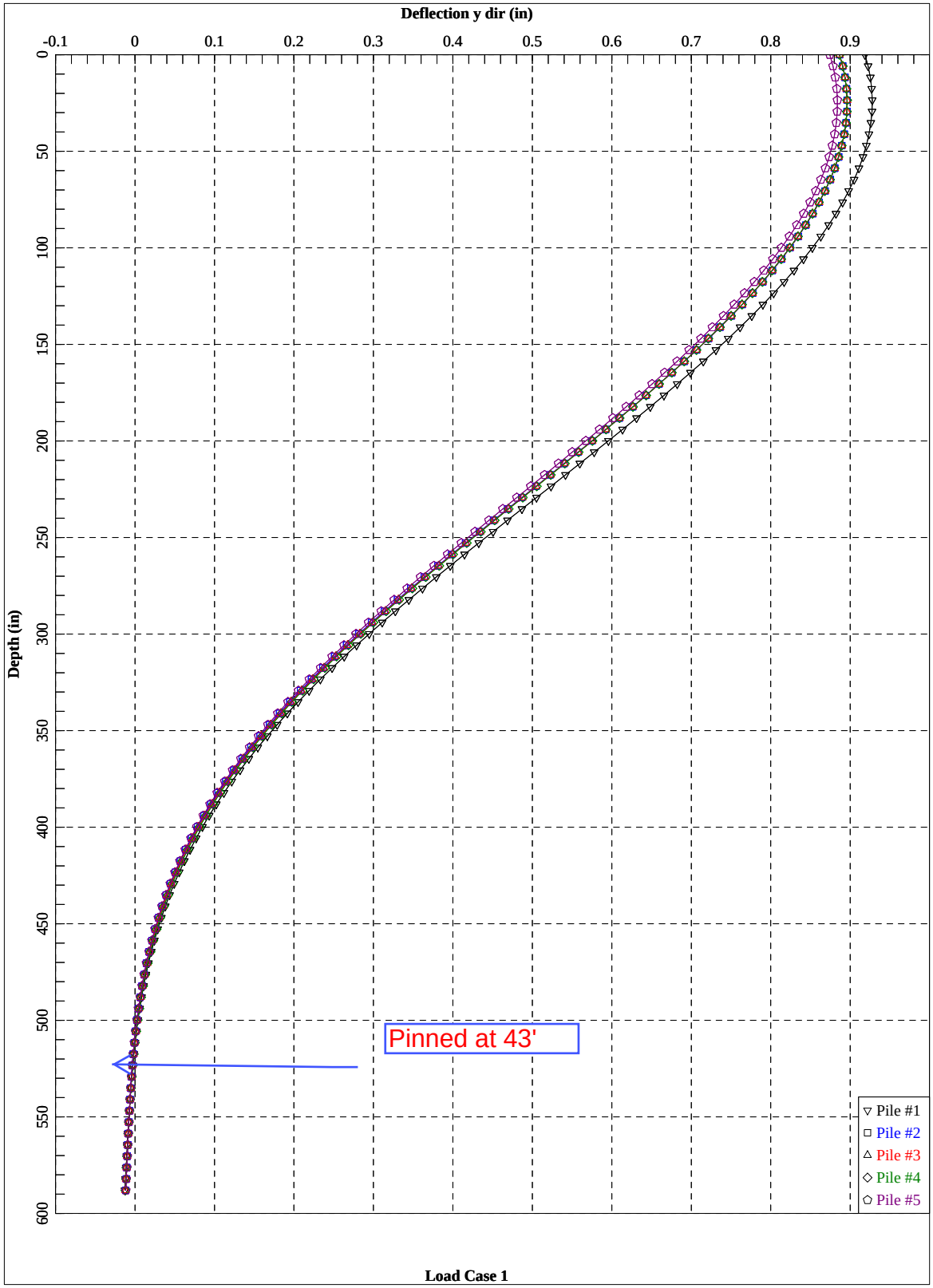
* MAXIMUM VALUES AND LOCATIONS *

PILE	DEFLECTION		BENDING MOMENT		SHEAR FORCE		SOIL REACTION		TOTAL	FLEXURAL RIGIDITY	
	y-DIR	z-DIR	z-DIR	y-DIR	y-DIR	z-DIR	y-DIR	z-DIR	STRESS	z-DIR	y-DIR
	IN	IN	LBS-IN	LBS-IN	LBS	LBS	LBS/IN	LBS/IN	LBS/IN**2	LBS-IN**2	LBS-IN**2
*****	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********
1	0.9180	0.0000	1.3000E+06	0.0000	1.3000E+04	0.0000	249.00	0.0000	1.4700E+04	9.1000E+10	9.1000E+10
x(IN)	23.520	0.0000	364.56	0.0000	0.0000	0.0000	346.92	0.0000	0.0000	0.0000	0.0000
2	0.8870	0.0000	1.2800E+06	0.0000	1.2100E+04	0.0000	240.00	0.0000	2.0000E+04	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	364.56	0.0000	0.0000	0.0000	346.92	0.0000	0.0000	0.0000	0.0000
3	0.8870	0.0000	1.2700E+06	0.0000	1.2500E+04	0.0000	242.00	0.0000	1.6700E+04	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	364.56	0.0000	0.0000	0.0000	346.92	0.0000	0.0000	0.0000	0.0000
4	0.8870	0.0000	1.2600E+06	0.0000	1.2800E+04	0.0000	244.00	0.0000	1.3300E+04	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	364.56	0.0000	0.0000	0.0000	346.92	0.0000	0.0000	0.0000	0.0000
5	0.8740	0.0000	1.2600E+06	0.0000	1.2100E+04	0.0000	237.00	0.0000	1.8600E+04	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	364.56	0.0000	0.0000	0.0000	346.92	0.0000	0.0000	0.0000	0.0000
Max.	0.9180	0.0000	1.3000E+06	0.0000	1.3000E+04	0.0000	249.00	0.0000	2.0000E+04	9.1000E+10	9.1000E+10
Pile N.	1	1	1	1	1	1	1	1	2	1	1

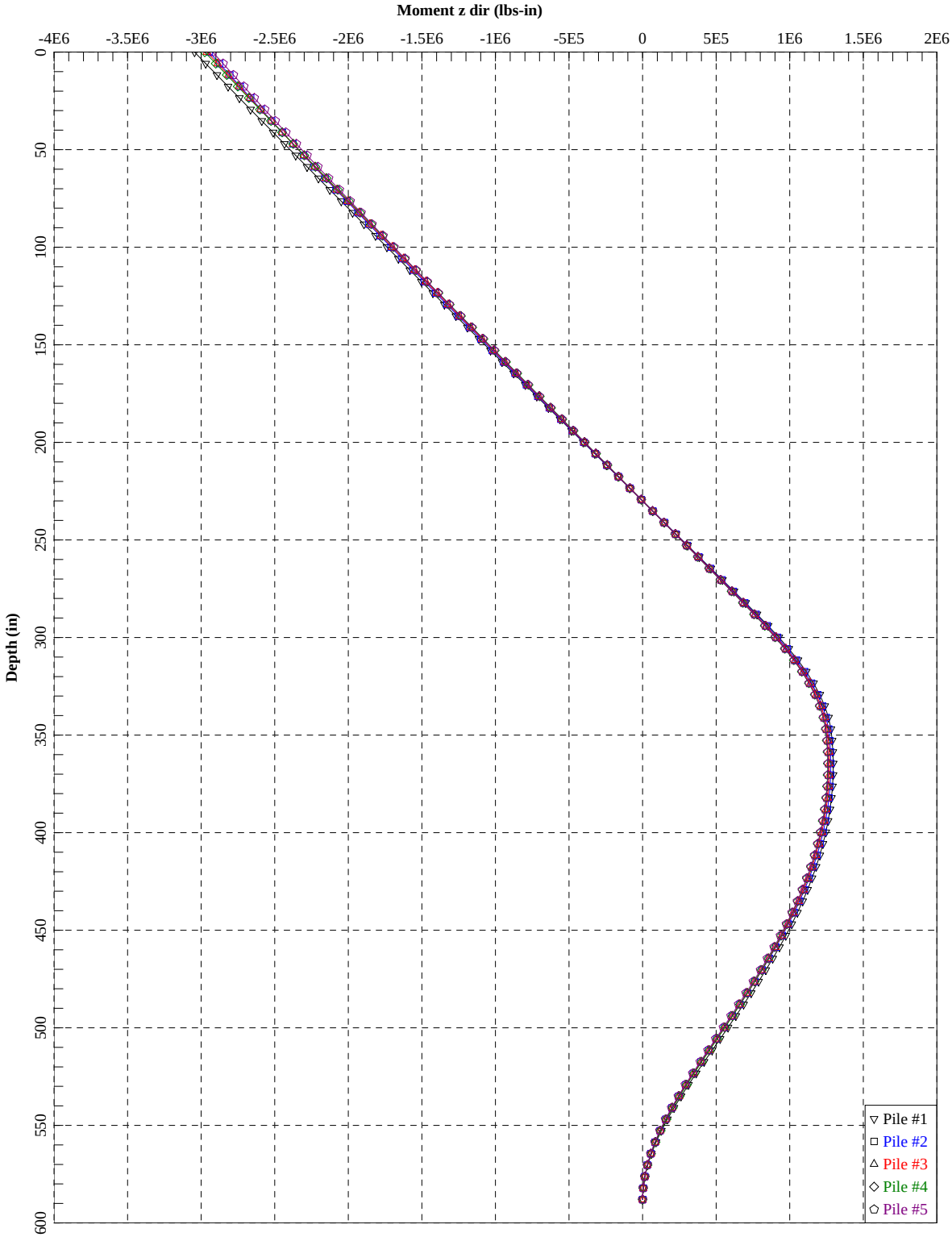
Max deflection=0.9"

Max stress=20 ksi

Extreme II Load Case

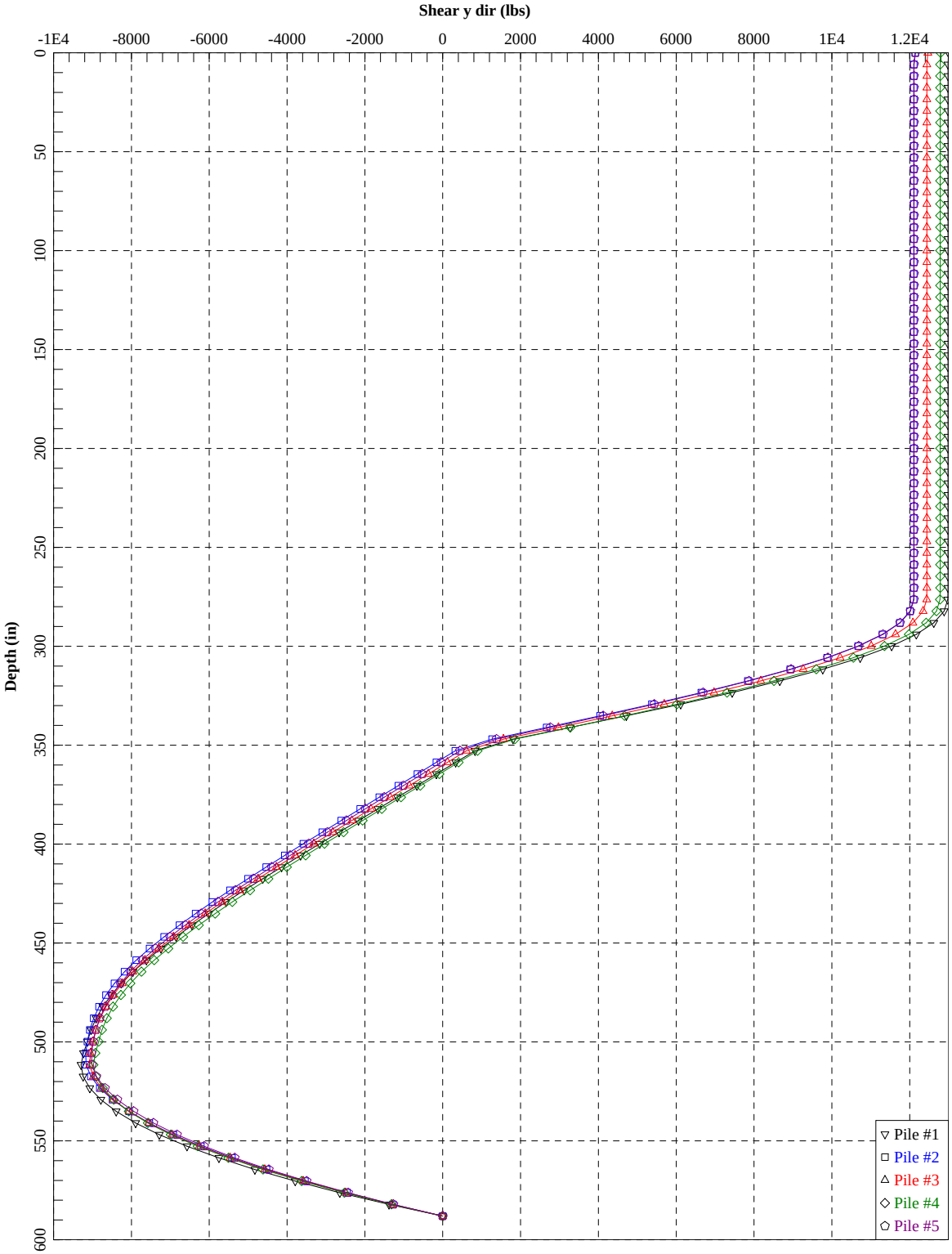


Extreme II Load Case



Load Case 1

Extreme II Load Case



Load Case 1

Pier 2
PP24x0.625 piles
Corroded Section - 1/8" loss on outside of the pile from bottom of cap to 5' below mudline
No concrete fill included
Extreme II and Service I Load Cases

```
                                                  25781 5 pile Bent Pier 2 - corroded section.gp8o
======================================================================

                 GROUP for Windows, Version 8.0.15

                  Serial Number :  364300562

                  Analysis of A Group of Piles
               Subjected to Axial and Lateral Loading

                (c) Copyright ENSOFT, Inc., 1987-2010
                      All Rights Reserved

======================================================================


This program is licensed to :

Jennifer Baron
GZA GeoEnvironmental, Inc.

Path to file locations      : P:\09 Jobs\0025700s\09.0025781.00 Thomaston Bridge MaineDOT\Work\Calcs\GROUP\Input_Output\
Name of input data file     : 25781 5 pile Bent Pier 2 - corroded section.gp8d
Name of output file         : 25781 5 pile Bent Pier 2 - corroded section.gp8o
Name of output summary file : 25781 5 pile Bent Pier 2 - corroded section.gp8t
Name of plot output file    : 25781 5 pile Bent Pier 2 - corroded section.gp8p
Name of runtime file        : 25781 5 pile Bent Pier 2 - corroded section.gp8r


----------------------------------------------------------------------
                      Time and Date of Analysis
----------------------------------------------------------------------

          Date:  April 14, 2014     Time:  09:36:41




           *****     INPUT INFORMATION     *****


Thomaston - corroded section

ANALYSIS TYPE = 3D ANALYSIS

UNITS SYSTEM  = ENGL


* TABLE C *  LOAD AND CONTROL PARAMETERS


** LOAD CASES **


NUMBER OF LOAD CASES :    2


LOAD CASE    :     1
CASE NAME    : extreme II
LOAD TYPE    : Dead, DL
SCALE FACTOR :       1.0000

* CONCENTRATED LOADS *

   NL    VERT.LOAD    HR.LOAD Y    HR.LOAD Z    MOMENT X     MOMENT Y     MOMENT Z      COORD X      COORD Y      COORD Z
           LBS          LBS          LBS         LBS-IN       LBS-IN       LBS-IN         IN           IN           IN
   1      39500.0      0.00000      0.00000      0.00000      0.00000      0.00000      0.00000     -180.000      0.00000
```

2	39500.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-90.0000	0.00000
3	39500.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
4	39500.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	90.0000	0.00000
5	39500.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	180.000	0.00000
6	6.21300E+05	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
7	3.01600E+05	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-30.0000	0.00000
8	82000.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	165.000	0.00000
9	0.00000	86000.0	0.00000	0.00000	0.00000	0.00000	120.000	-212.000	0.00000
10	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	-215.000	0.00000
11	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	-90.0000	0.00000
12	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	0.00000	0.00000
13	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	90.0000	0.00000
14	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	215.000	0.00000

* EQUIVALENT CONC. LOAD AT ORIGIN *

VER.LOAD X,LBS	HOR.LOAD Y,LBS	HOR.LOAD Z,LBS
1.20240E+06	1.02450E+05	0.00000

MOMENT X,LBS-IN	MOMENT Y,LBS-IN	MOMENT Z,LBS-IN
0.00000	0.00000	8.79900E+06

* THE LOADING IS STATIC *

* CONTROL PARAMETERS *

```
   TOLERANCE ON CONVERGENCE OF FOUNDATION REACTION,     =  1.00000E-04
   TOLERANCE ON DETERMINATION OF DEFLECTIONS            =  1.00000E-04 IN
   MAX NO OF ITERATIONS ALLOWED FOR FOUNDATION ANALYSIS =      100
   MAXIMUM NO OF ITERATIONS ALLOWED FOR PILE ANALYSIS   =      100
   FACTOR TO APPLY THE LOAD IN INCREMENTS               =   1.0000
   MINIMUM FACTOR FOR LOAD INCREMENTS                   =   1.0000
   PRINT RESULTS ONLY AT PILE CAP
```

```
LOAD CASE    :     2
CASE NAME    : Service I
LOAD TYPE    : Dead, DL
SCALE FACTOR :       1.0000
```

* CONCENTRATED LOADS *

NL	VERT.LOAD LBS	HR.LOAD Y LBS	HR.LOAD Z LBS	MOMENT X LBS-IN	MOMENT Y LBS-IN	MOMENT Z LBS-IN	COORD X IN	COORD Y IN	COORD Z IN
1	31600.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-180.000	0.00000
2	31600.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-90.0000	0.00000
3	31600.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
4	31600.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	90.0000	0.00000
5	31600.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	180.000	0.00000
6	4.97000E+05	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
7	3.81400E+05	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-30.0000	0.00000
8	65600.0	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	165.000	0.00000
9	0.00000	8750.00	0.00000	0.00000	0.00000	0.00000	-168.000	-30.0000	0.00000
10	0.00000	7860.00	0.00000	0.00000	0.00000	0.00000	-84.0000	-225.000	0.00000
11	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	-225.000	0.00000
12	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	-90.0000	0.00000
13	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	0.00000	0.00000
14	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	90.0000	0.00000
15	0.00000	3290.00	0.00000	0.00000	0.00000	0.00000	180.000	225.000	0.00000

* EQUIVALENT CONC. LOAD AT ORIGIN *

VER.LOAD X,LBS	HOR.LOAD Y,LBS	HOR.LOAD Z,LBS
1.10200E+06	33060.0	0.00000

MOMENT X,LBS-IN MOMENT Y,LBS-IN MOMENT Z,LBS-IN

```
                                                    25781 5 pile Bent Pier 2 - corroded section.gp8o
            0.00000          0.00000       1.44876E+06


* THE LOADING IS STATIC *


* CONTROL PARAMETERS *
    TOLERANCE ON CONVERGENCE OF FOUNDATION REACTION,     =  1.00000E-04
    TOLERANCE ON DETERMINATION OF DEFLECTIONS            =  1.00000E-04 IN
    MAX NO OF ITERATIONS ALLOWED FOR FOUNDATION ANALYSIS =      100
    MAXIMUM NO OF ITERATIONS ALLOWED FOR PILE ANALYSIS   =      100
    FACTOR TO APPLY THE LOAD IN INCREMENTS               =   1.0000
    MINIMUM FACTOR FOR LOAD INCREMENTS                   =   1.0000
    PRINT RESULTS ONLY AT PILE CAP


* TABLE D *   ARRANGEMENT OF PILE GROUPS

  GROUP  CONNECTY CONNECTZ  PILE PROP   P-Y CURVE  L-S CURVE  T-R CURVE
    1      FIX      FIX         1          0          0          0
    2      FIX      FIX         1          0          0          0
    3      FIX      FIX         1          0          0          0
    4      FIX      FIX         1          0          0          0
    5      FIX      FIX         1          0          0          0

  GROUP     CorX,IN      CorY,IN      CorZ,IN     ALPHA,DEG     BETA,DEG   GROUND,IN  SPR.y,LBS-IN SPR.z,LBS-IN
    1        0.00000     -180.000      0.00000      0.00000      101.770      336.000      0.00000      0.00000
    2        0.00000     -90.0000      0.00000      0.00000      90.0000      336.000      0.00000      0.00000
    3        0.00000      0.00000      0.00000      0.00000      90.0000      336.000      0.00000      0.00000
    4        0.00000      90.0000      0.00000      0.00000      90.0000      336.000      0.00000      0.00000
    5        0.00000      180.000      0.00000      0.00000      78.2300      336.000      0.00000      0.00000


* TABLE E *   PILE GEOMETRY AND PROPERTIES
             PILE TYPE = 1 - DRIVEN PILE
                       = 2 - DRILLED SHAFT

  PROP   SECTS   INC     PILE TYPE      LENGTH, IN        E,LBS/IN**2
    1      2     100          1             564.00        2.9000E+07

* PILE SECTIONS *

  PROP    SECT      FROM,IN         TO,IN    CROSS SECT
    1       1        0.00000        396.000       2
    1       2        396.000        564.000       1

* PILE CROSS SECTIONS *

 CROSS SECTION       :     1
 SECTION NAME        : 24"
 CROSS SECTION TYPE  : CIRCULAR PIPE
 EXTERNAL DIAMETER   :       24.0000 IN
 INTERNAL DIAMETER   :       22.7500 IN
 SHEAR MODULUS       :    1.0000     LBS/IN**2
 HEAR MODULUS       : ,      1.00000 LBS/IN**2

 CROSS SECTION       :     2
 SECTION NAME        : Corroded
 CROSS SECTION TYPE  : CIRCULAR PIPE
 EXTERNAL DIAMETER   :       23.7500 IN
 INTERNAL DIAMETER   :       22.7500 IN
 SHEAR MODULUS       :    1.0000     LBS/IN**2
 HEAR MODULUS       : ,      1.00000 LBS/IN**2

* PILE CROSS SECTIONS PROPERTIES *

  SECT      DIAM,IN     AREA,IN**2     Iz,IN**4      Iy,IN**4      GJ,LBS-IN**2     Mn,LBS-IN      Vn,LBS
```

```
    1        24.0000       45.8967       3136.93       3136.93     7.27768E+10       0.00000       0.00000
    2        23.7500       36.5210       2468.88       2468.88         4937.76       0.00000       0.00000


* TABLE F *   SOIL DATA


SOILS INFORMATION

    GROUND SURFACE          =      336.000  IN

    3 LAYER(S) OF SOIL

    LAYER  1
    THE SOIL IS A SAND
                                                      TOP OF LAYER    BOTTOM OF LAYER
    X COORDINATE                          (IN)           336.000           504.000
    EFFECTIVE UNIT WEIGHT         (LBS/IN**3)        0.0333000         0.0333000
    FRICTION ANGLE                  (DEGREES)           30.0000           30.0000
    P-Y SUBGRADE MODULUS          (LBS/IN**3)           20.0000           20.0000
    ULTIMATE UNIT SIDE FRICTION   (LBS/IN**2)           0.00000           0.00000
    ULTIMATE UNIT TIP RESISTANCE  (LBS/IN**2)           0.00000           0.00000

    LAYER  2
    THE SOIL IS A SAND
                                                      TOP OF LAYER    BOTTOM OF LAYER
    X COORDINATE                          (IN)           504.000           564.000
    EFFECTIVE UNIT WEIGHT         (LBS/IN**3)        0.0360000         0.0360000
    FRICTION ANGLE                  (DEGREES)           38.0000           38.0000
    P-Y SUBGRADE MODULUS          (LBS/IN**3)           90.0000           90.0000
    ULTIMATE UNIT SIDE FRICTION   (LBS/IN**2)           0.00000           0.00000
    ULTIMATE UNIT TIP RESISTANCE  (LBS/IN**2)           0.00000           0.00000

    LAYER  3
    THE LAYER IS A VUGGY LIMESTONE
                                                      TOP OF LAYER    BOTTOM OF LAYER
    X COORDINATE                          (IN)           564.000           600.000
    EFFECTIVE UNIT WEIGHT         (LBS/IN**3)        0.0560000         0.0560000
    UNIAXIAL COMPRESSIVE STRENGTH (LBS/IN**2)           4000.00           4000.00
    ULTIMATE UNIT SIDE FRICTION   (LBS/IN**2)           0.00000           0.00000
    ULTIMATE UNIT TIP RESISTANCE  (LBS/IN**2)           17000.0           17000.0

  Note :  Default values will be generated for
          ULTIMATE UNIT SIDE FRICTION  and ULTIMATE UNIT TIP RESISTANCE
          when input values are 0.


* TABLE H *   AXIAL LOAD VS SETTLEMENT

LOAD-SETTLEMENT CURVES GENERATED INTERNALLY

  NUM OF CURVES  1

   CURVE  1          NUM OF POINTS 19

       POINT        AXIAL LOAD,LBS       SETTLEMENT, IN
         1           -1.00587E+05             -2.04302
         2               -94711.6             -1.04051
         3               -91773.9             -0.53926
         4               -58537.5             -0.12503
         5               -40705.5           -0.0673909
         6               -11988.6           -0.0151153
         7               -6033.84         -7.57229E-03
         8               -1206.77         -1.51446E-03
         9               -120.677         -1.51446E-04
        10                0.00000              0.00000
        11                2609.05          1.37428E-03
```

River Bottom

Till

Rock (Tip of Pile at Top of Rock), 47' below top of pile: El. -38

12	26090.5	0.0137428
13	1.29536E+05	0.0684013
14	2.55140E+05	0.13527
15	6.17288E+05	0.35421
16	8.48335E+05	0.51892
17	8.72349E+05	0.92942
18	8.75287E+05	1.43068
19	8.81162E+05	2.43319

* TABLE I * TORS. MOM. VS ANGLE ROT.

TORQUE-ROTATION CURVES GENERATED INTERNALLY

NUM OF CURVES 1

CURVE 1 NUM OF POINTS 19

POINT	TORS.MOMEN,LBS/IN	ROT. ANGLE,Rad.
1	-91.0458	-7.44256
2	-91.0450	-7.35829
3	-91.0446	-7.31615
4	-91.0359	-7.28177
5	-91.0251	-7.27670
6	-90.9433	-7.26679
7	-45.5734	-3.63880
8	-9.27544	-0.73625
9	-1.10654	-0.0830232
10	0.00000	0.00000
11	1.10654	0.0830232
12	9.27544	0.73625
13	45.5734	3.63880
14	90.9433	7.26679
15	91.0251	7.27670
16	91.0359	7.28177
17	91.0446	7.31615
18	91.0450	7.35829
19	91.0458	7.44256

***** COMPUTATION RESULTS *****

Thomaston - corroded section

***** LOAD CASES RESULTS *****

LOAD CASE : 1
CASE NAME : extreme II
LOAD TYPE : Dead, DL

* TABLE L * COMPUTATION ON PILE CAP

* EQUIVALENT CONC. LOAD AT ORIGIN *

VERT. LOAD, LBS HOR. LOAD Y, LBS HOR. LOAD Z, LBS

```
   1.20240E+06        1.02450E+05            0.00000

MOMENT X ,IN-LBS    MOMENT Y,IN-LBS    MOMENT Z,IN-LBS
       0.00000            0.00000        8.79900E+06


* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *


  VERTICAL ,IN      HORIZONTAL Y,IN     HORIZONTAL Z,IN
       0.13362            1.32766            0.00000

ANGLE ROT. X,RAD    ANGLE ROT. Y,RAD    ANGLE ROT. Z,RAD
       0.00000            0.00000        1.16899E-03


NUMBER OF GLOBAL ITERATIONS =    4


LOAD CASE :     2
CASE NAME : Service I
LOAD TYPE : Dead, DL


* TABLE L *   COMPUTATION ON PILE CAP


* EQUIVALENT CONC. LOAD AT ORIGIN *


 VERT. LOAD, LBS    HOR. LOAD Y, LBS   HOR. LOAD Z, LBS
   1.10200E+06           33060.0            0.00000

MOMENT X ,IN-LBS    MOMENT Y,IN-LBS    MOMENT Z,IN-LBS
       0.00000            0.00000        1.44876E+06


* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *


  VERTICAL ,IN      HORIZONTAL Y,IN     HORIZONTAL Z,IN
       0.11918            0.40407            0.00000

ANGLE ROT. X,RAD    ANGLE ROT. Y,RAD    ANGLE ROT. Z,RAD
       0.00000            0.00000        3.49417E-04


NUMBER OF GLOBAL ITERATIONS =    4
```

25781 5 pile Bent Pier 2 - corroded section.gp8t

Time and Date of Analysis

Date: April 14, 2014 Time: 09:36:41

***** COMPUTATION RESULTS *****

Thomaston - corroded section

***** LOAD CASES RESULTS *****

LOAD CASE : 1
CASE NAME : extreme II
LOAD TYPE : Dead, DL

* TABLE L * COMPUTATION ON PILE CAP

* EQUIVALENT CONC. LOAD AT ORIGIN *

VERT. LOAD, LBS	HOR. LOAD Y, LBS	HOR. LOAD Z, LBS
1.20240E+06	1.02450E+05	0.00000

MOMENT X ,IN-LBS	MOMENT Y,IN-LBS	MOMENT Z,IN-LBS
0.00000	0.00000	8.79900E+06

* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *

VERTICAL ,IN	HORIZONTAL Y,IN	HORIZONTAL Z,IN
0.13362	1.32766	0.00000

ANGLE ROT. X,RAD	ANGLE ROT. Y,RAD	ANGLE ROT. Z,RAD
0.00000	0.00000	1.16899E-03

THE GLOBAL STRUCTURAL COORDINATE SYSTEM

---

* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. X,IN	DISP. Y,IN	DISP. Z,IN	ROT. X,RAD	ROT. Y,RAD	ROT. Z,RAD
1	0.3440	1.3277	0.0000	0.0000	0.0000	1.1690E-03
2	0.2388	1.3277	0.0000	0.0000	0.0000	1.1690E-03
3	0.1336	1.3277	0.0000	0.0000	0.0000	1.1690E-03
4	2.8411E-02	1.3277	0.0000	0.0000	0.0000	1.1690E-03
5	-7.6798E-02	1.3277	0.0000	0.0000	0.0000	1.1690E-03
MINIMUM	-7.6798E-02	1.3277	0.0000	0.0000	0.0000	1.1690E-03
Pile N.	5	1	1	1	1	1
MAXIMUM	0.3440	1.3277	0.0000	0.0000	0.0000	1.1690E-03
Pile N.	1	1	1	1	1	1

25781 5 pile Bent Pier 2 - corroded section.gp8t

* PILE TOP REACTIONS *

PILE GROUP	FOR. X,LBS	FOR. Y,LBS	FOR. Z,LBS	MOM X,LBS-IN	MOM Y,LBS-IN	MOM Z,LBS-IN	STRESS,LBS/IN**2
1	1.2474E+05	-1.3940E+04	0.0000	0.0000	0.0000	3.0753E+06	1.8214E+04
2	4.2643E+05	1.0579E+04	0.0000	0.0000	0.0000	2.9505E+06	2.5868E+04
3	2.5204E+05	1.1132E+04	0.0000	0.0000	0.0000	2.9832E+06	2.1250E+04
4	5.3851E+04	1.1760E+04	0.0000	0.0000	0.0000	3.0199E+06	1.6000E+04
5	3.4533E+05	8.2919E+04	0.0000	0.0000	0.0000	2.9440E+06	2.3880E+04
MINIMUM	5.3851E+04	-1.3940E+04	0.0000	0.0000	0.0000	2.9440E+06	1.6000E+04
Pile N.	4	1	1	1	1	5	4
MAXIMUM	4.2643E+05	8.2919E+04	0.0000	0.0000	0.0000	3.0753E+06	2.5868E+04
Pile N.	2	5	1	1	1	1	2

Max pile stress 26 ksi

THE PILE COORDINATE SYSTEM (LOCAL AXES)

* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. x,IN	DISP. y,IN	DISP. z,IN	ROT. x,RAD	ROT. y,RAD	ROT. z,RAD
1	6.5986E-02	1.3699	0.0000	0.0000	0.0000	1.1690E-03
2	0.2388	1.3277	0.0000	0.0000	0.0000	1.1690E-03
3	0.1336	1.3277	0.0000	0.0000	0.0000	1.1690E-03
4	2.8411E-02	1.3277	0.0000	0.0000	0.0000	1.1690E-03
5	0.1956	1.3154	0.0000	0.0000	0.0000	1.1690E-03
MINIMUM	2.8411E-02	1.3154	0.0000	0.0000	0.0000	1.1690E-03
Pile N.	4	5	1	1	1	1
MAXIMUM	0.2388	1.3699	0.0000	0.0000	0.0000	1.1690E-03
Pile N.	2	1	1	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	AXIAL,LBS	LAT. y,LBS	LAT. z,LBS	MOM x,LBS-IN	MOM y,LBS-IN	MOM z,LBS-IN	STRESS,LBS/IN**2
1	1.2496E+05	1.1799E+04	0.0000	0.0000	0.0000	3.0753E+06	1.8214E+04
2	4.2643E+05	1.0579E+04	0.0000	0.0000	0.0000	2.9505E+06	2.5868E+04
3	2.5204E+05	1.1132E+04	0.0000	0.0000	0.0000	2.9832E+06	2.1250E+04
4	5.3851E+04	1.1760E+04	0.0000	0.0000	0.0000	3.0199E+06	1.6000E+04
5	3.5499E+05	1.0733E+04	0.0000	0.0000	0.0000	2.9440E+06	2.3880E+04
MINIMUM	5.3851E+04	1.0579E+04	0.0000	0.0000	0.0000	2.9440E+06	1.6000E+04
Pile N.	4	2	1	1	1	5	4
MAXIMUM	4.2643E+05	1.1799E+04	0.0000	0.0000	0.0000	3.0753E+06	2.5868E+04
Pile N.	2	1	1	1	1	1	2

* EFFECTS FOR LATERALLY LOADED PILE *

* MINIMUM VALUES AND LOCATIONS *

PILE	DEFLECTION y-DIR IN	DEFLECTION z-DIR IN	BENDING MOMENT z-DIR LBS-IN	BENDING MOMENT y-DIR LBS-IN	SHEAR FORCE y-DIR LBS	SHEAR FORCE z-DIR LBS	SOIL REACTION y-DIR LBS/IN	SOIL REACTION z-DIR LBS/IN	TOTAL STRESS LBS/IN**2	FLEXURAL RIGIDITY z-DIR LBS-IN**2	FLEXURAL RIGIDITY y-DIR LBS-IN**2
1	-4.4100E-02	0.0000	-3.0800E+06	0.0000	-1.6300E+04	0.0000	-688.00	0.0000	2720.0	7.1600E+10	7.1600E+10
x(IN)	564.00	0.0000	0.0000	0.0000	513.24	0.0000	564.00	0.0000	564.00	0.0000	0.0000
2	-4.2800E-02	0.0000	-2.9500E+06	0.0000	-1.6200E+04	0.0000	-675.00	0.0000	9290.0	7.1600E+10	7.1600E+10
x(IN)	564.00	0.0000	0.0000	0.0000	513.24	0.0000	564.00	0.0000	564.00	0.0000	0.0000
3	-4.2600E-02	0.0000	-2.9800E+06	0.0000	-1.6000E+04	0.0000	-674.00	0.0000	5490.0	7.1600E+10	7.1600E+10
x(IN)	564.00	0.0000	0.0000	0.0000	513.24	0.0000	564.00	0.0000	564.00	0.0000	0.0000
4	-4.2400E-02	0.0000	-3.0200E+06	0.0000	-1.5700E+04	0.0000	-672.00	0.0000	1170.0	7.1600E+10	7.1600E+10
x(IN)	564.00	0.0000	0.0000	0.0000	513.24	0.0000	564.00	0.0000	564.00	0.0000	0.0000
5	-4.2300E-02	0.0000	-2.9400E+06	0.0000	-1.6000E+04	0.0000	-670.00	0.0000	7730.0	7.1600E+10	7.1600E+10
x(IN)	564.00	0.0000	0.0000	0.0000	513.24	0.0000	564.00	0.0000	564.00	0.0000	0.0000
Min.	-4.4100E-02	0.0000	-3.0800E+06	0.0000	-1.6300E+04	0.0000	-688.00	0.0000	1170.0	7.1600E+10	7.1600E+10

Page 2

25781 5 pile Bent Pier 2 - corroded section.gp8t

Pile N.	1	1	1	1	1	1	1	1	4	1	1

* MAXIMUM VALUES AND LOCATIONS *

PILE	DEFLECTION y-DIR IN	DEFLECTION z-DIR IN	BENDING MOMENT z-DIR LBS-IN	BENDING MOMENT y-DIR LBS-IN	SHEAR FORCE y-DIR LBS	SHEAR FORCE z-DIR LBS	SOIL REACTION y-DIR LBS/IN	SOIL REACTION z-DIR LBS/IN	TOTAL STRESS LBS/IN**2	FLEXURAL RIGIDITY z-DIR LBS-IN**2	FLEXURAL RIGIDITY y-DIR LBS-IN**2
1	1.3700	0.0000	1.5900E+06	0.0000	1.1800E+04	0.0000	247.00	0.0000	1.8200E+04	9.1000E+10	9.1000E+10
x(IN)	28.200	0.0000	411.72	0.0000	0.0000	0.0000	406.08	0.0000	0.0000	400.44	400.44
2	1.3300	0.0000	1.5700E+06	0.0000	1.0600E+04	0.0000	236.00	0.0000	2.5900E+04	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	411.72	0.0000	0.0000	0.0000	406.08	0.0000	0.0000	400.44	400.44
3	1.3300	0.0000	1.5600E+06	0.0000	1.1200E+04	0.0000	238.00	0.0000	2.1200E+04	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	411.72	0.0000	0.0000	0.0000	406.08	0.0000	0.0000	400.44	400.44
4	1.3300	0.0000	1.5500E+06	0.0000	1.1800E+04	0.0000	240.00	0.0000	1.6000E+04	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	411.72	0.0000	0.0000	0.0000	406.08	0.0000	0.0000	400.44	400.44
5	1.3200	0.0000	1.5500E+06	0.0000	1.0800E+04	0.0000	234.00	0.0000	2.3900E+04	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	411.72	0.0000	0.0000	0.0000	406.08	0.0000	0.0000	400.44	400.44
Max.	1.3700	0.0000	1.5900E+06	0.0000	1.1800E+04	0.0000	247.00	0.0000	2.5900E+04	9.1000E+10	9.1000E+10
Pile N.	1	1	1	1	1	1	1	1	2	1	1

LOAD CASE : 2
CASE NAME : Service I
LOAD TYPE : Dead, DL

* TABLE L * COMPUTATION ON PILE CAP

* EQUIVALENT CONC. LOAD AT ORIGIN *

Max deflection=1.4"

Max stress=26 ksi

VERT. LOAD, LBS	HOR. LOAD Y, LBS	HOR. LOAD Z, LBS
1.10200E+06	33060.0	0.00000

MOMENT X ,IN-LBS	MOMENT Y,IN-LBS	MOMENT Z,IN-LBS
0.00000	0.00000	1.44876E+06

* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *

VERTICAL ,IN	HORIZONTAL Y,IN	HORIZONTAL Z,IN
0.11918	0.40407	0.00000

ANGLE ROT. X,RAD	ANGLE ROT. Y,RAD	ANGLE ROT. Z,RAD
0.00000	0.00000	3.49417E-04

THE GLOBAL STRUCTURAL COORDINATE SYSTEM
---

* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. X,IN	DISP. Y,IN	DISP. Z,IN	ROT. X,RAD	ROT. Y,RAD	ROT. Z,RAD
1	0.1821	0.4041	0.0000	0.0000	0.0000	3.4942E-04
2	0.1506	0.4041	0.0000	0.0000	0.0000	3.4942E-04
3	0.1192	0.4041	0.0000	0.0000	0.0000	3.4942E-04
4	8.7728E-02	0.4041	0.0000	0.0000	0.0000	3.4942E-04
5	5.6280E-02	0.4041	0.0000	0.0000	0.0000	3.4942E-04
MINIMUM	5.6280E-02	0.4041	0.0000	0.0000	0.0000	3.4942E-04
Pile N.	5	1	1	1	1	1
MAXIMUM	0.1821	0.4041	0.0000	0.0000	0.0000	3.4942E-04

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25781 5 pile Bent Pier 2 - corroded section.gp8t

Pile N.	1	1	1	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	FOR. X,LBS	FOR. Y,LBS	FOR. Z,LBS	MOM X,LBS-IN	MOM Y,LBS-IN	MOM Z,LBS-IN	STRESS,LBS/IN**2
1	1.7798E+05	-3.3309E+04	0.0000	0.0000	0.0000	9.6685E+05	9607.4
2	2.8053E+05	3396.6	0.0000	0.0000	0.0000	9.1030E+05	1.2060E+04
3	2.2490E+05	3450.8	0.0000	0.0000	0.0000	9.1347E+05	1.0552E+04
4	1.6584E+05	3508.3	0.0000	0.0000	0.0000	9.1683E+05	8950.7
5	2.5274E+05	5.6013E+04	0.0000	0.0000	0.0000	8.7618E+05	1.1302E+04
MINIMUM	1.6584E+05	-3.3309E+04	0.0000	0.0000	0.0000	8.7618E+05	8950.7
Pile N.	4	1	1	1	1	5	4
MAXIMUM	2.8053E+05	5.6013E+04	0.0000	0.0000	0.0000	9.6685E+05	1.2060E+04
Pile N.	2	5	1	1	1	1	2

THE PILE COORDINATE SYSTEM (LOCAL AXES)
---

* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. x,IN	DISP. y,IN	DISP. z,IN	ROT. x,RAD	ROT. y,RAD	ROT. z,RAD
1	9.5819E-02	0.4327	0.0000	0.0000	0.0000	3.4942E-04
2	0.1506	0.4041	0.0000	0.0000	0.0000	3.4942E-04
3	0.1192	0.4041	0.0000	0.0000	0.0000	3.4942E-04
4	8.7728E-02	0.4041	0.0000	0.0000	0.0000	3.4942E-04
5	0.1375	0.3841	0.0000	0.0000	0.0000	3.4942E-04
MINIMUM	8.7728E-02	0.3841	0.0000	0.0000	0.0000	3.4942E-04
Pile N.	4	5	1	1	1	1
MAXIMUM	0.1506	0.4327	0.0000	0.0000	0.0000	3.4942E-04
Pile N.	2	1	1	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	AXIAL,LBS	LAT. y,LBS	LAT. z,LBS	MOM x,LBS-IN	MOM y,LBS-IN	MOM z,LBS-IN	STRESS,LBS/IN**2
1	1.8104E+05	3697.2	0.0000	0.0000	0.0000	9.6685E+05	9607.4
2	2.8053E+05	3396.6	0.0000	0.0000	0.0000	9.1030E+05	1.2060E+04
3	2.2490E+05	3450.8	0.0000	0.0000	0.0000	9.1347E+05	1.0552E+04
4	1.6584E+05	3508.3	0.0000	0.0000	0.0000	9.1683E+05	8950.7
5	2.5886E+05	3279.4	0.0000	0.0000	0.0000	8.7618E+05	1.1302E+04
MINIMUM	1.6584E+05	3279.4	0.0000	0.0000	0.0000	8.7618E+05	8950.7
Pile N.	4	5	1	1	1	5	4
MAXIMUM	2.8053E+05	3697.2	0.0000	0.0000	0.0000	9.6685E+05	1.2060E+04
Pile N.	2	1	1	1	1	1	2

* EFFECTS FOR LATERALLY LOADED PILE *

* MINIMUM VALUES AND LOCATIONS *

PILE	DEFLECTION		BENDING MOMENT		SHEAR FORCE		SOIL REACTION		TOTAL	FLEXURAL RIGIDITY	
	y-DIR	z-DIR	z-DIR	y-DIR	y-DIR	z-DIR	y-DIR	z-DIR	STRESS	z-DIR	y-DIR
	IN	IN	LBS-IN	LBS-IN	LBS	LBS	LBS/IN	LBS/IN	LBS/IN**2	LBS-IN**2	LBS-IN**2
1	-1.3300E-02	0.0000	-9.6700E+05	0.0000	-5100.0	0.0000	-224.00	0.0000	3940.0	7.1600E+10	7.1600E+10
x(IN)	564.00	0.0000	0.0000	0.0000	513.24	0.0000	564.00	0.0000	564.00	0.0000	0.0000
2	-1.2500E-02	0.0000	-9.1000E+05	0.0000	-4840.0	0.0000	-211.00	0.0000	6110.0	7.1600E+10	7.1600E+10
x(IN)	564.00	0.0000	0.0000	0.0000	513.24	0.0000	564.00	0.0000	564.00	0.0000	0.0000
3	-1.2500E-02	0.0000	-9.1300E+05	0.0000	-4820.0	0.0000	-211.00	0.0000	4900.0	7.1600E+10	7.1600E+10
x(IN)	564.00	0.0000	0.0000	0.0000	513.24	0.0000	564.00	0.0000	564.00	0.0000	0.0000
4	-1.2500E-02	0.0000	-9.1700E+05	0.0000	-4790.0	0.0000	-211.00	0.0000	3610.0	7.1600E+10	7.1600E+10
x(IN)	564.00	0.0000	0.0000	0.0000	513.24	0.0000	564.00	0.0000	564.00	0.0000	0.0000
5	-1.2000E-02	0.0000	-8.7600E+05	0.0000	-4620.0	0.0000	-202.00	0.0000	5640.0	7.1600E+10	7.1600E+10
x(IN)	564.00	0.0000	0.0000	0.0000	513.24	0.0000	564.00	0.0000	564.00	0.0000	0.0000

25781 5 pile Bent Pier 2 - corroded section.gp8t

Min.	-1.3300E-02	0.0000	-9.6700E+05	0.0000	-5100.0	0.0000	-224.00	0.0000	3610.0	7.1600E+10	7.1600E+10
Pile N.	1	1	1	1	1	1	1	1	4	1	1

* MAXIMUM VALUES AND LOCATIONS *

PILE	DEFLECTION		BENDING MOMENT		SHEAR FORCE		SOIL REACTION		TOTAL	FLEXURAL RIGIDITY	
	y-DIR	z-DIR	z-DIR	y-DIR	y-DIR	z-DIR	y-DIR	z-DIR	STRESS	z-DIR	y-DIR
	IN	IN	LBS-IN	LBS-IN	LBS	LBS	LBS/IN	LBS/IN	LBS/IN**2	LBS-IN**2	LBS-IN**2
*****	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********	**********
1	0.4330	0.0000	5.0600E+05	0.0000	3700.0	0.0000	75.100	0.0000	9610.0	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	406.08	0.0000	0.0000	0.0000	406.08	0.0000	0.0000	400.44	400.44
2	0.4040	0.0000	4.7700E+05	0.0000	3410.0	0.0000	70.400	0.0000	1.2100E+04	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	406.08	0.0000	0.0000	0.0000	406.08	0.0000	0.0000	400.44	400.44
3	0.4040	0.0000	4.7600E+05	0.0000	3460.0	0.0000	70.600	0.0000	1.0600E+04	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	406.08	0.0000	0.0000	0.0000	406.08	0.0000	0.0000	400.44	400.44
4	0.4040	0.0000	4.7500E+05	0.0000	3510.0	0.0000	70.800	0.0000	8950.0	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	411.72	0.0000	0.0000	0.0000	406.08	0.0000	0.0000	400.44	400.44
5	0.3840	0.0000	4.5500E+05	0.0000	3290.0	0.0000	67.500	0.0000	1.1300E+04	9.1000E+10	9.1000E+10
x(IN)	0.0000	0.0000	411.72	0.0000	0.0000	0.0000	406.08	0.0000	0.0000	400.44	400.44
Max.	0.4330	0.0000	5.0600E+05	0.0000	3700.0	0.0000	75.100	0.0000	1.2100E+04	9.1000E+10	9.1000E+10
Pile N.	1	1	1	1	1	1	1	1	2	1	1

Max deflection=0.4"

Max stress=12 ksi

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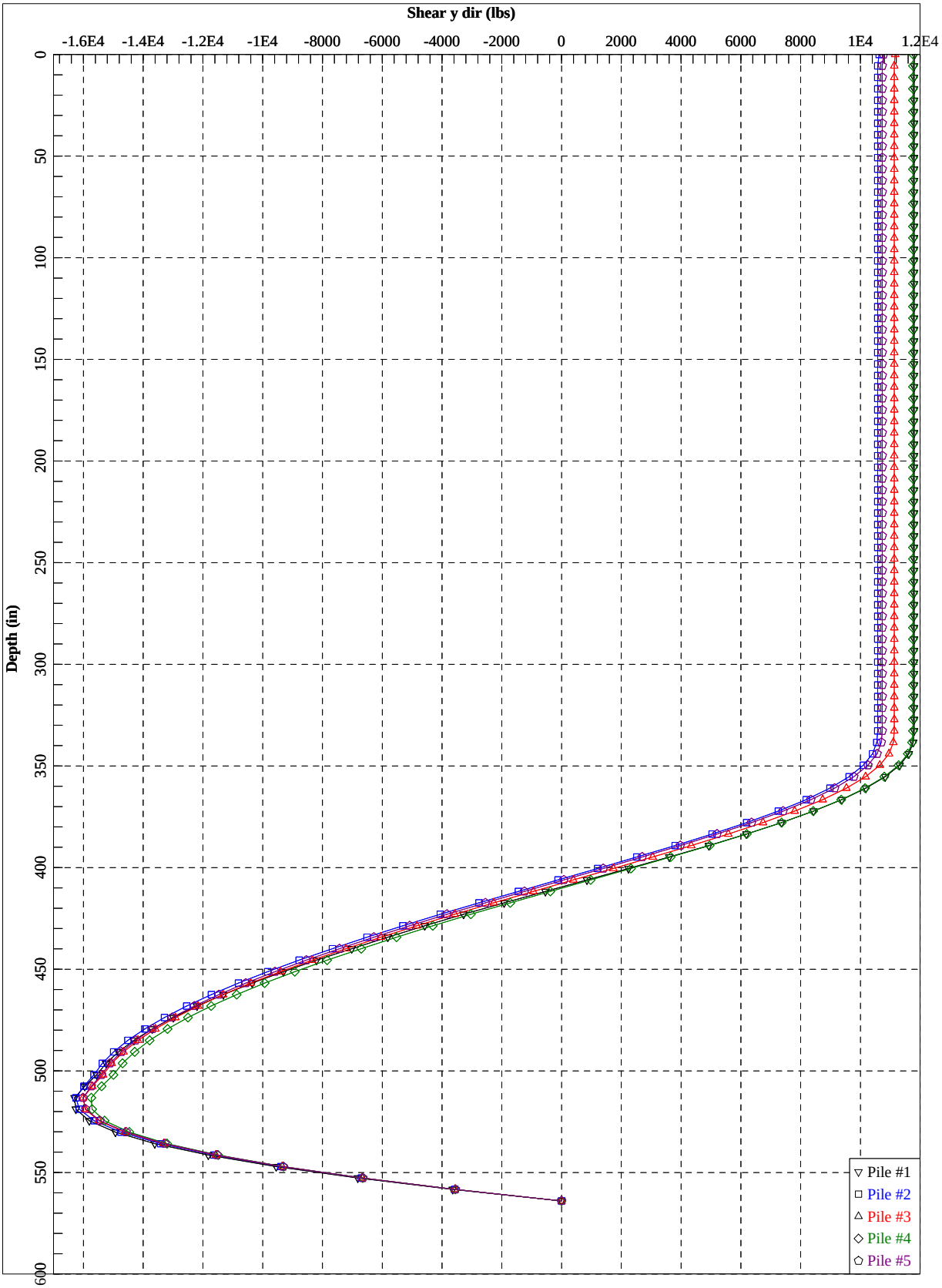
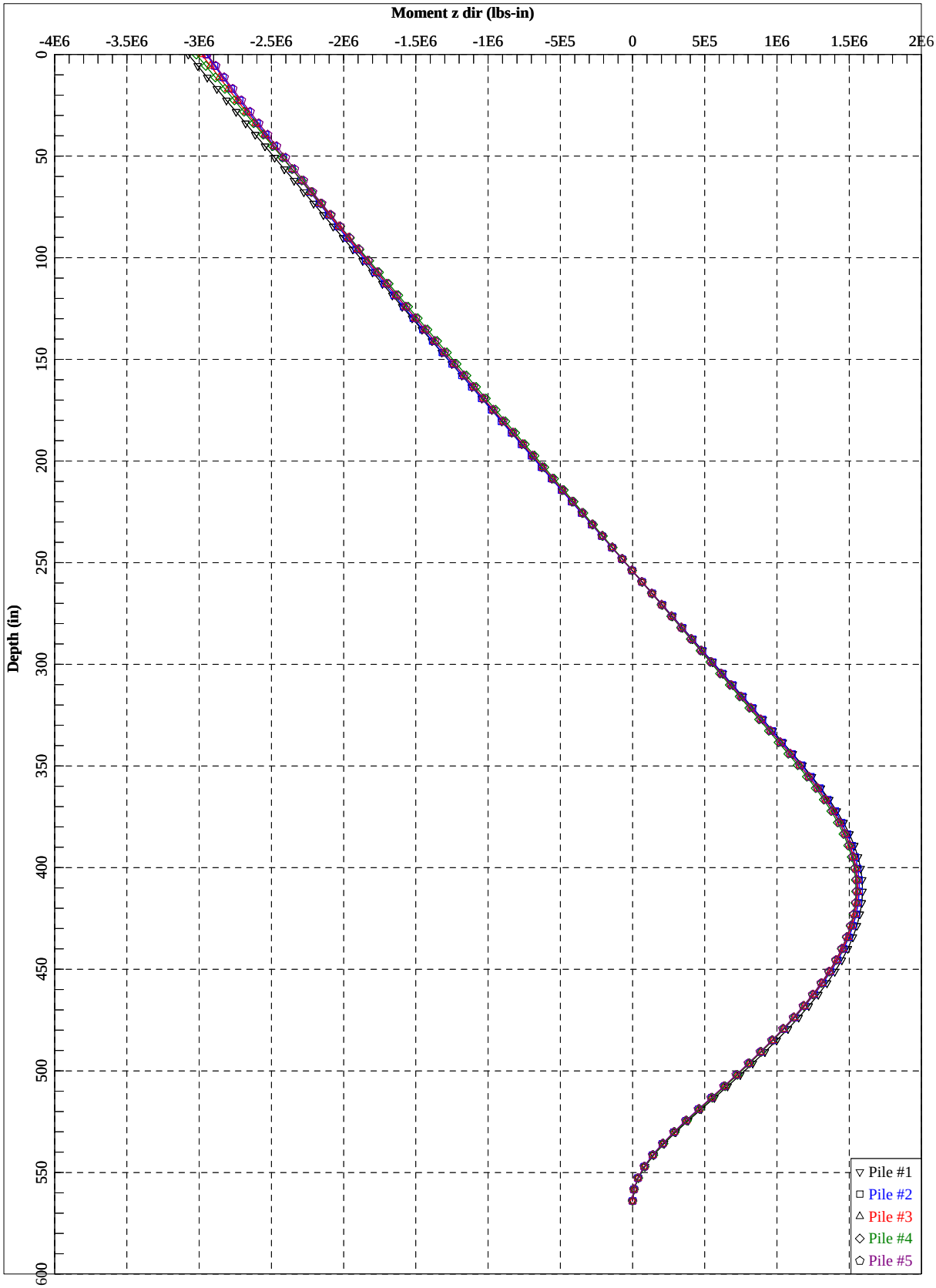
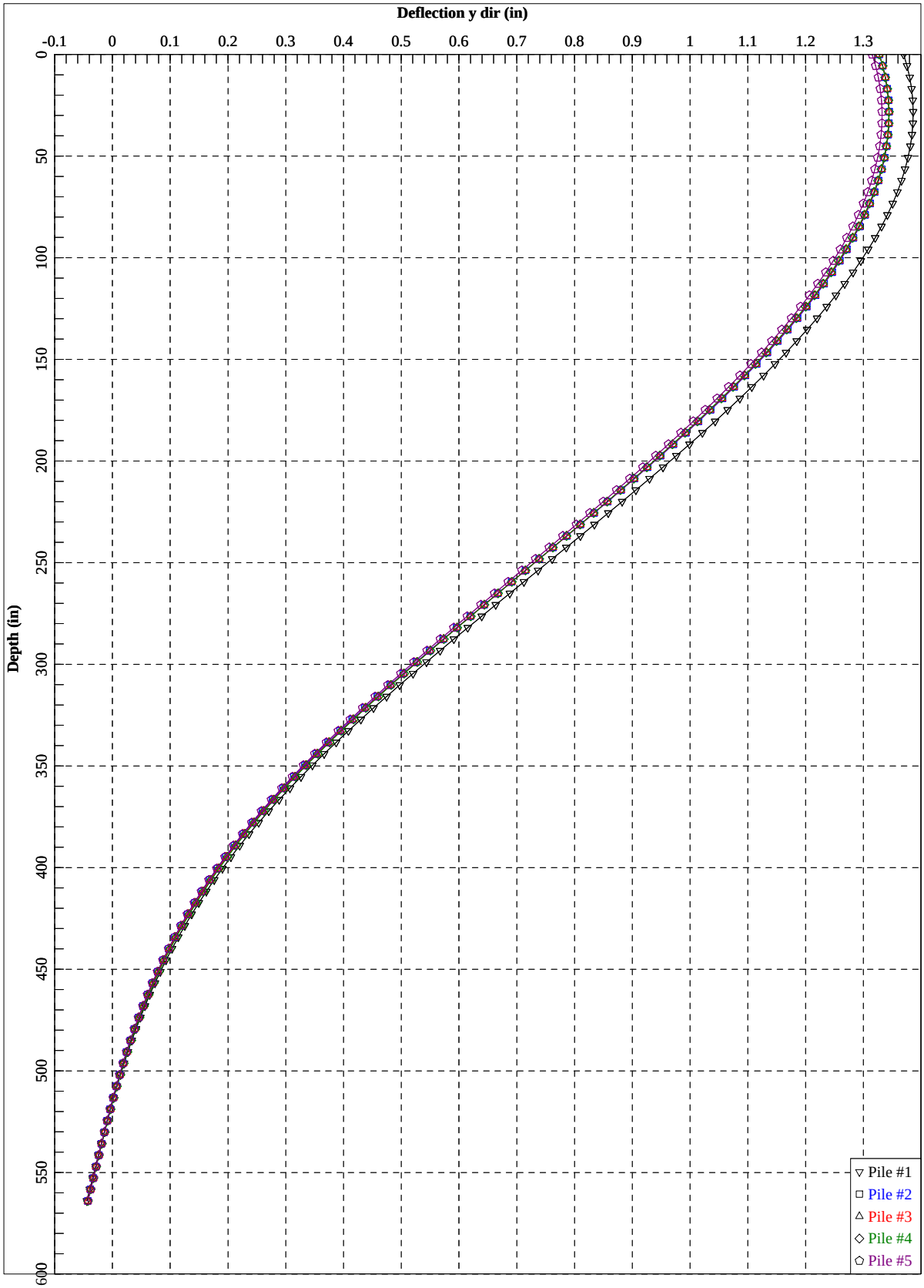
Extreme II Load Case



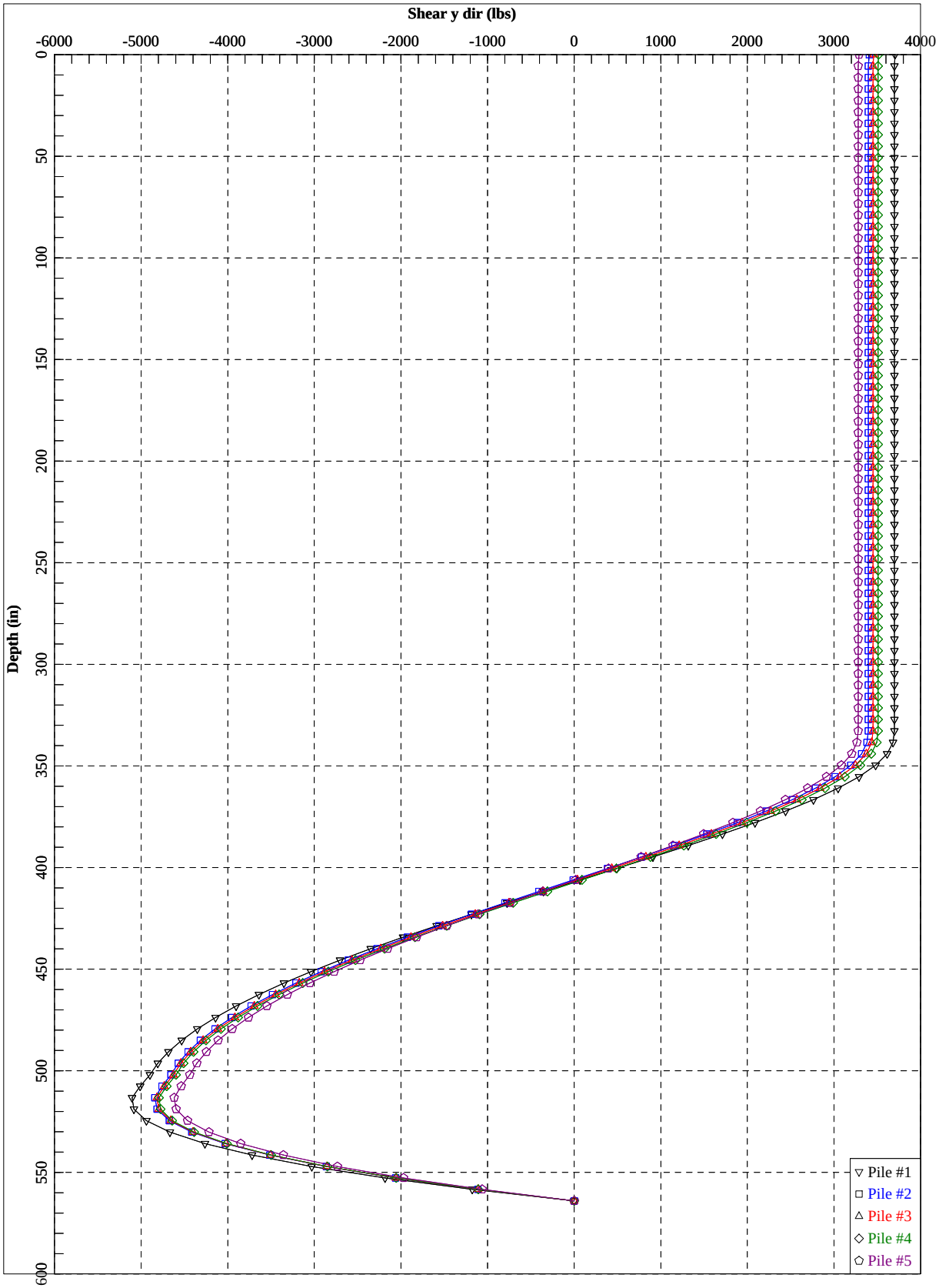
Extreme II Load Case



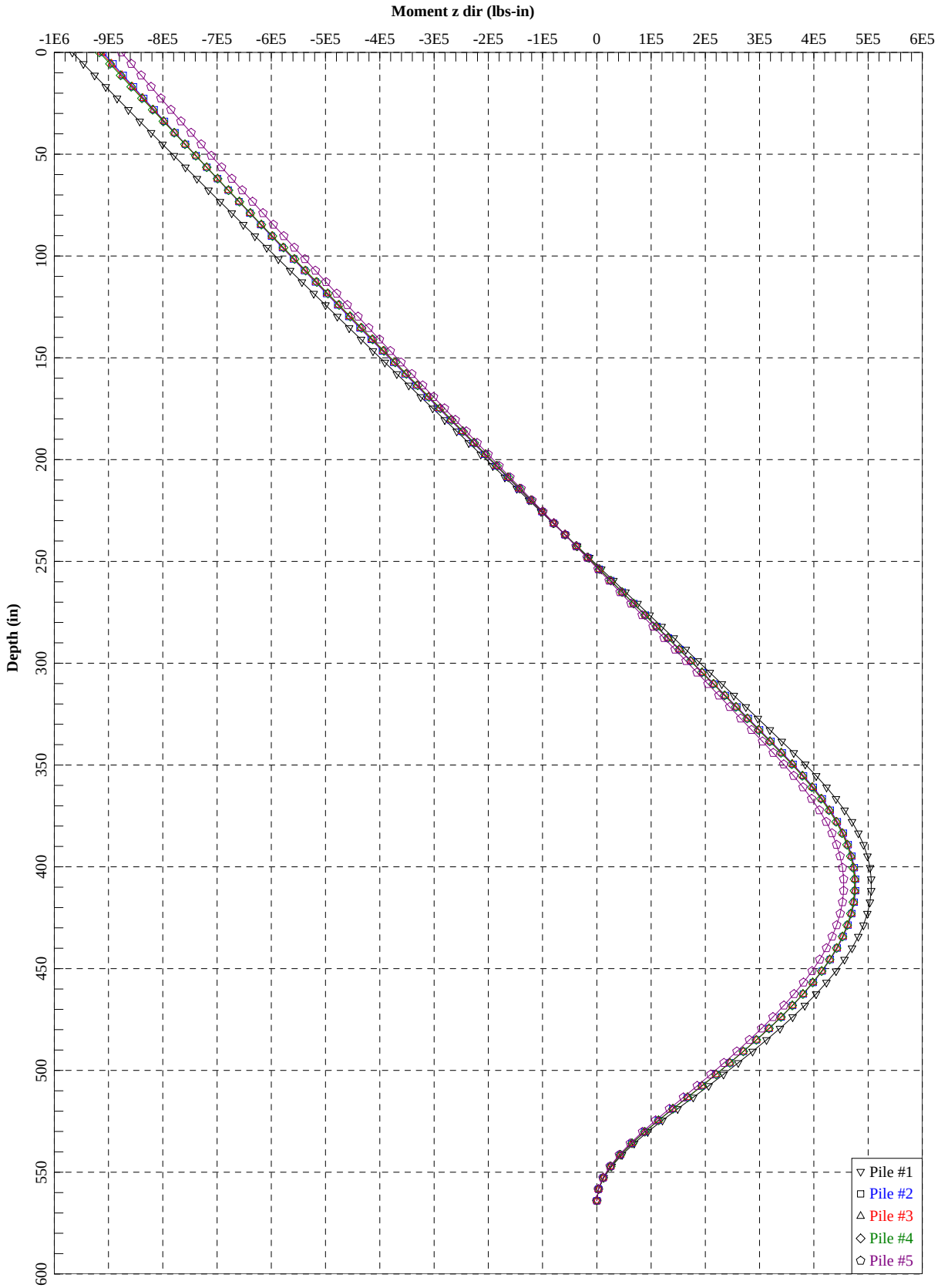
Extreme II Load Case



Service I Load Case



Service I Load Case



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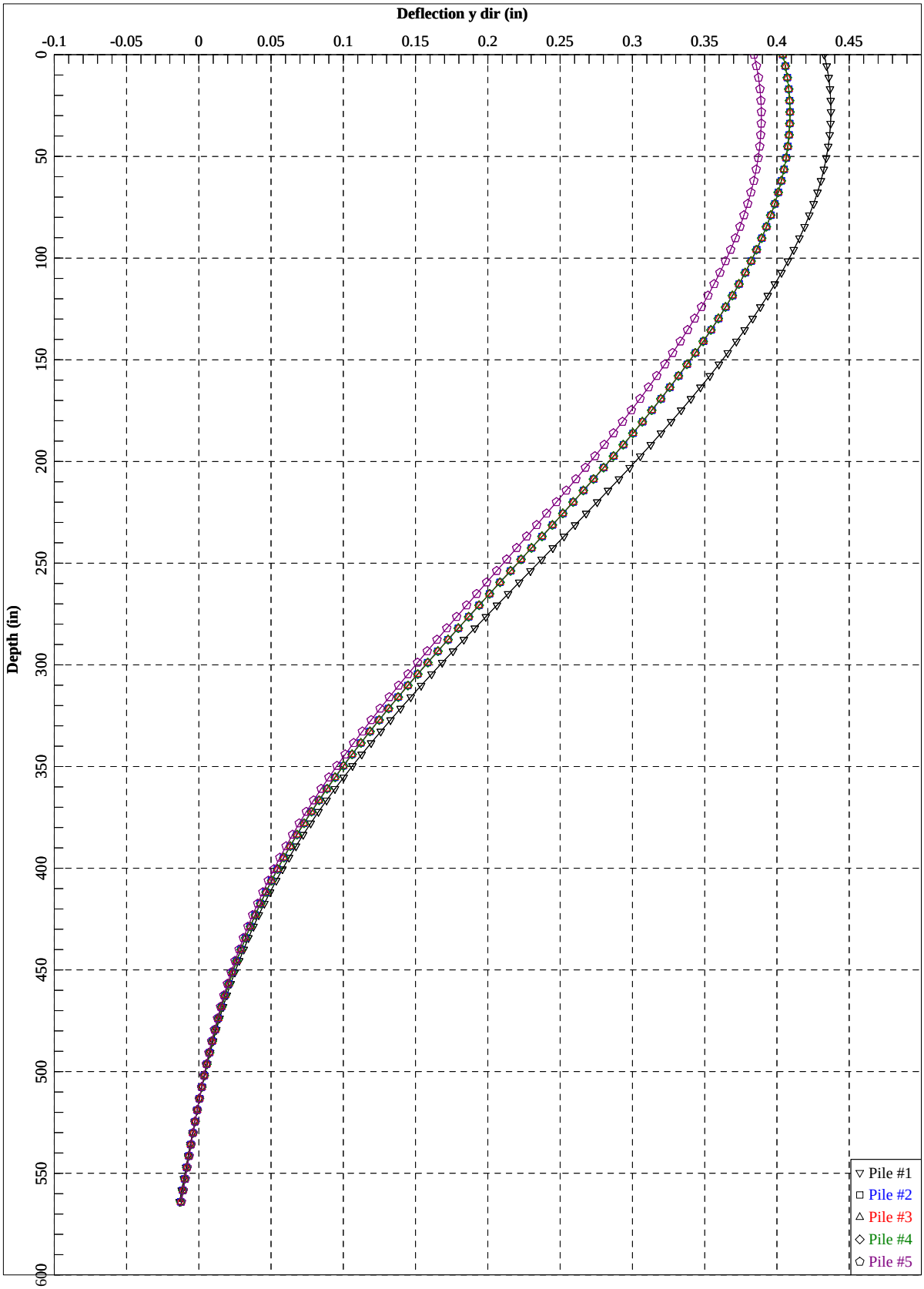

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Engineers and Scientists

JOB: 09.0025781.00 Wadsworth St.
SUBJECT: Site Class for Abutments
SHEET: 1 OF 9
CALCULATED BY ARB 8/20/14
CHECKED BY

Objective

Determine site class and seismic design parameters for proposed bridge.

Methodology

Shear wave velocity was used to determine the site class in accordance with the AASHTO 2012 LRFD Bridge Design Specification, 6th Ed, Tables 3.10.3.1-1 and C3.10.3.1-1.

The soil profile includes a mixture of sand and silt soils. Therefore, the N-bar method (Method B) is strongly influenced by lower blows in the silt that are not representative of the true shear wave velocity (Vs). Therefore, Method A, based on shear wave velocity, is used to determine site class. Empirical relationships based on correlations with SPT N-values were used to calculate Vs.

Only the soil profile was included to analyze the site class, which varies from approximately 30 to 55 feet deep at the abutments; the presence of bedrock in the upper 100 feet was conservatively neglected. Four borings were drilled for the two abutments (BB-TSGR-101, -201, -103 and -205). However, sampling was limited in borings BB-TSGR-201 and 205, making it difficult to identify the thickness of soil associated with available blow count data. Therefore, only data from borings BB-TSGR-101 and -103 was considered in the evaluation.

Empirical relationships used to calculate Vs are summarized below:

Seed et al., 1986 (sands and silts):

$$G = 1{,}000\ K_2(\sigma' m)^{1/2}$$

$$K2 = 20(N1)_{60}^{1/3}$$

$$Vs = (G/\rho)^{0.5}$$

Sykora, 1987 (sands, silts and clays):

$$Vs\ (fps) = 250\ N^{0.17}\ D^{0.2}$$ (sands and nonplastic silts)

$$Vs\ (fps) = 195\ N^{0.17}\ D^{0.2}$$ (clays)

Results

Calculated shear wave velocities for the two empirical methods considered range from 618 to 651 ft/sec (see Sheets 2-3). Therefore, the site is classified as Site Class D , with average Vs between 600 and 1,200 ft/sec. Design seismic parameters for the site were generated using the online USGS Design Maps Tool, Version 3.10 (see Sheets 4-9) and are presented below:

Fpga=Fa=1.6 Fv=1.4 As=0.10 g Sds=0.21 g Sd1=0.10g

References

1. Seed, H.B., Wong, R.T., Idriss, I.M., and Tokimatsu, K. (1986), "Moduli and Damping for Dynamic Analyses of Cohesionless Soils," ASCE, Journal of the Geotechnical Engineering, 112(11).

2. Sykora, D.W. (1987), "Examination of Existing Shear Wave Velocity and Shear Modulus Correlations in Soils," Misc, Paper GL-87-22, U.S. Army Corps of Engineers WES, Vicksburg, MS, 1987.

EMPIRICAL SHEAR WAVE VELOCITY CALCULATION
Boring BB-TSGR-101

Existing Ground Surface Elevation (ft NAVD 88)	14.6
Finish Ground Surface Elevation (ft NAVD 88)	19
New Fill Thickness (ft)	4.4
SPT Energy Transfer Ratio	83
Groundwater Depth (ft)	19

Sample ID	Existing Midpoint Depth	Final Midpoint Depth	Midpoint Elev.	Soil Type	Split Spoon Sampler Blows per 6"				N	N_{60}	Total Unit Weight	Total Stress	Water Pressure	Effective Stress	Ko	Mean Effective Stress	$(N_1)_{60}$	Layer Thickness, di	Vs1, Seed et al 1986 (SPT)	di/Vs1	Sykora 87 Empirical Constant	Vs2, Sykora 87 (SPT)	di/Vs2
	(ft)	(ft)	(ft)		0-6"	6-12"	12-18"	18-24"	(bpf)	(bpf)	(pcf)	(psf)	(psf)	(ksf)		(ksf)	(bpf)	(ft)	(fps)	(ft/fps)		(fps)	(ft/fps)
NF		2.2	16.8	SAND					30	30	125	275	0	0.28	0.40	0.17	83	4.4	537	0.008	250	522	0.008
1D	3	7.4	11.6	SAND	11	13	11	8	24	33	120	899	0	0.90	0.40	0.54	51	4.5	679	0.007	250	677	0.007
2D	6	10.4	8.6	SAND	4	3	3	4	6	8	120	1259	0	1.26	0.40	0.76	11	5.5	570	0.010	250	572	0.010
3D	11	15.4	3.6	SILT	7	3	2	5	5	7	120	1859	0	1.86	0.40	1.12	7	5.0	591	0.008	195	468	0.011
4D	16	20.4	-1.4	SILT	5	5	4	3	9	12	125	2484	0	2.48	0.40	1.49	11	3.0	670	0.004	195	547	0.005
5D	21	25.4	-6.4	SAND	3	4	13	14	17	24	125	3109	125	2.98	0.40	1.79	20	6.0	768	0.008	250	817	0.007
6D	26	30.4	-11.4	SAND	5	14	20	25	34	47	125	3734	437	3.30	0.40	1.98	38	4.0	876	0.005	250	952	0.004

Total Profile Thickness (ft) 32 **Avg Vs1 (fps)** 651 **Avg Vs2 (fps)** 618

Notes:

1. Yellow cells indicate numerical input data; white cells indicate values calculated using formulas.
2. Correlated Vs1 values developed using SPT N-value data from boring Seed et al (1986) empirical relationship, using the following equation:
   $Vs=[(20{,}000*N_{1,60}^{0.333}(\sigma'_m)^{0.5})/\rho]^{0.5}$
   where $N_{1,60}$=overburden-corrected SPT value, σ'_m=mean effective stress, ρ=density, and 20,000 is an empirical constant.
3. Correlated Vs2 values developed using SPT N-value data from boring and Sykora (1987) empirical relationship, using the following equation:
   $Vs=250\,N_{60}^{0.17}\,D^{0.2}$ (sands and nonplastic silts) and $Vs=195\,N^{0.17}\,D^{0.2}$ (clays)
   where N_{60}=energy-corrected SPT value, σ'_m=mean effective stress, ρ=density, and 20,000 is an empirical constant.

EMPIRICAL SHEAR WAVE VELOCITY CALCULATION
Boring BB-TSGR-103

Existing Ground Surface Elevation (ft NAVD 88)	14.6
Finish Ground Surface Elevation (ft NAVD 88)	15
New Fill Thickness (ft)	0.4
SPT Energy Transfer Ratio	83
Groundwater Depth (ft)	14.6

Sample ID	Existing Midpoint Depth	Final Midpoint Depth	Midpoint Elev.	Soil Type	Split Spoon Sampler Blows per 6"				N	N_{60}	Total Unit Weight	Total Stress	Water Pressure	Effective Stress	Ko	Mean Effective Stress	$(N_1)_{60}$	Layer Thickness, di	Vs, Seed et al 1986 (SPT)	di/Vs1	Sykora 87 Empirical Constant	Vs, Sykora 87 (SPT)	di/Vs2
	(ft)	(ft)	(ft)		0-6"	6-12"	12-18"	18-24"	(bpf)	(bpf)	(pcf)	(psf)	(psf)	(ksf)		(ksf)	(bpf)	(ft)	(fps)	(ft/fps)		(fps)	(ft/fps)
NF		0.2	14.8	SAND					30	30	120	24	0	0.02	0.40	0.01	282	0.4	365	0.001	250	323	0.001
1D	3	3.4	11.6	SAND	5	6	12	15	18	25	120	408	0	0.41	0.40	0.24	57	5.0	568	0.009	250	552	0.009
2D	6	6.4	8.6	SILT	7	5	5	5	10	14	120	768	0	0.77	0.40	0.46	23	5.0	572	0.009	195	442	0.011
3D	11	11.4	3.6	SAND	4	4	4	4	8	11	120	1368	0	1.37	0.40	0.82	14	5.0	607	0.008	250	612	0.008
4D	16	16.4	-1.4	SAND	3	5	8	5	13	18	120	1968	87	1.88	0.40	1.13	19	5.0	694	0.007	250	715	0.007
5D	21	21.4	-6.4	SAND	2	1	2	1	3	4	120	2568	399	2.17	0.40	1.30	4	5.0	556	0.009	250	588	0.009
6D	26	26.4	-11.4	SAND	2	1	1	1	2	3	120	3168	711	2.46	0.40	1.47	3	5.0	531	0.009	250	572	0.009
7D	31	31.4	-16.4	SILT	0	0	1	1	1	1	115	3743	1023	2.72	0.40	1.63	1	5.0	492	0.010	195	411	0.012
8D	36	36.4	-21.4	SAND	2	2	2	2	4	6	120	4343	1335	3.01	0.40	1.80	5	5.0	617	0.008	250	686	0.007
9D	41	41.4	-26.4	SAND	0	3	6	5	9	12	120	4943	1647	3.30	0.40	1.98	10	5.0	716	0.007	250	808	0.006
10D	46	46.4	-31.4	SILT	21	33	31	34	64	89	125	5568	1959	3.61	0.40	2.17	68	5.0	988	0.005	195	900	0.006
11D	50	50.4	-35.4	SAND	50	100	100		200	277	125	6068	2209	3.86	0.40	2.32	205	5.0	1208	0.004	250	1424	0.004

Total Profile Thickness (ft)	55	Avg Vs1 (fps)	637	Avg Vs2 (fps)	624

Notes:

1. Yellow cells indicate numerical input data; white cells indicate values calculated using formulas.
2. Correlated Vs1 values developed using SPT N-value data from boring Seed et al (1986) empirical relationship, using the following equation:
   $Vs1=[(20{,}000*N_{1,60}^{0.333}(\sigma'_m)^{0.5})/\rho]^{0.5}$
   where $N_{1,60}$=overburden-corrected SPT value, σ'_m=mean effective stress, ρ=density, and 20,000 is an empirical constant.
3. Correlated Vs2 values developed using SPT N-value data from boring and Sykora (1987) empirical relationship, using the following equation:
   $Vs=250\ N_{60}^{0.17}\ D^{0.2}$ (sands and nonplastic silts) and $Vs=195\ N^{0.17}\ D^{0.2}$ (clays)
   where N_{60}=energy-corrected SPT value, σ'_m=mean effective stress, ρ=density, and 20,000 is an empirical constant.

Seismic Design, Thomaston
Sheet 3 of 9

USGS Design Maps Detailed Report

2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design (44.072°N, 69.188°W)

Site Class D – "Stiff Soil"

Article 3.4.1 — Design Spectra Based on General Procedure

Note: Maps in the 2009 AASHTO Specifications are provided by AASHTO for Site Class B. Adjustments for other Site Classes are made, as needed, in Article 3.4.2.3.

From Figure 3.4.1-2 [1]	PGA = 0.059 g
From Figure 3.4.1-3 [2]	S_S = 0.131 g
From Figure 3.4.1-4 [3]	S_1 = 0.040 g

Article 3.4.2.1 — Site Class Definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Article 3.4.2.

Table 3.4.2.1–1 Site Class Definitions

SITE CLASS	SOIL PROFILE NAME	Soil shear wave velocity, $\bar{v}_s$, (ft/s)	Standard penetration resistance, $\bar{N}$	Soil undrained shear strength, $\bar{s}_u$, (psf)
A	Hard rock	$\bar{v}_s > 5{,}000$	N/A	N/A
B	Rock	$2{,}500 < \bar{v}_s \leq 5{,}000$	N/A	N/A
C	Very dense soil and soft rock	$1{,}200 < \bar{v}_s \leq 2{,}500$	$\bar{N} > 50$	>2,000 psf
D	Stiff soil profile	$600 \leq \bar{v}_s < 1{,}200$	$15 \leq \bar{N} \leq 50$	1,000 to 2,000 psf
E	Stiff soil profile	$\bar{v}_s < 600$	$\bar{N} < 15$	<1,000 psf
E	—	Any profile with more than 10 ft of soil having the characteristics: 1. Plasticity index $PI > 20$, 2. Moisture content $w \geq 40\%$, and 3. Undrained shear strength $\bar{s}_u < 500$ psf		
F	—	Any profile containing soils having one or more of the following characteristics: 1. Soils vulnerable to potential failure or collapse under seismic loading such as liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils. 2. Peats and/or highly organic clays ($H > 10$ feet of peat and/or highly organic clay where H = thickness of soil) 3. Very high plasticity clays ($H > 25$ feet with plasticity index $PI > 75$) 4. Very thick soft/medium stiff clays ($H > 120$ feet)		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Article 3.4.2.3 — Site Coefficients

Table 3.4.2.3-1 (for F_{pga})—Values of F_{pga} as a Function of Site Class and Mapped Peak Ground Acceleration Coefficient

Site Class	Mapped Peak Ground Acceleration				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See AASHTO Article 3.4.3				

Note: Use straight-line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.059 g, F_{PGA} = 1.600

Table 3.4.2.3-1 (for F_a)—Values of F_a as a Function of Site Class and Mapped Short-Period Spectral Acceleration Coefficient

Site Class	Spectral Response Acceleration Parameter at Short Periods				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See AASHTO Article 3.4.3				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = D and S_S = 0.131 g, F_a = 1.600

Table 3.4.2.3-2—Values of F_v as a Function of Site Class and Mapped 1-sec Period Spectral Acceleration Coefficient

Site Class	Mapped Spectral Response Acceleration Coefficient at 1-sec Periods				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See AASHTO Article 3.4.3				

Note: Use straight-line interpolation for intermediate values of S_1

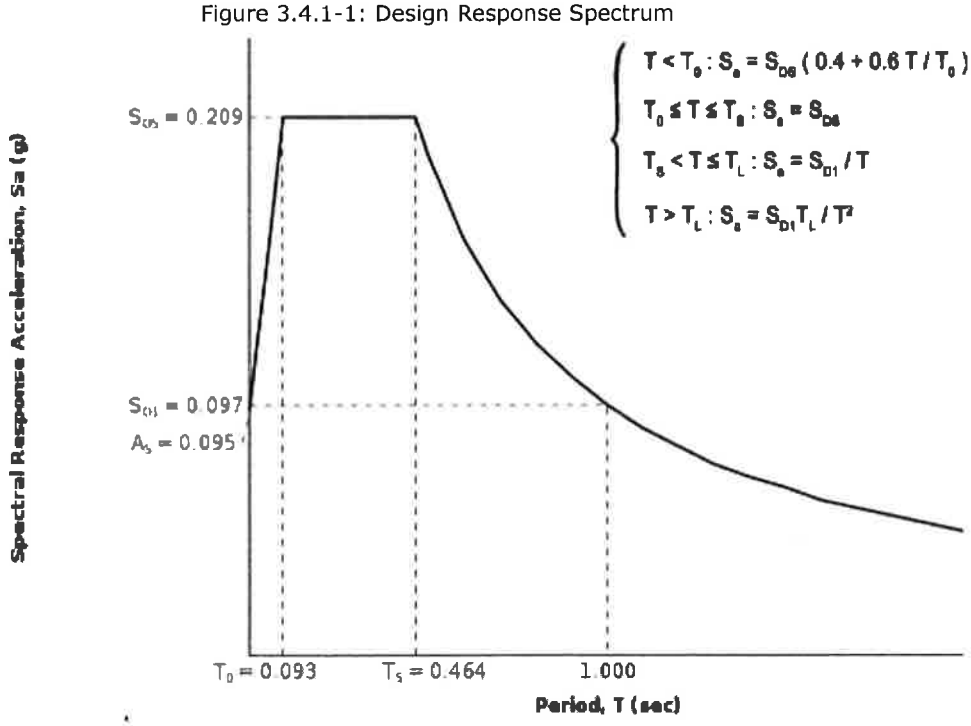
For Site Class = D and S_1 = 0.040 g, F_v = 2.400

Equation (3.4.1-1): $A_S = F_{PGA}\ PGA = 1.600 \times 0.059 = 0.095$ g

Equation (3.4.1-2): $S_{DS} = F_a\ S_S = 1.600 \times 0.131 = 0.209$ g

Equation (3.4.1-3): $S_{D1} = F_v\ S_1 = 2.400 \times 0.040 = 0.097$ g

Figure 3.4.1-1: Design Response Spectrum



Article 3.5 - Selection of Seismic Design Category (SDC)

Table 3.5-1—Partitions for Seismic Design Categories A, B, C, and D

VALUE OF S_{D1}	SDC
$S_{D1} < 0.15g$	A
$0.15g \leq S_{D1} < 0.30g$	B
$0.30g \leq S_{D1} < 0.50g$	C
$0.50g \leq S_{D1}$	D

For S_{D1} = 0.097 g, Seismic Design Category = A

Seismic Design Category ≡ "the design category in accordance with Table 3.5-1" = A

References

1. *Figure 3.4.1-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-2.pdf
2. *Figure 3.4.1-3*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-3.pdf
3. *Figure 3.4.1-4*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/AASHTO-2009-Figure-3.4.1-4.pdf