SUPPLEMENTAL GEOTECHNICAL DESIGN REPORT
ROUTE 1 VIADUCT BRIDGE REHABILITATION
MAINEDOT WIN 19273.00
BATH, MAINE

BY

Haley & Aldrich, Inc.
Portland, Maine

FOR

Maine Department of Transportation
Augusta, Maine

File No. 40725-000
5 September 2014

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5 September 2014
File No. 40725-000

Maine Department of Transportation
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Attention: Laura Krusinski, P.E.
Senior Geotechnical Engineer

Subject: Supplemental Geotechnical Design Report
Route 1 Viaduct Bridge Rehabilitation
MaineDOT WIN 19273.00
Bath, Maine

Ladies and Gentlemen:

We are pleased to submit herewith our report entitled, "Supplemental Geotechnical Design Report, U.S. Route 1 Viaduct Bridge Rehabilitation, MaineDOT WIN 19273.00, Bath, Maine," prepared in accordance with our proposal, dated 10 March 2014 and executed by your Wayne Frankhauser, Jr. on 13 March 2014 and with the provisions of our General Consultant Agreement (GCA) with the Maine Department of Transportation (MaineDOT), No. CT20110614000000006492.

This supplemental geotechnical design report (SGDR) has been prepared in support of FIGG Bridge Engineers, Inc. (FIGG) submission of a preliminary design report (PDR) to MaineDOT. This SGDR presents the results of limited subsurface investigation and non-destructive testing programs and also provides preliminary geotechnical design recommendations for the subject project.

Thank you for the opportunity to help support MaineDOT on this project. We look forward to providing continued assistance to you during subsequent phases of the project. Please contact us if you have any questions or comments on the information provided herein.

Sincerely yours,
HALEY & ALDRICH, INC.

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Enclosures
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1. INTRODUCTION

This Supplemental Geotechnical Design Report (SGDR) presents the results of supplemental phase subsurface and non-destructive testing (NDT) programs, engineering evaluations, and preliminary geotechnical design recommendations conducted and developed by Haley & Aldrich, Inc. (Haley & Aldrich) on behalf of the Maine Department of Transportation (MaineDOT) for the proposed rehabilitation of the Route 1 Viaduct Bridge (viaduct bridge) in Bath, Maine (see Figure 1, Project Locus). We coordinated our technical evaluations with FIGG Bridge Engineers, Inc. (FIGG), the bridge designer, throughout the preliminary phase of the project in support of their development and submission of a Preliminary Design Report (PDR) to MaineDOT.

Please recall that we previously completed subsurface investigation and laboratory testing programs and completed feasibility level evaluations, which were summarized in our Preliminary Geotechnical Design Report (PGDR) dated 17 September 2013. The PGDR was developed in support of FIGG's preparation and submission of a Feasibility Study to MaineDOT.

1.1 Horizontal Coordinate System and Elevation Datum

Plan locations of test borings are reported as northing and easting coordinates relative to the Maine State Plane Coordinate System, North American Datum of 1983 (NAD 83), Maine 2000 West Zone. The project elevation datum and elevations referenced herein are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

Please note that the historic viaduct bridge information, including drawings, reports and other documents reference an assumed vertical datum. The relationship between the assumed vertical datum and the NAVD 88 datum is as follows:

$$\text{NAVD 88 Datum} = \text{Assumed Vertical Datum} - 48.18 \text{ feet}$$

The vertical datum relationship shown above was determined based on our review of original viaduct bridge construction documentation provided by MaineDOT (see Appendix E).

1.2 Existing Bridge Structure

The existing viaduct bridge carries U.S. Route 1 traffic and serves as the west approach to the Sagadahoc Bridge over the Kennebec River in Bath, Maine (see Figure 1, Project Locus).

Based on our review of historic viaduct bridge drawings (drawings) provided by MaineDOT, it is our understanding that the structure was originally constructed in 1957 and consists of an approximate 1,300-ft long, 34-ft wide (two travel lanes, one in each direction), 20-span structure supported on two abutments and 19 piers. Span lengths vary between approximately 60 and 76 ft. The foundation support type for the abutments and piers is variable and consists of one of the following:

- Cast-in-place (CIP) concrete spread footings bearing on bedrock (i.e., Abutment 1, Piers 1, 2, 4, 8, 9 and 13)
- CIP concrete spread footings constructed on seal concrete, bearing on bedrock (i.e., Piers 3 and 14)
- HP10x42 steel H-piles (i.e., Piers 5-7, 10-12, 15-19 and Abutment 2)

Based on our review of the drawings it is unclear what allowable bearing pressure was used to design the footings that support the existing abutments/piers. However, the drawings do indicate that the maximum pressure at the corner of the spread footings is 13.7 kips per square foot (ksf). In addition, HP10x42 steel H-piles supporting the abutments/piers were designed based on "allowable loads" equal to 67.6 kips (Abutment 2) and 92.6 kips, respectively. As-built pile lengths varied significantly within individual pile groups and along the viaduct bridge alignment as discussed in subsequent sections of this report. Select piles supporting the viaduct bridge piers and Abutment 2 are battered at 4V:1H and 5V:1H, respectively. Historic bridge drawings are provided for reference in Appendix E.

1.3 Proposed Bridge Rehabilitation

Based on our discussions with MaineDOT and FIGG, we understand that the viaduct bridge has been repaired/rehabilitated many times since its original construction in the late 1950's. The most recent rehabilitation occurred in 2006, which consisted of repairs to the concrete deck. In general, the structure has deteriorated over time due to continued and on-going leakage of expansion joints on diaphragms, bearings and beam seats.

As a result and based on information provided by FIGG, several repair and replacement alternatives for the deteriorating viaduct bridge deck, superstructure and substructure were considered during the feasibility study stage of the project, which met the following criteria:

- Rehabilitate and/or replace the superstructure, pier columns/caps and/or substructures as needed
- Minimize the use and number of expansion joints
- 50-year design life for rehabilitated and replacement components.
- Economical construction cost
- Perform demolition and construction from above
- Limit viaduct bridge closure to the period between November and April and to a 4-month maximum duration

As summarized in FIGG's Feasibility Study Report, it was determined that replacement of the superstructure and completing repairs or rehabilitating the pier caps/columns and substructures, if necessary, was the preferred alternative, which met the criteria outlined above.

The purpose of the preliminary design phase is to advance the design of the preferred alternative selected during the feasibility stage and to provide more specific recommendations which will be advanced during final design. The geotechnical information summarized herein was used during the preliminary design phase of the subject project.

2. SUBSURFACE EXPLORATION PROGRAMS

2.1 Historic Explorations

A preliminary phase subsurface investigation, consisting of rod soundings, was completed in late 1956 and early 1957 as part of the design and construction of the existing viaduct bridge. Rod soundings were completed at the following existing substructure locations: Pier Nos. 1, 3, 5, 9, 11, 16, 17 and 19. Upon completion of the preliminary subsurface investigation (i.e., the rod soundings), a supplemental investigation was conducted, which consisted of a minimum of 23 test borings, designated 1 through 23. The test borings were drilled to confirm findings from the preliminary subsurface investigation and to collect samples of compressible soils. Test borings were drilled at the following substructure locations: Abutment Nos. 1 and 2 and Pier Nos. 2, 4, 6 through 8, 10, 12 through 15 and 18.

The plan locations of the historic explorations (rod soundings and test borings) are provided on historic drawings included in Appendix E. Additionally, test boring logs documenting the subsurface conditions encountered in previously completed explorations are provided in Appendix A. Additional details of the subsurface exploration programs are provided in Reference No. 5.

2.2 Preliminary Phase Explorations

In addition, Haley & Aldrich completed a limited subsurface investigation in March 2013 as part of the feasibility phase of the current viaduct bridge rehabilitation project. A total of three test borings, designated BB-BROV-101 through BB-BROV-103, were drilled near Pier Nos. 6, 11 and between 18 and 19 to provide general subsurface information at specific substructure locations that were identified as critical locations relative to viaduct bridge rehabilitation/replacement alternatives that were being considered by FIGG and MaineDOT.

Additional details of the subsurface exploration programs are provided in Reference No. 5. The plan locations of the preliminary phase explorations (test borings) are shown on Figure 2. Test boring logs documenting the subsurface conditions encountered in previously completed explorations are provided in Appendix A.

2.3 Supplemental Phase Explorations

Haley & Aldrich completed a supplemental phase subsurface exploration program, consisting of test borings, test pits and non-destructive testing (NDT), at the site in March and April 2014. The purpose of the exploration program was to collect additional information for use in evaluating the reuse potential of existing pile-supported substructures.

Considering the need to collect sufficient information for use in evaluating the reuse potential of existing pile-supported substructures, locations were identified where test borings could be drilled and test pits could be excavated while minimizing impacts to pedestrian and vehicular traffic. As a result, six substructure locations were identified where subsurface explorations could be completed: Pier Nos. 5, 7, 10, 12, 19 and Abutment 2. Additional details of each part of the supplemental phase subsurface exploration program are provided in the following sections of this report.

2.3.1 Test Borings

A total of six test borings, designated BB-BROV-201 through BB-BROV-206, were drilled adjacent to Pier Nos. 5, 7, 10, 12, 19 and Abutment 2. The test boring locations were laid out in the field by Haley & Aldrich by taping/pacing distances from existing site features while considering potential drill rig access limitations prior the start of drilling. "As-drilled" test boring locations and ground surface elevations were determined in the field by MaineDOT using optical survey equipment upon the completion of drilling and were provided to Haley & Aldrich. The "as-drilled" coordinates of each test boring were related to station and offset distance and direction relative to the existing baseline by Haley & Aldrich as shown on Table I and the test borings logs included in Appendix A). The plan locations of the supplemental explorations are shown on Figure 2.

The test borings were drilled by MaineDOT and monitored by a Haley & Aldrich geologist between 25 March and 8 April 2014 using a truck-mounted CME 45C drill rig. Test borings were advanced to depths ranging from approximately 29 to 62 ft below existing ground surface (BGS) using a combination of 3 in. (NW-size) and 4 in. (HW-size) outside diameter (OD) steel casing.

Soil samples were generally collected continuously and/or at standard, 5-ft intervals, by driving a 1-3/8-in. ID split-spoon sampler with a 140-lb hammer dropped from a height of 30 in., as indicated on the test boring logs. The number of hammer blows required to advance the sampler through each 6 in. interval was recorded and is provided on the logs. The uncorrected SPT N-value (N-uncorrected) is defined as the total number of blows required to advance the sampler through the middle 12 in. of the 24-in. sampling interval. The drill rig was equipped with a calibrated automatic hammer per MaineDOT requirements. The energy-corrected SPT N-value (N_{60}) is equal to the uncorrected N-value multiplied by the hammer efficiency factor divided by 0.867 (i.e., 86.7 percent theoretical hammer efficiency).

Test borings were advanced approximately 15 to 18 ft into bedrock using a 2.0-in. (NQ-size) ID diamond-tipped core barrel.

All soil and bedrock samples are available for review upon request. The available soil and bedrock samples are currently being stored at the Haley & Aldrich laboratory facility in Portland, Maine.

Upon test boring completion, 2 in. OD solid PVC riser pipe was installed from the bottom of the borehole up to the existing ground surface. The annulus between the PVC and the borehole was backfilled with filter sand. The PVC was filled with fresh (non-saline) water and was outfitted with a flush-mounted steel guard roadway box assembly. The PVC was installed to facilitate NDT, as discussed in subsequent sections of this report.

2.3.2 Test Pits

Shallow test pits were excavated at Pier Nos. 5, 7, 10, 12 and 19 and Abutment 2 upon completion of the test borings by Shaw Brothers Construction, Inc. of Gorham, Maine. The primary purposed of the shallow test pits was to expose a portion of the existing pile caps in the immediate vicinity of the previously completed test borings (and PVC casing) so that the substructure could be impacted as part of the non-destructive testing. Each shallow test pit was excavated approximately 3 to 4 ft BGS using a Takeuchi TB 175 rubber-tracked mini-excavator.

Upon completion of the NDT deep test pits were excavated at Pier Nos. 5, 7, 10 and 19 by Reed & Reed, Inc. of Woolwich, Maine. During the course of the deep test pit excavation work it was determined by Haley & Aldrich that a deep test pit could not be excavated at Pier 12 due to impacts on vehicular traffic. In addition, the deep test pit excavation at Abutment 2 was abandoned prior to completion due to the nature of excavated material and volume of water entering the excavation, which in combination posed health and safety issues for personnel entering the excavation, as judged by Haley & Aldrich.

The primary purpose of the deep test pit excavations was to provide an opportunity to perform a visual inspection of one or more existing steel H-piles for signs of corrosion as it related to evaluating their reuse potential. In general, the test pits were excavated to approximately 2 ft below the bottom of the piles caps (8 to 9 ft BGS). Each test pit was temporarily braced using a trench box, dewatered using open sumps and backfilled using excavated material upon completion.

Documentation of the deep test pit excavations is provided in Appendix C and findings are summarized in subsequent sections of this report.

2.3.3 Non-Destructive Testing

As a result of significant differences that existed between pile tip elevations estimated based on pile lengths provided on the historic drawings and the top of rock elevations encountered in historic and preliminary and supplemental phase test borings it was necessary to complete a limited amount of non-destructive testing to better understand actual pile tip elevations, which was necessary for estimating static pile resistance and evaluating reuse potential.

Upon completion of the test borings and shallow test pits, NDT was completed by Olson Engineering, Inc. (Olson) of Wheat Ridge, Colorado at Pier Nos. 5, 7, 10, 12 and 19 and Abutment 2. In general, the parallel seismic (PS) was used to estimate existing pile lengths and pile tip elevations in the immediate vicinity of the previously completed test borings. The PS method involves physically impacting any part of the substructure (concrete pile cap or individual pile) and measuring compression and/or shear waves travelling down the pile with a hydrophone or geophone receiver placed inside of PVC casing installed in boreholes drilled immediately adjacent to the substructures.

Additional testing details and results of the NDT are provided in Appendix D, are discussed in subsequent sections of this report and are also summarized in Table III.

3. GENERALIZED SUBSURFACE CONDITIONS

The subsurface conditions encountered at the site were somewhat variable along the length of the proposed viaduct bridge and generally consist of the following geologic units presented in order of increasing depth below ground surface: man-placed fill, organic deposits, marine deposits, glacial till and bedrock. Refer to Table II for a summary of the soil units and encountered thicknesses at each test boring (preliminary and supplemental) location. A general description of each soil/bedrock unit is provided separately, below. The discussion below is limited to the conditions encountered in the supplemental phase test borings only. Detailed soil and bedrock descriptions are provided on the test boring logs included Appendix A. Preliminary phase and historic test borings are also provided for reference in Appendix A.

Please note that soil descriptions provided on the test boring logs and summarized below do not represent actual field conditions other than at the specific test boring locations. The actual conditions will likely vary from those described herein.

3.1 Soil Unit and Bedrock Conditions

3.1.1 Bituminous Concrete/Man-Placed Fill

A surficial layer of bituminous concrete was encountered at the ground surface in each test boring with the exception of BB-BROV-206. The thickness of the bituminous concrete was approximately 1 in. A 1-ft thick layer of topsoil was encountered at the ground surface in BB-BROV-206.

An approximate 5 to 14-ft thick layer of man-placed fill was encountered in each test boring directly beneath the surficial layer of bituminous concrete or topsoil. In general, the man-placed fill soil consisted of the coarse to fine SAND with varying amounts of silt and gravel. The lower portions of the layer at some test boring locations consisted of reworked CLAY with sand and asphalt or ORGANIC SOIL with sand, wood and rubber fragments.

Granular portions of the stratum were very loose to medium dense. Cohesive portions of the deposit were very soft to stiff.

3.1.2 Organic Deposit

An approximate 3.5 to 4.5-ft thick layer of organic soil was encountered beneath the man-placed fill in test borings BB-BROV-205 and BB-BROV-206. The unit consisted of ORGANIC SOIL with varying amounts of fine sand and woody/fibrous organic matter and was very soft to soft. A strong petroleum-like odor was present within the deposit at test boring BB-BROV-206.

3.1.3 Marine Sand/Silt/Clay Deposit

An approximate 7-ft thick layer of medium dense marine sand consisting of coarse to fine SAND with silt and gravel, was encountered beneath the man-placed fill in test boring BB-BROV-201.

Marine silt/clay was encountered directly beneath the man-placed fill or organic soil in each test boring with the exception of BB-BROV-201. The deposit ranged in thickness from approximately 11 ft to 37 ft and generally increased from west to east, towards the Kennebec River. The upper portions of the deposit typically consisted of stiff to very stiff silty CLAY, which transitioned into medium stiff silty CLAY with increasing depth. In-situ vane shear test results within the lower portion of the deposit ranged from approximately 500 to 850 pounds per square foot (psf) and typically decreased with depth to approximately El. -15. Below El. -15 the undrained shear strength is generally uniform with depth and equal to approximately 500 psf.

3.1.4 Glacial Till Deposit

A limited thickness of glacial till was encountered in test borings BB-BROV-202 and BB-BROV-205 directly beneath the marine silt/clay layer and overlying bedrock. The deposit ranged in thickness from approximately 1 to 2 ft and consisted of clayey, fine to medium SAND (SC/SM) with some silt and trace gravel.

Due to the limited thickness of glacial till encountered and pre-determined sampling intervals, limited SPT data was obtained. Therefore, an accurate assessment of the density/consistency of the soil layer could not be determined.

3.1.5 Bedrock

As described above, bedrock was encountered and sampled in each test boring. The top of bedrock surface varied along the length of the viaduct bridge from approximately 14 ft (BB-BROV-201) to 47 ft (BB-BROV-205) BGS, corresponding to El. 15 to El. -36. As shown on the historic drawings, the top of bedrock surface is highly variable and is considered erratic along the length of the viaduct bridge. The type of bedrock encountered generally consisted of:

- Moderately hard to very hard, fresh to slightly weathered SCHIST of the Cape Elizabeth Formation. Primary joints were oriented at low to moderate angles, very close to moderately spaced and were generally tight to open. Joint surfaces were occasionally moderately weathered and silt coated.

Rock quality designation (RQD) is a common parameter that is used to help assess the competency of sampled bedrock. RQD is defined as the sum of pieces of recovered bedrock greater than 4 in. in length divided by the total length of recovered bedrock. RQD values for bedrock encountered at the site ranged from 57 to 97 percent.

Photographs of the sampled bedrock are provided for reference in Appendix B.

3.2 Groundwater Conditions

Water levels were measured in the recently completed boreholes at depths ranging from approximately 2 to 7 ft BGS, prior to backfilling, as indicated on the test boring logs. In general, shallower water levels were observed in test borings drilled closer to the Kennebec River (BB-BROV-205 and BB-BROV-206). Please note that observation wells were not installed in any of the recently completed boreholes. The water depths measured in the test borings may not be representative of actual groundwater levels and could have been influenced by drilling means/methods. In general, groundwater levels can be expected to fluctuate, subject to seasonal variation, local soil conditions, tide, topography and precipitation. Water levels encountered during construction may differ from those summarized above.

4. SUPPLEMENTAL GEOTECHNICAL EVALUATIONS

Supplemental geotechnical design recommendations for the subject project, as discussed and provided herein, were developed in accordance with the following documents:

- AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications, Sixth Edition, 2012 with Errata dated June 2012.
- MaineDOT Bridge Design Guide (BDG), August 2003 with Interim Revisions through August 2008.

Furthermore, the supplemental design recommendations are based, in part, on the subsurface conditions encountered in the test boring referenced herein. It should be noted that test borings were drilled at discrete locations adjacent to existing substructure locations and variations in subsurface conditions are likely to exist over short distances (within the plan limits of a substructure unit). Our evaluations are based on the assumption that subsurface conditions are somewhat similar to those encountered in the referenced test borings.

Please note that the supplemental geotechnical evaluations and preliminary design recommendations included herein relate to the existing viaduct bridge pier foundations only. Based on our discussions with FIGG we understand that the existing viaduct bridge abutments will be evaluated during final design.

4.1 Preliminary Evaluation of Existing Steel H-piles

As discussed in previous sections of this report, Pier Nos. 5 through 7, 10 through 12, 15 through 19 and Abutment 2 are supported on HP10x42 steel H-piles. Evaluation of their reuse potential in supporting a rehabilitated/replacement superstructure is presented below.

4.1.1 Pile Length Estimates and Bearing Stratum

4.1.1.1 Pile Lengths Estimated from NDT

An understanding of existing pile lengths at each substructure location is necessary to make judgments regarding pile bearing strata and ultimately static pile resistance. During the feasibility phase of the project estimated pile tip elevations were calculated based on the top of pile elevations and estimated pile lengths shown on the historic drawings because as-built pile length information was not available. The calculated pile tip elevations were then compared to the subsurface conditions shown on historic and/or preliminary phase test boring logs. Given the subsurface conditions encountered in the test borings (presence of marine clay, limited thickness of glacial till and depth to bedrock) it was our opinion that the existing piles likely gain the majority of their resistance through end bearing in/on bedrock.

However, because significant differences existed between the calculated pile tip elevations and the top of rock elevations it was necessary to complete a limited amount of NDT to better understand actual pile tip elevations and for a more reliable assessment of existing pile reuse potential. As discussed previously, NDT was completed by Olson at Pier Nos. 5, 7, 10, 12 and 19 and Abutment 2 using the PS method. NDT provided a single estimate of pile length at each of the six substructure locations, which likely represents the individual pile closest to the point of impact and to the temporary PVC casing installed in boreholes drilled adjacent to the substructures with the exception of Pier 19 where two distinct pile tips were visible in the NDT data.

In general, the quality of data collected by Olson was "good" with the exception of Abutment 2 ("moderate quality") and Pier 12 ("poor quality"), which directly corresponds to the degree in confidence in the estimated pile lengths/pile tip elevations. Due to the poor quality data collected at Pier 12 a pile length(s) could not be estimated. The pile lengths estimated by Olson using NDT data are summarized in Table III and shown comparatively to as-built pile lengths/tip elevations and top of bedrock elevations. Additional testing details and results of the NDT are provided in Appendix D.

4.1.1.2 As-Built Pile Lengths

Upon completion of the NDT program MaineDOT provided records of as-built pile length information. Based on our review of the as-built pile documentation pile lengths vary considerably within individual groups (pier location) ranging from approximately 5 ft (Pier Nos. 5 and 11) to 53 ft (Pier 17). On average, pile lengths varied by up to approximately 20 ft across the 14-ft x 16-ft pile cap footprint. Assuming the existing piles were driven to practicable refusal in/on bedrock as discussed in subsequent sections of this report, the large range in as-built pile lengths within individual pile groups suggests a highly erratic bedrock surface. As-built pile lengths at each substructure location are summarized in Table III and are shown comparatively to pile lengths/tip elevations estimated from NDT results. A complete record of as-built pile length information provided by MaineDOT is included in Appendix E.

4.1.2 Original Pile Design Capacity

Existing pier pile analyses are shown on the historic drawings (sheet 36 of 148) and are summarized below.

"Allowable load = 6 ksi x 12.35in^2 x 125% = 92.6 kips"

Based on our review of the historic drawings we understand that the bridge was designed in accordance with the Maine State Highway Commission Standard Specifications, which requires pile steel to conform to the material requirements of ASTM A-7. The allowable yield stress for ASTM A-7 steel at the time the viaduct bridge was constructed is 6 kips per square inch (ksi), which is consistent with the first component of the pier pile analyses shown above. Please note that an allowable yield stress equal to 6 ksi corresponds to a factor of safety equal to 5.5 based on an ultimate yield stress of 33 ksi, which is the minimum requirement for ASTM A-7 steel at the time the viaduct bridge was constructed.

In addition, multiple historic and recent references were reviewed which indicate that the cross-sectional area of an HP10x42 steel H-pile is 12.35 in^2. This is consistent with the second component of the pier pile analyses shown above. It should be noted that the original design capacity (92.6 kips) did not account for a reduction in cross-sectional area due to corrosion.

Finally, pier pile analyses shown on the historic drawings are based on "Group III Loading". Based on our review of previous editions of the AASHTO Standard Specifications for Highway Bridges, we understand that an increase in allowable pile yield stress equal to 125 percent was allowed where "Group III Loading" controlled the design of the pile group. This is consistent with the third component of the pier pile analyses shown above.

Based on the information summarized above, it is our opinion that the pile design was based on the structural capacity of the pile assuming that the piles would be driven to practicable refusal in/on bedrock. This assumption is also consistent with statements included in Reference Nos. 1 and 2 (historic design reports).

4.1.3 Conclusions

As discussed above, pile tip elevations were estimated using NDT techniques and as-built pile data provided by MaineDOT. Based on our review of this information in comparison to the top of bedrock elevations encountered in the test borings (historic, preliminary and supplemental) some differences exist as shown in Table III. It is our opinion that differences exist primarily due to the extreme variability in the bedrock surface over short distances as evidenced by the fact that pile lengths within the Pier 17 pile group (14 ft x 16 ft plan area) vary up to approximately 53 ft. It is also our opinion that the original pile design capacity is based on the structural capacity of the pile, which is consistent with statements included in historic geotechnical reports that recommend "bearing piles driven to ledge" be used to support certain piers/abutments (see Reference Nos. 1 and 2).

Based on the information presented herein, we judge that the existing piles are end bearing in/on bedrock and that subsequent pile evaluations consider the structural resistance of the pile in accordance with AASHTO LRFD Article 6.9.4.1 (refer to Section 4.1.5 of this report).

4.1.4 Corrosion and Deterioration

The reuse potential of the existing HP10x42 steel H-piles included consideration of corrosion in accordance with AASHTO LRFD Section 10.7.5. As discussed in Section 2.3.2 of this report, a limited number of test pits were excavated so that existing piles located within the limits of the excavation (limited to 1 or 2 piles per location) could be inspected for signs of section loss due to corrosion (see Appendix C for test pit documentation).

In general, observed piles appeared in good condition with no visible signs of corrosion. Minor "chipping" of an apparent pile coating was observed on the pile exposed within the Pier 7 test pit excavation. In addition, slight pitting of the steel was observed on the pile exposed within the Pier 19 test pit excavation. However, multiple flange width measurements taken on each exposed pile indicated thicknesses equal to or greater than thicknesses representative of HP10x42 steel H-piles produced around the time the viaduct bridge was originally constructed (0.42 in.).

Based on the subsurface conditions encountered in test borings drilled immediately adjacent to and observed within the test pit excavations, the existing piles were primarily embedded within moist to wet naturally-deposited soil. In addition, observations made during excavation of the test pits at Pier 19 and Abutment 2 (abandoned) suggest that the static groundwater level in this area may be approximately equal to the bottom of pile cap elevation. It is our opinion that this type of environment has a low corrosive potential.

Although it is our opinion that the likelihood is low, it is possible that piles not observed have experienced some degree of section loss due to corrosion. Therefore, we recommend that the design axial compressive pile resistance of existing steel H-piles and other pile evaluations consider a 1/16-in. reduction in cross-sectional area over the planned 50-year design life of the rehabilitated viaduct bridge to account for corrosion. Please note that these recommendations are based on the observation of a limited number of existing piles at a select number of pier locations on the days and at the times noted.

4.1.5 Recommended Axial Compressive Pile Resistance

As stated in Section 4.1.3, it is our opinion that the existing piles likely bear in/on bedrock and that their geotechnical resistance is controlled by the structural resistance of the pile as discussed in AASHTO LRFD Section 10.7.3.2.3. Therefore, we recommend that the reuse potential of existing steel H-piles be evaluated considering a nominal compressive resistance based on the structural resistance of the pile, in accordance with AASHTO LRFD Section 6.9.4.1 and consideration of a 1/16-in. reduction in cross-sectional area over the planned 50-year design life due to corrosion. The structural resistance factor (Article 6.5.4.2) for axial resistance of piles in compression and subject to damage due to severe driving is 0.5, therefore:

Corrosion Loss (in.)	Factored Structural Resistance (kip)
0	204
1/32	170
1/16	140 (recommended resistance)

4.2 Preliminary Substructure Loading

Preliminary loads (compressive, lateral, moments) for each existing viaduct bridge pier and for service (SER I), strength (STR I) and extreme event (EE II) load combinations were developed by FIGG and provided to Haley & Aldrich. The loads were developed for use in evaluating reuse potential of existing spread footing and steel H-pile pier foundations. We understand that the loads were developed by FIGG based on the following:

- One continuous superstructure (i.e., only two bridge joints, one at each abutment)
- Multi-T and NEXT beam superstructure alternatives
- Bearing friction coefficient equal to 0.10
- Live load (LL) calculated based on modified and unmodified HL-93 loading for both one and two lanes of vehicular traffic

Maximum and minimum loads were also provided for STR I and EE II limit states for each combination of superstructure alternative (Multi-T or NEXT beam), live load (modified or unmodified HL-93) and one or two lanes of vehicular traffic. It should be noted that preliminary abutment loads were not developed by FIGG during this phase of the project and therefore were not considered as part of the existing viaduct bridge foundation evaluations summarized in the subsequent sections of this report.

In addition, piers were categorized by FIGG based on tributary length as follows:

- Substructure Group I (less than 61-ft long): Pier Nos. 1 through 5 and Pier 19
- Substructure Group II (61 to 64-ft long): Pier Nos. 6 through 13 and Pier 18
- Substructure Group III (64 to 76-ft long): Pier Nos. 14 through 17

In general, preliminary substructure loads are somewhat greater for longer spans (Substructure Group III piers) and lower for short spans (Substructure Group I piers). Preliminary substructure loading information is provided in Appendix F.

4.3 Preliminary Pile Group Evaluations

Pile group evaluations were completed at select pier locations based on preliminary substructure loading provided by FIGG, pile cap geometry and pile spacing shown on the historic drawings and the subsurface conditions encountered in test borings. Due to the large number of load combinations at each pile-supported substructure location, five pier locations were selected for further evaluation, which represent each Substructure Group (piers categorized based on tributary length) and the range in pile lengths and subsurface conditions encountered along the viaduct bridge. Preliminary pile group evaluations were completed at Pier Nos. 7, 10, 12, 15 and 19.

Pile group analyses were performed using the computer program FB-MultiPier, which is a nonlinear finite element analysis program that is capable of analyzing multiple bridge pier structures interconnected by bridge spans. The program couples nonlinear structural finite element analysis with nonlinear static soil models for axial, lateral and torsional soil behavior to provide a system of analysis for coupled bridge pier structures and foundation systems.

FB-MultiPier files for each substructure were prepared by Haley & Aldrich, including substructure loading data and pile group geometry (pile cap dimensions, pile layout and spacing, etc.). The files included subsurface soil, rock and groundwater conditions based on the conditions encountered in the design phase test borings as well as the structural resistance calculations summarized in previous sections of this report. Where significant differences in pile length existed within individual groups, based on as-built documentation provided by MaineDOT, the average pile length was modeled since FB-MultiPier only allows modeling of one pile length for a single group of piles. As discussed with FIGG and MaineDOT, the pile to pile cap connection was modeled as a pinned connection.

Initial evaluations were completed and results were presented to FIGG and MaineDOT. The initial results suggested that the adequacy of the pile groups at each pier location evaluated was controlled by the STR I limit state load combination, particularly the magnitude of the longitudinal moments, which are a result of having one continuous superstructure (i.e., only two bridge joints, one at each abutment). Based on our discussions with FIGG and MaineDOT we understand that a design exception will be used allowing the use of unmodified HL-93 live loads (lower as compared to modified HL-93). As a result, subsequent and more detailed pile group evaluations were completed considering STR I limit state load combinations, Multi-T and NEXT beam superstructure alternatives and unmodified HL-93 live load. This limited the total number of load combinations at each of the five pier locations being analyzed to four (for each superstructure alternative) as summarized below.

- Load Case 1 (LC1) – one lane of live load, minimum STR I load combination
- Load Case 2 (LC2) – two lanes of live load, minimum STR I load combination
- Load Case 3 (LC3) – one lane of live load, maximum STR I load combination
- Load Case 4 (LC4)– two lanes of live load, maximum STR I load combination

Results of the preliminary pile group evaluations are summarized below. Detailed results are provided for reference in Appendix F.

Substructure Location (Substructure Group[1])	Range in No. of Piles Exceeding Factored Structural Resistance[2]		Range in No. of Piles Subject to Tension		Range in No. of Piles Exceeding Yield Strength[3]		Range in No. of Piles With D/C > 1.0[4]	
	Multi-T Alt.	NEXT Alt.	Multi-T Alt.	NEXT Alt.	Multi-T Alt.	NEXT Alt.	Multi-T Alt.	NEXT Alt.
Pier 7 (II)	0-3	1-3	0-1	0-1	0-4	1-5	2-7	4-10
Pier 10 (II)	0-2	0-2	0	0	2-5	2-9	0-10	2-11
Pier 12 (II)	1-3	1-3	0-3	1-2	5-12	8-13	6-11	6-12
Pier 15 (III)	3-6	3-8	0-1	0-1	9-14	10-15	9-13	10-15
Pier 19 (I)	0-2	0-3	0-1	0-1	0-9	2-9	2-9	6-11

[1] – Refer to Section 4.2.
[2] – Refer to Section 4.1.5 (140 kip factored structural resistance).
[3] – Refer to Section 4.1.2 (33 ksi minimum ultimate yield stress).
[4] – D/C ratio calculated using the interaction equation and resistance factors for combined axial and flexural loading as specified in AASHTO LRFD.

Based on the results of the preliminary pile group evaluations summarized above we offer the following:

- Regardless of superstructure alternative (i.e., Multi-T or NEXT beam), it is our opinion that Group III substructures (long spans; pile-supported Pier Nos. 15 through 17) will require rehabilitation/strengthening considering that up to one-half of the piles within a group (up to 8 piles) exceed the factored structural resistance. In addition, depending on which load combination is being considered (LC1 through LC4), anywhere from one-half to virtually all of the total number of piles in the group (i.e., 9 to 15 of 16 piles) exceed yield strength and demand-to-capacity ratio limits (33 ksi and 1.0).
- Regardless of superstructure alternative (i.e., Multi-T or NEXT beam), it is our opinion that Group II substructures (intermediate spans; pile-supported Pier Nos. 6 and 7, 10 through 12 and 18) will also require rehabilitation/strengthening considering that up to approximately one-third to three-quarters of the total number of piles within the group (i.e., 4 to 13 of 16 piles) exceed yield strength and demand-to-capacity limits depending on which load combination (LC1 through LC4) is being considered. In addition, up to approximately 20 percent of the piles within a group (i.e., up to 3 of 16 piles) exceed the factored structural resistance.
- Regardless of superstructure alternative (i.e., Multi-T or NEXT beam), it is our opinion that Group I substructures (short spans; pile-supported Pier Nos. 5 and 19) will likely not require rehabilitation/strengthening given the limited number of piles that exceed limiting values (factored structural resistance, yield strength and demand-to-capacity ratio). Given the limited number of piles within the group and the limited length of pile over which the overstress occurs, it is our opinion that a more detailed analysis during final design will likely indicate that rehabilitation/strengthening of Group I substructures will not be required.
- Based on our discussions with FIGG we understand that the current pile-to-pile cap connection offers little to no tension resistance. However, given the limited number of piles within a pile group, regardless of Substructure Group or superstructure alternative being considered, that are subject to tension it is our opinion that this is acceptable. A more detailed analysis of structural and geotechnical tension resistance during final design will be required to confirm.

Please recall that the preliminary pile group evaluations summarized above are based on loads developed by FIGG for <u>one continuous superstructure</u> (i.e., only two bridge joints, one at each abutment), which results in large longitudinal moments. Based on our discussions with FIGG, we understand that breaking the superstructure up into multiple (two or three) segments to reduce the magnitude of the loads (particularly the longitudinal moments) will result in a reduction in the number of substructures that require rehabilitation/strengthening. As a result, we understand that FIGG will include placeholder costs in the construction cost estimate to be included in the PDR for the rehabilitation/strengthening of four substructures for the Multi-T superstructure alternative and six substructures for the NEXT beam superstructure alternative as discussed in the following section of this report. We also understand that the number of substructures requiring rehabilitation/strengthening for each superstructure alternative was determined by FIGG based on results of the preliminary pile group evaluations presented herein and a comparison of the superstructure loads for one-segment and two-segment superstructures (for both Multi-T and NEXT beam superstructure alternatives). In addition, the substructure rehabilitation/strengthening placeholder costs that will be included in the PDR will be approximately 12 percent higher for the NEXT beam alternative as compared to the Multi-T alternative due to the difference in the magnitude of the loads as judged by FIGG.

Please note that preliminary pile group evaluations modeling loads for a two- or three-segment superstructure were not completed as part of this phase of work and were not used as justification for determining the number of substructures requiring rehabilitation/strengthening. It is possible that the actual number of substructures requiring rehabilitation/strengthening may be more or less than the number used to develop the cost estimate included in the PDR.

4.4 Preliminary Pile-Supported Substructure Rehabilitation Recommendations

As discussed above, based on the results of the preliminary pile group evaluations several pile-supported piers will require rehabilitation/strengthening to adequately resist the demands of the new superstructure (Multi-T or NEXT beam alternatives). The exact number of piers and the extent of rehabilitation at each pier location will be determined during final design once decisions are made regarding the overall length of the structure (one segment with two bridge joints or multiple segments with a greater number of joints), superstructure loads are refined and more detailed analyses are completed.

However, several rehabilitation alternatives (deep foundation elements) were evaluated considering impacts to the existing structure and to adjacent vehicular traffic during installation, the ability to install the elements in low headroom conditions while the existing viaduct bridge remains in service and the ability of the elements to provide sufficient additional resistance. Based on these criteria and the extreme variability of the bedrock surface it is our opinion that drilled micropiles are the most technically feasible deep foundation alternative that can be used to rehabilitate/strengthen existing pile supported piers that are determined to be deficient during final design. We recommend that FIGG include placeholder costs for the rehabilitation/strengthening of substructures in the construction cost estimate included in the PDR. In addition, we recommend that placeholder costs be based on the following:

- Assume ten micropiles at each substructure location (20 total micropiles)
- Assume average 40 ft total micropile length, which includes 20 ft of soil drilling and a 20 ft rock socket
- Assume micropile outside diameter equal to 10.5 in.
- Assume permanent steel casing (F_y = 60 ksi) with ½-in. thick (min.) wall
- Assume reinforcement consists of a centralized grade 150 all-thread No. 11 bar extending full length (40 ft)
- Assume micropiles are tremie grouted using 5,000 psi (min. at 28 days) neat cement grout
- Assume two static (tension) load tests at non-production micropile locations prior to beginning production micropile installation
- Include contingency for laboratory verification testing of grout cube samples collected during micropile installation
- Include contingency for containing/collecting drill spoils and chemical testing/disposal depending on which pier substructures are retrofitted and whether soils are environmentally impacted
- Include contingency for obstruction removal/drilling

Additional items that should be considered in developing costs include but may not be limited to the following: mobilization, survey layout, excavation, dewatering, traffic control, reinforcing, cast-in-place concrete for enlarging the pile caps, and additional pile cap steel reinforcement.

Please note that the recommendations included herein relative to rehabilitation of pile-supported substructures that are deemed deficient during final design are not based on any technical evaluations. Therefore, the actual number, size and length of micropiles needed to rehabilitate the substructures may be more or less than what has been assumed for the purposed developing a cost estimate for the PDR.

4.5 Elastic Pile Compression

Settlement due to elastic shortening of the steel H-piles was evaluated based on the SER I limit state axial compressive loads provided by FIGG and the total number of piles at each pile-supported substructure location. Please note that calculating elastic shortening on the piles based on the total SER I limit state axial compressive load divided by the total number of piles within the group is approximate and the actual amount of deformation could be slightly more or less than the values summarized herein and shown in the calculations included in Appendix F. Estimating the elastic shortening in this manner was required since pile group evaluations were not completed at each substructure location modeling the preliminary SER I limit state loads provided by FIGG such that maximum factored pile forces could be calculated. We anticipate that elastic pile compression calculations and results will be revised/updated during final design.

In general, the calculated elastic shortening of the piles at each substructure location are less than ¼ in. The magnitude of displacement is generally greater for piles supporting piers located within Substructure Group III (long tributary lengths) and for longer piles supporting piers along the eastern third of the viaduct bridge (approximately Pier Nos. 15 through 19).

Please note that the actual magnitude of displacement will likely be less than the estimated values primarily due to the fact that the piles will be subject to some amount of vertical load during construction and will be dependent on the difference in load between what is removed (demolished) and what is added during construction. In general, the elastic shortening of the piles is anticipated to occur primarily during construction, soon after the new superstructure loads are applied.

4.6 Preliminary Evaluation of Existing Spread Footings

As discussed in previous sections of this report, Pier Nos. 1 through 4, 8 through 9, 13, 14 and Abutment 1 are supported on either CIP concrete spread footings bearing on bedrock or CIP concrete spread footings constructed on seal concrete, bearing on bedrock. Evaluation of their reuse potential in supporting a rehabilitated/replacement superstructure is presented below. Detailed calculations are provided for reference in Appendix F.

4.6.1 Original Design Bearing Capacity

Based on our review of the drawings it is unclear what allowable bearing pressure was used to design the footings that support the existing abutments/piers. However, the drawings do indicate that the maximum pressure at the corner of the spread footings is 13.7 ksf.

4.6.2 Recommended Bearing Resistance

The adequacy of the existing spread footing foundations was evaluated based on factored bearing resistances equal to 20 and 44 ksf, respectively at the STR I and EE II limit states. Factored bearing resistances were calculated in accordance with the semi-empirical procedures presented in AASHTO LRFD Section 10.6.3.2.2, specifically the procedures outlined by Carter & Kulhawy (1988).

Please note that the methodology outlined in AASHTO LRFD for determining bearing resistance of footings on bedrock is partially dependent on interpretive grading assigned to the recovered bedrock core in accordance with the Rock Mass Rating (RMR) System. The rock mass data collected during the preliminary phase of the project was used, in part, to adjust the RMR to account for strike and dip orientations relative to the existing spread footing foundations.

Based on the preliminary substructure loading information provided by FIGG and the existing spread footing dimensions shown on the historic drawings modified to account for eccentricity, contact pressures were calculated and compared to the factored bearing resistance (20 ksf) at each pier location (please recall that loads were not provided for Abutment 1). In general, the resulting contact pressures at the STR I limit states were less than 20 ksf with the exception of the following locations:

- Pier 8
  - NEXT beam superstructure alternative (20.1 ksf; one out of four load combinations)

- Pier 9
  - NEXT beam superstructure alternative (20.2 ksf; one out of four load combinations)

- Pier 13
  - Multi-T beam superstructure alternative (21.0 to 21.6 ksf; two out of four load combinations)
  - NEXT beam superstructure alternative (21.6 ksf; two out of four load combinations)

- Pier 14
  - Multi-T beam superstructure alternative (21.3 to 23.5 ksf; three out of four load combinations)
  - NEXT beam superstructure alternative (22.7 to 24.6 ksf; three out of four load combinations)

Calculated contact pressures under EE II limit state loading at each substructure location were below the calculated factored bearing resistance equal to 44 ksf (nominal bearing resistance with resistance factor equal to 1.0 under EE loading).

4.6.3 Sliding Resistance

Sliding resistance was evaluated in accordance with AASHTO LRFD Section 10.6.3.4 based on preliminary substructure loads provided by FIGG and neglecting lateral passive soil resistance (for conservatism).

Sliding resistance between the base of concrete seal and bedrock subgrade was evaluated using a coefficient of sliding friction equal to 0.49 (interface friction angle, δ, of 26 degrees) and STR and EE limit state resistance factors equal to 0.8 and 1.0, respectively in accordance with AASHTO LRFD Table 10.5.5.2.2-1. In general, the available factored sliding resistance was in all cases equal to or greater than the factored loads.

4.6.4 Eccentricity

Eccentricity was evaluated in accordance with AASHTO LRFD Section 10.6.3.3 and 10.6.4.2 based on preliminary substructure loads provided by FIGG. In general, the calculated eccentricities were in all cases less than the limiting value (0.45 x footing length/width for footings bearing on bedrock).

4.6.5 Settlement

Settlement was evaluated based on the SER I limit state axial compressive loads provided by FIGG and effective footing dimensions in accordance with the methodology outlined in AASHTO LRFD Section 10.6.2.4.4. The results indicate a negligible amount (less than 0.1 in.) of vertical deformation occurring under SER I limit state loading conditions.

Please note that the actual magnitude of displacement will likely be less than the estimated values primarily due to the fact that the footings will be subject to some amount of vertical load during construction and will be dependent on the difference in load between what is removed (demolished) and what is added during construction. In general, the elastic settlement of the footings is anticipated to occur primarily during construction, soon after the new superstructure loads are applied.

4.7 Preliminary Spread Footing-Supported Substructure Rehabilitation Recommendations

Based on the results of preliminary spread footing evaluations it is our opinion that rehabilitation will not be required. Contact pressures calculated at the STR I limit state at Pier Nos. 8, 9, 13 and 14 for the Multi-T and/or NEXT beam superstructure alternatives slightly exceeded the factored bearing resistance (20 ksf). However, once a more detailed analysis is completed during final design we anticipate the factored bearing resistance will increase such that the current results will be acceptable and rehabilitation/strengthening will not be required. As a result, we recommend that place holder costs for spread footing substructure rehabilitation/strengthening not be included in the PDR.

5. FUTURE WORK

Based on the results of the preliminary geotechnical analyses summarized herein and our subsequent discussions with FIGG and MaineDOT additional geotechnical work scope items required for final design are summarized below.

- Coordinate with FIGG as necessary to understand the superstructure rehabilitation alternative(s) that has been selected for final design.
- Based on the selected superstructure rehabilitation alternative(s) and associated loads developed by FIGG, conduct spread footing and pile group evaluations as necessary to determine the actual number of abutment/pier substructures that require rehabilitation/strengthening.
- Complete a laboratory testing program necessary to determine engineering properties needed to design abutment/pier substructure rehabilitation/strengthening. We currently anticipate laboratory testing will be completed on rock core samples collected during the supplemental phase subsurface investigation.
- Conduct evaluations necessary to determine the type, size, number and general layout of micropiles needed to rehabilitate/strengthen pier/abutment foundations that are determined to be deficient during final design.
- Prepare final geotechnical design report summarizing results of evaluations and recommendations.
- Prepare geotechnical-related drawings and special provisions related to pier/abutment substructure rehabilitation/strengthening, if necessary.
- Provide construction oversight of pier/abutment substructure rehabilitation/strengthening, if necessary.

Please note that we do not currently anticipate the need to conduct additional subsurface investigations during final design.

REFERENCES

1. Report entitled, "Subsurface Investigations For Carlton Bridge and West Approach in Bath, Maine," prepared by the Maine State Highway Commission, Soil Division, dated April 1957.

2. Report entitled, "Detailed Study in the Vicinity of the Abutment at Station 398+78, Carlton Bridge, Carlton, Maine," prepared by the Maine State Highway Commission, Soils Division, dated 8 May 1957.

3. Drawings entitled, "Carlton Bridge Widening and West Approach between Bath and Woolwich, Sagadahoc County, Maine, Federal Aid Project No. U-026-1(8)," prepared by the State of Maine State Highway Commission, dated January 1957, Sheet Nos. 1, 10-14, 33-39 of 148.

4. Report entitled, "Supplemental Field and Laboratory Investigation, Bath/Woolwich Bridge, Bath and Woolwich, Maine," prepared by Haley & Aldrich, Inc. for the Maine Department of Transportation, dated 30 May 1997.

5. Report entitled, "Preliminary Geotechnical Design Report, U.S. Route 1 Viaduct Bridge Rehabilitation, MaineDOT WIN 19273.00, Bath, Maine," prepared by Haley & Aldrich, Inc. for the Maine Department of Transportation, dated 17 September 2013.

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TABLE I
Test Boring Location Data
Route 1 Viaduct Bridge Rehabilitation
MaineDOT WIN 19273.00
Bath, Maine

Project Phase	Test Boring No.[1]	Approximate Substructure Location	Ground Surface Elevation[3]	Station[4]	Offset Distance & Direction[4]	Coordinates[2]	
						Northing	Easting
Preliminary	BB-BROV-101	Pier 6	27.4	389 + 49.9	10.4' LT	393,314	1,131,000
	BB-BROV-102	Pier 10 / Pier 11	17.2	392 + 81.3	9.9' LT	393,408	1,131,318
	BB-BROV-103	Pier 18 / Pier 19	7.8	397 + 84.7	18.9' RT	393,523	1,131,808
Supplemental	BB-BROV-201	Pier 5	29.0	389 + 4.9	8.2' RT	393,283	1,130,962
	BB-BROV-202	Pier 7	24.9	390 + 30.1	6.5' RT	393,321	1,131,082
	BB-BROV-203	Pier 10	19.2	392 + 21.4	4.7' LT	393,386	1,131,262
	BB-BROV-204	Pier 12	15.7	393 + 48.0	4.7' LT	393,422	1,131,383
	BB-BROV-205	Pier 19	8.6	398 + 10.7	6.0' RT	393,543	1,131,830
	BB-BROV-206	Abutment 2	10.4	398 + 71.3	19.2' RT	393,542	1,131,890

Notes:
[1] - Test boring locations are shown on Figure 2, Site and Subsurface Exploration Location Plan.
[2] - As-drilled coordinates of test borings were determined by MaineDOT using GPS survey equipment, are measured in feet and reference NAD83, Maine 2000 West Zone coordinate system. Coordinates shown were rounded to the nearest 1 ft.
[3] Ground surface elevations at test boring locations were determined in the field by MaineDOT using optical survey equipment, are measured in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
[4] Station and offset information estimated by Haley & Aldrich based on baseline information provided by FIGG.

Prepared By:	BCS	7/11/2014
Checked By:	JLL	7/16/2014
Reviewed By:	WAC	8/4/2014

TABLE II
Test Boring Subsurface Data
Route 1 Viaduct Bridge Rehabilitation
MaineDOT WIN 19273.00
Bath, Maine

Project Phase	Test Boring No.[1]	Ground Surface Elevation[2]	Approximate Strata Thickness[3] (ft)					Approximate Elevation of Top of Competent Bedrock	Approximate Elevation of Bottom of Exploration
			Bituminous Concrete/ Man-Placed Fill	Organic Deposit	Marine Deposit (Clay)	Marine Deposit (Sand)	Glacial Till / Weathered Bedrock		
Preliminary	BB-BROV-101	27.4	4.3	NE	5.0	NE	3.0	15.1	10.2
	BB-BROV-102	17.2	2.5	NE	7.5	NE	0.7	6.5	1.3
	BB-BROV-103	7.8	3.0	5.5	38.3	NE	2.2	-41.2	-46.2
Supplemental	BB-BROV-201	29.0	6.5	NE	NE	7.1	NE	15.4	0.5
	BB-BROV-202	24.9	5.5	NE	14.0	NE	1.9	3.5	-12.3
	BB-BROV-203	19.2	5.0	NE	11.7	6.8	NE	-4.3	-19.5
	BB-BROV-204	15.7	14.2	NE	11.3	NE	0.8	-10.6	-25.2
	BB-BROV-205	8.6	2.0	4.5	36.9	NE	3.1	-37.9	-52.9
	BB-BROV-206	10.4	8.0	NE	33.0	NE	NE	-30.6	-48.9

Notes:
[1] Test boring locations are shown on Figure 2, Site and Subsurface Exploration Location Plan.
[2] Ground surface elevations at test boring locations were determined in the field by MaineDOT using optical survey equipment, are measured in feet and reference the North American Vertical Datum of 1988 (NAVD 88).
[3] "NE" indicates stratum was not encountered in test boring.

Prepared By:	BCS	7/11/2014
Checked By:	JLL	7/16/2014
Reviewed By:	WAC	8/4/2014

TABLE III

Existing Abutment/Pier Pile Tip and Bedrock Elevation Comparison
Route 1 Viaduct Bridge Rehabilitation
MaineDOT WIN 19273.00
Bath, Maine

Substructure Location	Test Boring/ Test Probe No.[6]	Pile Cap Elevations[1,4]			Pile Lengths (ft)			Pile Tip Elevations[4]			Top of Bedrock Elevation[5,6]	
		Top of Pile Cap	Top of Pile	Bottom of Pile Cap	As-Built Low End[2]	As-Built High End[2]	NDT[3]	As-Built Low End	As-Built High End	NDT	Historic Test Boring/Probe*	Recent Test Boring
Pier 5	BB-BROV-201, Rod Sounding*	27.1	24.1	23.1	9.3	13.9	9.2	10.2	14.8	14.9	14.3*	15.4
Pier 6	BB-BROV-101, Boring No. 4	24.7	21.7	20.7	9.3	28	--	-6.3	12.4	--	4.1	15.1
Pier 7	BB-BROV-202, Boring No. 5	22.8	19.8	18.8	15.7	37.2	13.5	-17.4	4.1	6.3	-2.4	3.5
Pier 10	BB-BROV-203, Boring No. 14	17.2	14.2	13.2	9.8	18.7	19.7	-4.5	4.4	-5.5	1.8	-4.3
Pier 11	BB-BROV-102, Rod Sounding*	15.7	12.7	11.7	5.3	10	--	2.7	7.4	--	25.8*	6.5
Pier 12	BB-BROV-204, Boring No. 7	14.0	11.0	10.0	15	22.5	--	-11.5	-4.0	--	-8.5	-10.6
Pier 15	Boring No. 13	9.2	6.2	5.2	8.8	44	--	-37.8	-2.6	--	-13.3	--
Pier 16	Rod Sounding*	7.3	4.3	3.3	28.1	44	--	-39.7	-23.8	--	-28.4*	--
Pier 17	Rod Sounding*	6.6	3.6	2.6	34.3	87	--	-83.4	-30.7	--	-32.4*	--
Pier 18	BB-BROV-103, Boring No. 17	5.7	2.7	1.7	34.7	63.7	--	-61.0	-32.0	--	-50.0	-41.2
Pier 19	BB-BROV-205, Rod Sounding*	6.5	3.5	2.5	31.3	46.2	20.3/37.7	-42.7	-27.8	-17.2/-34.6	-30.2*	-37.9
Abutment 2	BB-BROV-206, Boring Nos. 19, 20	7.8	4.8	3.8	24.7	44.8	30.0	-40.0	-19.9	-25.2	-28.2/-23.4	-30.6

Notes:

[1] - Taken from historic bridge drawings provided by MaineDOT and converted to NAVD 88 datum by Haley & Aldrich by subtracting 48.64 ft.

[2] - Taken from as-built construction documentation provided by MaineDOT.

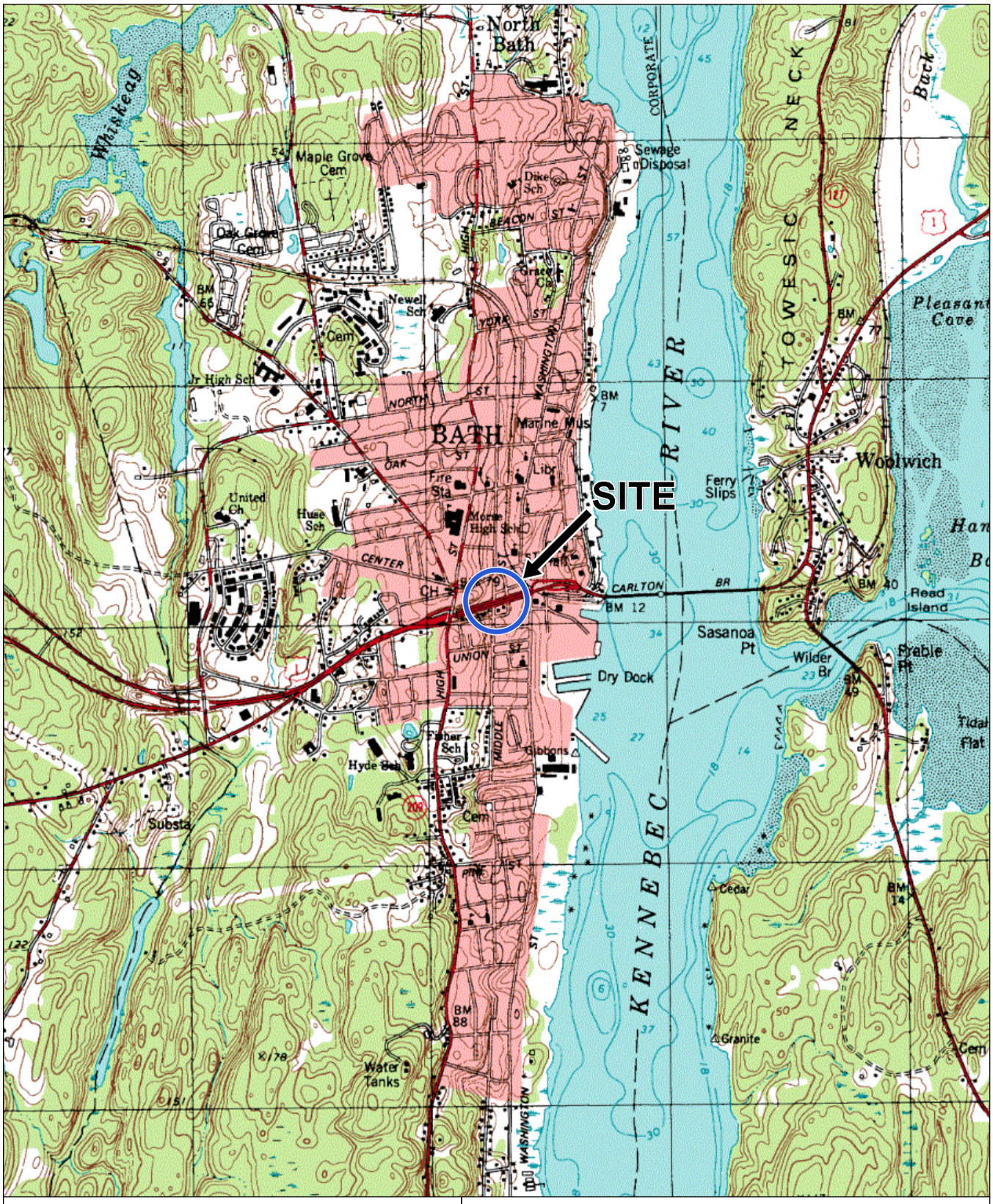
[3] - NDT = non-destructive testing; testing performed by Olson Engineering on 8 April 2014.

[4] - Elevations reference the North American Vertical Datum of 1988.

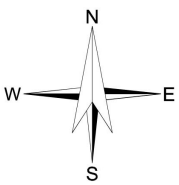
[5] - Taken from recent test boring logs and interpreted from historic probe/test boring logs by Haley & Aldrich.

[6] - * indicates elevation is approximate and is based on rod probe refusal surface elevation and may not be indicative of top of bedrock.

Prepared By:	BCS	7/11/2014
Checked By:	JLL	7/16/2014
Reviewed By:	WAC	8/4/2014



SITE COORDINATES: 43°54'40"N,69°49'6"W



U.S.G.S. QUADRANGLE: BATH, ME

HALEY & ALDRICH

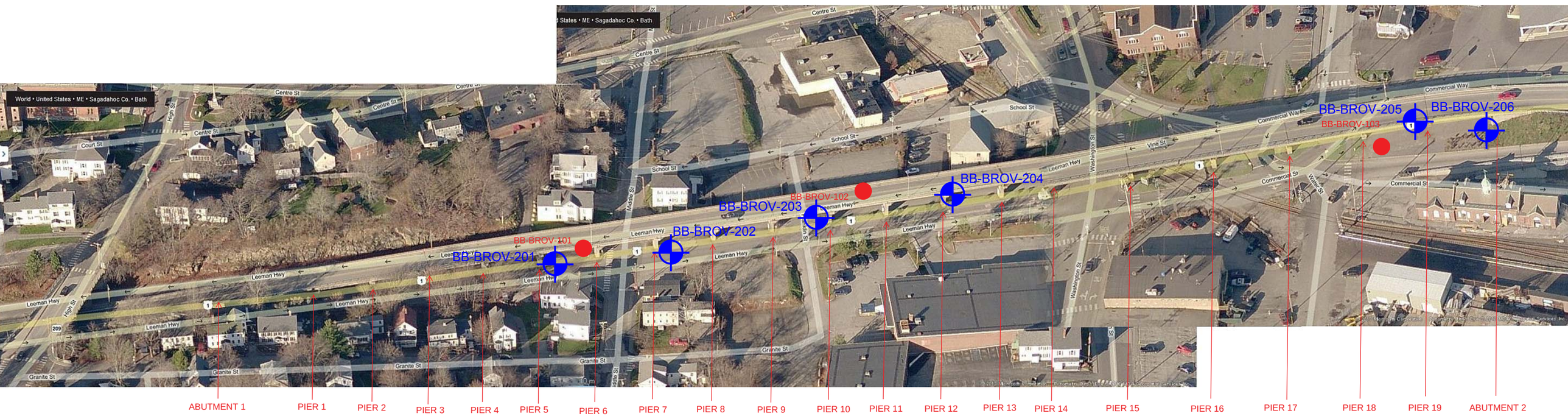
ROUTE 1 VIADUCT BRIDGE REHABILITATION
MAINEDOT WIN 19273.00
BATH, MAINE

PROJECT LOCUS

SCALE: 1:24,000
AUGUST 2014

FIGURE 1

38751-030 1.PDF



NOTES:

1. LOCATIONS OF TEST BORINGS ARE APPROXIMATE AND WERE DETERMINED BY TAPING/PACING DISTANCES FROM EXISTING SITE FEATURES. "AS-DRILLED" LOCATION WERE SURVEYED BY MAINEDOT UPON THE COMPLETION OF DRILLING.

2. SUBSURFACE EXPLORATIONS WERE MONITORED IN THE FIELD BY HALEY & ALDRICH, INC.

3. REFER TO APPENDIX A FOR TEST BORING LOGS.

LEGEND:

BB-BROV-101 — DESIGNATION AND APPROXIMATE LOCATION OF TEST BORING DRILLED BY MAINE TEST BORINGS OF HERMON, MAINE IN MARCH 2013

BB-BROV-201 — DESIGNATION AND APPROXIMATE LOCATION OF TEST BORING DRILLED BY THE MAINE DEPARTMENT OF TRANSPORTATION IN MARCH AND APRIL 2014

HALEY & ALDRICH

ROUTE 1 VIADUCT BRIDGE REHABILITATION
MAINEDOT WIN 19273.00
BATH, MAINE

SITE AND SUBSURFACE EXPLORATION LOCATION PLAN

SCALE: AS SHOWN
JULY 2014

FIGURE 2

G:\PROJECTS\40725 - route 1 viaduct bath\Preliminary Design Report\Figures\Figure 2.pdf
bsteinert

APPENDIX A

Test Boring Logs

Supplemental Phase
Test Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly-graded gravels, gravel sand mixtures, little or no fines
		GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS (little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly-graded sands, gravelly sand, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures.
FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts
	HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.

Desired Soil Observations: (in this order)

Color (Munsell color chart)
Moisture (dry, damp, moist, wet, saturated)
Density/Consistency (from above right hand side)
Name (sand, silty sand, clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Bonding (well, moderately, loosely, etc., if applicable)
Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)
Geologic Origin (till, marine clay, alluvium, etc.)
Unified Soil Classification Designation
Groundwater level

TERMS DESCRIBING DENSITY/CONSISTENCY

Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) silty or clayey gravels; and (3) silty, clayey or gravelly sands. Consistency is rated according to standard penetration resistance.

Modified Burmister System

Descriptive Term	Portion of Total
trace	0% - 10%
little	11% - 20%
some	21% - 35%
adjective (e.g. sandy, clayey)	36% - 50%

Density of Cohesionless Soils	Standard Penetration Resistance N-Value (blows per foot)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

Fine-grained soils (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) gravelly, sandy or silty clays; and (3) clayey silts. Consistency is rated according to shear strength as indicated.

Consistency of Cohesive soils	SPT N-Value blows per foot	Approximate Undrained Shear Strength (psf)	Field Guidelines
Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily Penetrates
Soft	2 - 4	250 - 500	Thumb easily penetrates
Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort
Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

$$RQD = \frac{\text{sum of the lengths of intact pieces of core* > 100 mm}}{\text{length of core advance}}$$

*Minimum NQ rock core (1.88 in. OD of core)

Correlation of RQD to Rock Mass Quality

Rock Mass Quality	RQD
Very Poor	<25%
Poor	26% - 50%
Fair	51% - 75%
Good	76% - 90%
Excellent	91% - 100%

Desired Rock Observations: (in this order)

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Lithology (igneous, sedimentary, metamorphic, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90)
- -spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m)
- -tightness (tight, open or healed)
- -infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock mass quality (very poor, poor, etc.)
ref: AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A
Recovery

Maine Department of Transportation
Geotechnical Section
Key to Soil and Rock Descriptions and Terms
Field Identification Information

Sample Container Labeling Requirements:

PIN	Blow Counts
Bridge Name / Town	Sample Recovery
Boring Number	Date
Sample Number	Personnel Initials
Sample Depth	

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 1 Viaduct Bridge Rehabilitation
Location: Bath, Maine

Boring No.: BB-BROV-201
PIN: 19273.00

Driller:	MaineDOT	**Elevation (ft.)**	29.0	**Auger ID/OD:**	--
Operator:	C. Giles	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	S. Shay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	HW-300#/24 in. SS-140#/30
Date Start/Finish:	3-25-2014/3-25-2014	**Drilling Method:**	HW Drive	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1130962, N393283.3 (See Plan)	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	6.0
Hammer Efficiency Factor:	0.867	**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	28.90	-BITUMINOUS CONCRETE- 0.1	
	1D	24/15	2.0 - 4.0	7/16/13/8	29	42			Olive-brown, medium dense, fine to medium SAND -FILL-(SP)	
5	2D	24/9	5.0 - 7.0	4/6/7/27	13	19	HW	24.50	4.5 Dark olive-gray, wet, stiff, CLAY, some silt, trace sand, trace asphalt particles, appears reworked -FILL-(CL)	
							40	22.50	6.5 Olive-brown with weak mottling, medium dense, fine to coarse SAND, little silt -MARINE DEPOSIT-(SM)	
							68		Brown, medium dense, silty fine to medium SAND, trace gravel -MARINE DEPOSIT-(SM)	
10	3D	24/13	9.0 - 11.0	23/13/10/10	23	33	35			
							41			
							33			
							23			
							75(7.0")	15.40	13.6 Note: Attempted split spoon sample at 13.6 ft (several blows with no penetration/no recovery). Advanced borehole with roller bit to 14.0 ft prior to coring. Top of bedrock at El. 15.4	
15	R1	60/60	14.0 - 19.0	RQD = 90%			NQ Core		Gray/white, fine to coarse grained SCHIST. Highly foliated. Very hard, slightly weathered. Joints dipping at low angles, close to moderate, rough, planar, fresh, tight. Rock Mass Quality=Good -CAPE ELIZABETH FORMATION- Recovery=100% R1 Core Times (min:sec): 14.0-15.0' (5:00); 15.0-16.0' (5:00); 16.0-17.0' (5:00); 17.0-18.0' (6:00); 18.0-19.0' (6:00)	
20	R2	60/60	19.0 - 24.0	RQD = 87%					Gray, fine grained SCHIST. Weak foliation. Very hard, slightly weathered. Primary joints dipping at low angles, moderate, rough, planar, fresh, tight. Secondary high angle joints, wide, rough, stepped, fresh, tight. Rock Mass Quality=Good -CAPE ELIZABETH FORMATION- Recovery=100% R2 Core Times (min:sec): 19.0-20.0' (4:00); 20.0-21.0' (4:00); 21.0-22.0' (4:00); 22.0-23.0' (4:00); 23.0-24.0' (5:00)	
25	R3	54/54	24.0 - 28.5	RQD = 96%				0.50	Gray/white, fine to coarse grained SCHIST. Highly foliated. Very hard, slightly weathered. Joints dipping at low angles, close to moderate, rough, planar, fresh, tight. Core barrel jammed at 28.5 ft with portion of R2 recovered. Minimal water loss R1-R3. Rock Mass Quality=Excellent -CAPE ELIZABETH FORMATION- Recovery=100% R3 Core Times (min:sec): 24.0-25.0' (4:00); 25.0-26.0' (6:00);	
30										

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Installed 2-in. diameter solid PVC pipe in completed borehole to facilitate non-destructive testing. PVC pipe was installed from the bottom of the borehole up to existing ground surface.

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: Route 1 Viaduct Bridge Rehabilitation
Location: Bath, Maine

Boring No.: BB-BROV-201
PIN: 19273.00

Driller:	MaineDOT	**Elevation (ft.)**	29.0	**Auger ID/OD:**	--
Operator:	C. Giles	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	S. Shay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	HW-300#/24 in. SS-140#/30
Date Start/Finish:	3-25-2014/3-25-2014	**Drilling Method:**	HW Drive	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1130962, N393283.3 (See Plan)	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	6.0
Hammer Efficiency Factor:	0.867	**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										26.0-27.0' (4:00); 27.0-28.0' (5:00); 28.0-28.5' (6:00)	
										28.5	
										Bottom of Exploration at 28.5 feet below ground surface.	
35											
40											
45											
50											
55											
60											

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Installed 2-in. diameter solid PVC pipe in completed borehole to facilitate non-destructive testing. PVC pipe was installed from the bottom of the borehole up to existing ground surface.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 1 Viaduct Bridge Rehabilitation
Location: Bath, Maine

Boring No.: BB-BROV-202
PIN: 19273.00

Driller:	MaineDOT	**Elevation (ft.)**	24.9	**Auger ID/OD:**	--
Operator:	C. Giles	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	S. Shay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	HW-300#/24 in. SS-140#/30
Date Start/Finish:	3-26-2014/3-26-2014	**Drilling Method:**	HW Drive	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131082, N393320.6 (See Plan)	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	5.0
Hammer Efficiency Factor:	0.867	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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	24.80	-BITUMINOUS CONCRETE- 0.1	
	1D	24/3	2.0 - 4.0	2/3/2/2	5	7			Brown, loose, fine to coarse SAND -FILL-(SP)	
									Note: Stratum change based on change in effort to drive split spoon.	
5	2D	24/18	5.0 - 7.0	1/4/7/8	11	16	HW	19.40	5.5 Dark olive-gray to olive-brown, stiff, silty CLAY -MARINE DEPOSIT-(CL)	
							40			
							55			
							53			
	3D	24/24	9.0 - 11.0	3/4/4/4	8	12	37		Olive-brown, medium stiff, silty CLAY -MARINE DEPOSIT-(CL)	
10							55			
							63			
							71	12.40	12.5	
							70			
	4D	24/24	14.0 - 16.0	WOH(24.0")			53		Olive-brown grading to olive-gray, very soft, silty CLAY -MARINE DEPOSIT-(CL)	
15							43			
							46			
							47			
							47			
	5D	24/18	19.0 - 21.0	1/13/5/5	18	26	39	5.40	Olive-gray, very soft, silty CLAY -MARINE DEPOSIT-(CL)	
20							85		19.5	
	R1	60/60	21.9 - 26.9	RQD = 88%			50(5.0")	3.50	Gray, medium dense, clayey fine to medium SAND, some silt, trace gravel -GLACIOFLUVIAL DEPOSIT-(SC)	
							NQ Core		21.4	
									Top of Bedrock at El. 3.5 Note: Advanced borehole with roller bit to 21.9 ft prior to coring. Gray black/white, fine to coarse grained micaceous SCHIST. Highly foliated. Very hard to hard, slightly weathered. Joints at high angles, moderate, rough, stepped, fresh, tight. Occasional quartz/pegmatite intrusions. Rock Mass Quality=Good -CAPE ELIZABETH FORMATION- Recovery=100% R1 Core Times (min:sec): 21.9-22.9' (4:00); 22.9-23.9' (3:00); 23.9-24.9' (3:00); 24.9-25.9' (3:00); 25.9-26.9' (3:00) Gray black/white, fine to coarse grained micaceous SCHIST. Highly foliated. Very hard to hard, slightly weathered. Joints at low angles,	
25	R2	60/60	26.9 - 31.9	RQD = 78%						
30										

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Installed 2-in. diameter solid PVC pipe in completed borehole to facilitate non-destructive testing. PVC pipe was installed from the bottom of the borehole up to existing ground surface.

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.

Boring No.: BB-BROV-202

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 1 Viaduct Bridge Rehabilitation
Location: Bath, Maine

Boring No.: BB-BROV-202
PIN: 19273.00

Driller:	MaineDOT	**Elevation (ft.)**	24.9	**Auger ID/OD:**	--
Operator:	C. Giles	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	S. Shay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	HW-300#/24 in. SS-140#/30
Date Start/Finish:	3-26-2014/3-26-2014	**Drilling Method:**	HW Drive	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131082, N393320.6 (See Plan)	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	5.0
Hammer Efficiency Factor:	0.867	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MV = Unsuccessful Insitu Vane Shear Test attempt
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SSA = Solid Stem Auger
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WOH = weight of 140lb. hammer
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WO1P = Weight of one person
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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
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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							NQ Core			moderate, rough, stepped, fresh, tight, close with extremely fractured zone from approximately 29.0 to 30.0 ft. Rock Mass Quality=Good -CAPE ELIZABETH FORMATION- Recovery=100% R2 Core Times (min:sec): 26.9-27.9' (2:00); 27.9-28.9' (3:00); 28.9-29.9' (2:00); 29.9-30.9' (3:00); 30.9-31.9' (3:00)	
	R3	60/60	31.9 - 36.9	RQD = 97%							
35								-12.30		Gray/black/white, fine to coarse grained micaceous SCHIST. Highly foliated. Very hard, fresh. Joints at low angle angles, moderate, rough, stepped, fresh surfaces, tight. Rock Mass Quality=Excellent -CAPE ELIZABETH FORMATION- Recovery=100% R3 Core Times (min:sec): 31.9-32.9' (3:00); 32.9-33.9' (4:00); 33.9-34.9' (4:00); 34.9-35.9' (3:00); 35.9-36.9' (4:00)	
										37.2 **Bottom of Exploration at 37.2 feet below ground surface.**	
40											
45											
50											
55											
60											

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Installed 2-in. diameter solid PVC pipe in completed borehole to facilitate non-destructive testing. PVC pipe was installed from the bottom of the borehole up to existing ground surface.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 1 Viaduct Bridge Rehabilitation
Location: Bath, Maine

Boring No.: BB-BROV-203
PIN: 19273.00

Driller:	MaineDOT	**Elevation (ft.)**	19.2	**Auger ID/OD:**	--
Operator:	C. Giles	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	S. Shay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	HW-300#/24 in. SS-140#/30
Date Start/Finish:	3-27-2014/3-27-2014	**Drilling Method:**	HW Drive	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131262, N393385.7 (See Plan)	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	5.0
Hammer Efficiency Factor:	0.867	**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							SSA	19.10		-BITUMINOUS CONCRETE- 0.1	
	1D	24/14	2.0 - 4.0	2/8/12/22	20	29				Dark olive-brown, medium dense, fine to coarse SAND, little silt -FILL-(SP-SM)	
							7				
5	2D	24/21	5.0 - 7.0	9/10/13/13	23	33	23	14.20		5.0 Olive-brown, very stiff, CLAY, little silt -MARINE DEPOSIT-(CL)	
							40				
							55				
							69				
	3D	24/24	9.0 - 11.0	5/4/5/7	9	13	58			Olive-brown, stiff, CLAY, little silt -MARINE DEPOSIT-(CL)	
10							64				
							70				
							85	6.70		12.5	
							84				
	4D	24/24	14.0 - 16.0	WOH(24.0")			51			Olive-gray, very soft, silty CLAY -MARINE DEPOSIT-(CL)	
15							57				
							77	2.50		16.7	
							81			Note: Prepare borehole for vane shear test at 17.0 ft, advance roller bit to 16.0 ft, unable to push vane beyond 16.7 on possible sand or gravel. Aborted vane shear test.	
							73				
	5D	24/4	19.0 - 21.0	7/3/3/3	6	9	33			Dark olive-gray, medium stiff, sandy CLAY, little gravel -MARINE DEPOSIT-(CL/SC)	
20							37				
							53				
							200			Note: Possible highly weathered rock from 23.0 to 23.5 ft (unable to drive casing beyond 23.5 ft).	
	R1	60/60	23.7 - 28.7	RQD = 87%			100(6.0")	-4.30		23.5	
							NQ Core			Top of Bedrock at El. -4.3 Note: Advance borehole to 23.7 ft with roller bit prior to coring. Gray, coarse grained quartz/pegmatite SCHIST. Extremely foliated. Very hard, slightly weathered. Joints dipping at moderate angles, close, rough, planar, slightly weathered at micaceous zones, open. Rock Mass Quality=Good -CAPE ELIZABETH FORMATION- Recovery=100% R1 Core Times: not recorded Dark gray, fine to coarse grained SCHIST. Extremely foliated, pegmatite instrusions with occasional schist xenoliths. Very hard,	
25											
	R2	60/60	28.7 - 33.7	RQD = 88%							
30											

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Installed 2-in. diameter solid PVC pipe in completed borehole to facilitate non-destructive testing. PVC pipe was installed from the bottom of the borehole up to existing ground surface.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 1 Viaduct Bridge Rehabilitation
Location: Bath, Maine

Boring No.: BB-BROV-203
PIN: 19273.00

Driller:	MaineDOT	**Elevation (ft.)**	19.2	**Auger ID/OD:**	--
Operator:	C. Giles	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	S. Shay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	HW-300#/24 in. SS-140#/30
Date Start/Finish:	3-27-2014/3-27-2014	**Drilling Method:**	HW Drive	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131262, N393385.7 (See Plan)	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	5.0
Hammer Efficiency Factor:	0.867	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MD = Unsuccessful Split Spoon Sample attempt
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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							NQ Core			slightly weathered. Primary high angle joints, moderate, rough, stepped, slightly weathered, tight, occasional moderately dipping secondary joints present. -Rock Mass Quality=Good -CAPE ELIZABETH FORMATION- Recovery=100% R2 Core Times: not recorded	
	R3	60/60	33.7 - 38.7	RQD = 95%						White/gray, fine to coarse grained quartzite SCHIST. Foliation trending at moderate angles. Very hard, fresh. Joints dipping at moderate angles, wide except for two visible close, smooth, planar, micaceous joint surfaces, tight, minimal water loss R1-R3. Rock Mass Quality=Excellent -CAPE ELIZABETH FORMATION- Recovery=100% R3 Core Times: not recorded	
35								-19.50		38.7	
										Bottom of Exploration at 38.7 feet below ground surface.	
40											
45											
50											
55											
60											

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 1 Viaduct Bridge Rehabilitation
Location: Bath, Maine

Boring No.: BB-BROV-204
PIN: 19273.00

Driller:	MaineDOT	**Elevation (ft.)**	15.7	**Auger ID/OD:**	--
Operator:	C. Giles	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	S. Shay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	HW-300#/24 in. SS-140#/30
Date Start/Finish:	3-28-2014/4-01-2014	**Drilling Method:**	HW Drive	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131383, N393421.8 (See Plan)	**Casing ID/OD:**	SSA-2.25 in. ID; HW-4.0 in. ID	**Water Level*:**	7.3
Hammer Efficiency Factor:	0.867	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
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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							SSA	15.60		-BITUMINOUS CONCRETE- 0.1	
	1D	24/15	2.0 - 4.0	2/4/4/4	8	12				Brown, medium dense, fine to coarse SAND -FILL-(SP)	
5	2D	24/6	5.0 - 7.0	13/14/5/2	19	27	6	9.70		Brown, medium dense, fine to coarse SAND, coarse gravel piece lodged in spoon tip -FILL-(SP) 6.0	
							27			Note: Stratum change estimated on change in effort to drive split spoon.	
							25				
							18				
	3D	24/14	9.5 - 11.5	3/2/1/1	3	4	9(6.0)**			**Note: Casing dropped 6 in. during load in soft material. Very soft ORGANIC SOIL with up to 25% peat fibers, trace rubber piece, trace fine sand, no organic odor, possibly disturbed -FILL-(OL/OH)	
10							15				
							74				
	4D	24/14	12.0 - 14.0	47/25/27/33	52	75	16			Disturbed gravelly ORGANIC SOIL -FILL-(OL/OH)	
							98	2.70		13.0	
							18*	1.50		Wood, possible pile or buried log -FILL- 14.2	
15	5D	24/9	15.0 - 17.0	7/14/15/9	29	42	27*			Olive gray, very stiff, silty CLAY, trace sand, trace gravel -MARINE DEPOSIT-(CL)	
							32*				
							38*			Note: Advanced borehole with roller bit ahead of casing; * = casing blows in disturbed soil.	
							34*				
							36*				
20	6D	24/24	20.0 - 22.0	WOH(12.0)/3/2	4	6	21*			Olive-gray and olive-brown, soft, interbedded silty CLAY and sandy CLAY, trace fine gravel -MARINE DEPOSIT-(CL/SC)-	
							32*				
							43*				
							60*			Note: Advance roller bit ahead of casing to 24.5 ft. Possible cobble at 24.0 ft.	
	7D	18/12	24.5 - 26.0	6/4/100	104	150	84			Olive-gray, loose, clayey SAND -MARINE DEPOSIT-(SC)	
25							125	-9.80		25.5	
	R1	60/56	26.3 - 31.3	RQD = 88%			NQ Core	-10.60		Top of Bedrock at El. -9.8 Gray/black, very dense, highly weathered micaceous ROCK 26.3	
										Dark gray, fine to medium grained, micaceous SCHIST. Foliation generally trending at low angles with most drill breaks along foliation planes. Hard to moderately hard, slightly weathered. Joints dipping at low angles, moderate to wide, occasionally slickensided (bottom 1 ft of	
30											

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Installed 2-in. diameter solid PVC pipe in completed borehole to facilitate non-destructive testing. PVC pipe was installed from the bottom of the borehole up to existing ground surface.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 1 Viaduct Bridge Rehabilitation
Location: Bath, Maine

Boring No.: BB-BROV-204
PIN: 19273.00

Driller:	MaineDOT	**Elevation (ft.)**	15.7	**Auger ID/OD:**	--
Operator:	C. Giles	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	S. Shay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	HW-300#/24 in. SS-140#/30
Date Start/Finish:	3-28-2014/4-01-2014	**Drilling Method:**	HW Drive	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131383, N393421.8 (See Plan)	**Casing ID/OD:**	SSA-2.25 in. ID; HW-4.0 in. ID	**Water Level*:**	7.3
Hammer Efficiency Factor:	0.867	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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							NQ	Core			core), planar, fresh, tight. Rock Mass Quality=Good -CAPE ELIZABETH FORMATION- Recovery=93% R1 Core Times (min:sec): 26.3-27.3' (5:00); 27.3-28.3' (6:00); 28.3-29.3' (6:00); 29.3-30.3' (5:00); 30.3-31.3' (5:00)	
	R2	55/54	31.3 - 35.9	RQD = 58%							Dark gray, fine to medium grained, micaceous SCHIST. Foliation generally trending at low angles. Hard to moderately hard, slightly weathered. Joints dipping at low angles, moderate to wide, planar, fresh, tight. Middle 1.5 ft of core run moderately weathered with soft zones, occasional pegmatite intrusions, extremely fractured, may have recovered portion of R1 as core barrel jammed at 35.9 ft. Rock Mass Quality=Fair -CAPE ELIZABETH FORMATION- Recovery=98% R2 Core Times (min:sec): 31.3-32.3' (4:00); 32.3-33.3' (5:00); 33.3-34.3' (4:00); 34.3-35.3' (5:00); 35.3-35.9' (6:00)	
35	R3	60/56	35.9 - 40.9	RQD = 93%								
40									-25.20		Dark gray fine-grained SCHIST. Foliation trending at low angles with occasional drill breaks along foliation planes. Very hard to hard, fresh. Joints not apparent in R3 and assumed wide to very wide. Occasional quartzic intrusions up to 0.8 ft wide. Minimal water loss R1-R3. Rock Mass Quality=Excellent -CAPE ELIZABETH FORMATION- Recovery=93% R3 Core Times (min:sec): 35.9-36.9' (5:00); 36.9-37.9' (5:00); 37.9-38.9' (6:00); 38.9-39.9' (5:00); 39.9-40.9' (5:00)	
45											40.9 **Bottom of Exploration at 40.9 feet below ground surface.**	
50												
55												
60												

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 1 Viaduct Bridge Rehabilitation
Location: Bath, Maine

Boring No.: BB-BROV-205
PIN: 19273.00

Driller:	MaineDOT	**Elevation (ft.)**	8.6	**Auger ID/OD:**	--
Operator:	C. Giles	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	S. Shay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	HW-300#/24 in. SS-140#/30
Date Start/Finish:	4-07-2014/4-08-2014	**Drilling Method:**	HW Drive	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131830, N393542.7 (See Plan)	**Casing ID/OD:**	SSA-2.25 in. ID; HW-4.0 in. ID	**Water Level*:**	2.5
Hammer Efficiency Factor:	0.867	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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	1D	24/8	0.0 - 2.0	1/2/1/2	3	4	SSA			Brown, very loose, fine to coarse SAND -FILL-(SP)	
								6.60		2.0	
										Note: Viewed color/stratum change with hand dig to 3.0 ft.	
							HW				
5	2D	24/24	5.0 - 7.0	1/1/1/7	2	3	10			Black to dark olive-gray, very soft, ORGANIC SOIL, some fine to medium sand, 5% wood fibers, no odor -ORGANIC DEPOSITS-(OL/OH)	
							18	2.10		6.5	
							34			Medium stiff, mottled, silty CLAY -MARINE DEPOSIT-(CL)	
							26				
	3D	24/24	9.0 - 11.0	3/5/5/7	10	14	52			Olive-gray with weak mottling, stiff, silty CLAY -MARINE DEPOSIT-(CL)	
10							65				
							92				
							96				
							88				
							72				
15							51	-6.40		15.0	
							57				
							59				
							63				
	4D	24/24	19.0 - 21.0	WOH(24.0)			54			Dark gray, very soft, CLAY, little silt -MARINE DEPOSIT-(CL)	
20							35				
							39				
							32				
							25				
							25				
25							31				
							36				
							29				
							27				
30	5D	24/17	29.0 - 31.0	WOH(24.0)			48			Dark gray, very soft, CLAY, little silt -MARINE DEPOSIT-(CL)	

Remarks:

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 1 Viaduct Bridge Rehabilitation
Location: Bath, Maine

Boring No.: BB-BROV-205
PIN: 19273.00

Driller:	MaineDOT	**Elevation (ft.)**	8.6	**Auger ID/OD:**	--
Operator:	C. Giles	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	S. Shay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	HW-300#/24 in. SS-140#/30
Date Start/Finish:	4-07-2014/4-08-2014	**Drilling Method:**	HW Drive	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131830, N393542.7 (See Plan)	**Casing ID/OD:**	SSA-2.25 in. ID; HW-4.0 in. ID	**Water Level*:**	2.5
Hammer Efficiency Factor:	0.867	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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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							60				
							67				
							66				
							55				
							56				
35							71			Note: Non-standard sampling interval.	
							67				
							62				
							56				
							63				
40							65				
							68				
							73				
							128	-34.80		Note: Drill action indicates strata change at approximately 43.4 ft. 43.4	
	6D	11/9	44.5 - 45.4	22/100(5.0)			96(6.0)			Pink-gray, very dense, fine to coarse SAND, some gravel, grading to severely to completely weathered rock with gravel	
45							NQ CORE	-36.40		-GLACIAL TILL-(SP) 45.0	
	R1	60/57	46.5 - 51.5	RQD = 25*%				-37.90		Top of Bedrock at El. -36.4 Moderately to highly weathered ROCK Note: Weathered zone to 46.5 ft. Continued to advance borehole with roller bit to 46.5 ft. Advanced casing to seat in bedrock at 46.5 ft prior to coring. 46.5	
										Gray/gray-black, fine to medium grained pegmatitic SCHIST. Irregular foliatoin joints dipping at low angles. Very hard to hard, moderately weathered, very close, rough, planar, discolored or micaceous joint surfaces, tight to open. Rock Mass Quality=Very Poor -CAPE ELIZABETH FORMATION- Recovery=95% R1 Core Times (min:sec): 46.5-47.5' (7:00); 47.5-48.5' (10:00); 48.5-49.5' (8:00); 49.5-50.5' (8:00); 50.5-51.5' (8:00)	
50											
	R2	60/60	51.5 - 56.5	RQD = 57%							
										Similar to above with intermittent planar foliation. White/gray pegmatitic SCHIST, slightly weathered, low angle joints, close. Rock Mass Quality=Fair -CAPE ELIZABETH FORMATION- Recovery=100% R2 Core Times (min:sec): 51.5-52.5' (7:00); 52.5-53.5' (6:00); 53.5-54.5' (6:00); 54.5-55.5' (7:00); 55.5-56.5' (7:00)	
55											
	R3	60/60	56.5 - 61.5	RQD = 95%							
										Gray-green, medium grained pegmatitic SCHIST. Weak, planar foliation. Very hard, fresh. Joints dipping at low angles, moderately spaced, rough, planar at fresh micaceous zones, tight. Occasional	
60											

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Installed 2-in. diameter solid PVC pipe in completed borehole to facilitate non-destructive testing. PVC pipe was installed from the bottom of the borehole up to existing ground surface.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 1 Viaduct Bridge Rehabilitation
Location: Bath, Maine

Boring No.: BB-BROV-205
PIN: 19273.00

Driller:	MaineDOT	**Elevation (ft.)**	8.6	**Auger ID/OD:**	--
Operator:	C. Giles	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	S. Shay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	HW-300#/24 in. SS-140#/30
Date Start/Finish:	4-07-2014/4-08-2014	**Drilling Method:**	HW Drive	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131830, N393542.7 (See Plan)	**Casing ID/OD:**	SSA-2.25 in. ID; HW-4.0 in. ID	**Water Level*:**	2.5
Hammer Efficiency Factor:	0.867	**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.
60							NQ Core	-52.90		rehealed high angle secondary joint present. Minimal water loss R1-R3 during coring. Rock Mass Quality=Excellent -CAPE ELIZABETH FORMATION- Recovery=100% R3 Core times (min:sec): 56.5-57.5' (6:00); 57.5-58.5' (6:00); 58.5-59.5' (6:00); 59.5-60.5' (6:00); 60.5'61.5' (7:00) 61.5 **Bottom of Exploration at 61.5 feet below ground surface.** Note: * RQD modifier for moderately weathered rock.	
65											
70											
75											
80											
85											
90											

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Installed 2-in. diameter solid PVC pipe in completed borehole to facilitate non-destructive testing. PVC pipe was installed from the bottom of the borehole up to existing ground surface.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 1 Viaduct Bridge Rehabilitation
Location: Bath, Maine

Boring No.: BB-BROV-206
PIN: 19273.00

Driller:	MaineDOT	**Elevation (ft.)**	10.4	**Auger ID/OD:**	--
Operator:	C. Giles	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	S. Shay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	HW-300#/24 in. SS-140#/30
Date Start/Finish:	4-02-2014/4-03-2014	**Drilling Method:**	HW Drive	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131890, N393541.6 (See Plan)	**Casing ID/OD:**	SSA-2.25 in. ID; HW-4.0 in. ID	**Water Level*:**	4.5
Hammer Efficiency Factor:	0.867	**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							1		Dark brown, sandy LOAM	
							SSA	9.40	-TOPSOIL-(OL/OH) 1.0	
	1D	24/16	2.0 - 4.0	5/11/6/6	17	25			Orange-brown, medium dense, fine to coarse SAND -FILL-(SP)	
								5.90	4.5	
5	2D	24/19	5.0 - 7.0	1/1/1/1	2	3			Dark gray, very soft, sandy ORGANIC SOIL, trace fine gravel, trace peat fibers, strong gasoline-like odor -FILL-(OL/OH)	
								2.40	Note: Possible wood at 8.0 ft. 8.0	
10	3D	24/24	10.0 - 12.0	5/8/8/9	16	23	14		Olive-gray to olive-brown, very stiff, silty CLAY -MARINE DEPOSIT-(CL)	
							28			
							67			
							72		Note: Attempted vane shear test at 14.0 ft, unable to push.	
	4D	24/24	14.0 - 16.0	5/5/5/7	10	14	80		Olive-gray/olive-brown, stiff, silty CLAY -MARINE DEPOSIT-(CL)	
15							33			
							40			
							45			
							48		Note: Attempted vane shear test at 19.0 ft, only advanced 5 in.	
	5D	24/24	19.0 - 21.0	2/3/4/4	7	10	56		Olive-brown grading to gray, medium stiff, silty CLAY, transitioned to soft in bottom 9 in. -MARINE DEPOSIT-(CL)	
20							104			
							82			
							74	-12.10	22.5	
							67			
	6D	24/24	24.0 - 26.0	WOH(24.0)			62		Gray, very soft, silty CLAY, highly plastic -MARINE DEPOSIT-(CL)	
25	V1A		~~24.0 - 25.0~~	~~Su=600/80 psf~~					55x110 mm vane raw torque readings:	
	V1B		25.0 - 26.0	Su=660/80 psf			96		V1A: 155/20 in-lbs V1B: 170/20 in-lbs	
							98			
							98			
							93			
30	7D	24/24	29.0 - 31.0	WOH(24.0)			115		Gray, very soft, silty CLAY, highly plastic -MARINE DEPOSIT-(CL)	

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Installed 2-in. diameter solid PVC pipe in completed borehole to facilitate non-destructive testing. PVC pipe was installed from the bottom of the borehole up to existing ground surface.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 1 Viaduct Bridge Rehabilitation
Location: Bath, Maine

Boring No.: BB-BROV-206
PIN: 19273.00

Driller:	MaineDOT	**Elevation (ft.)**	10.4	**Auger ID/OD:**	--
Operator:	C. Giles	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	S. Shay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	HW-300#/24 in. SS-140#/30
Date Start/Finish:	4-02-2014/4-03-2014	**Drilling Method:**	HW Drive	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131890, N393541.6 (See Plan)	**Casing ID/OD:**	SSA-2.25 in. ID; HW-4.0 in. ID	**Water Level*:**	4.5
Hammer Efficiency Factor:	0.867	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
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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							127				
							123				
							123				
							91				
	8D	24/24	34.0 - 36.0	WOH(24.0)			86			Gray, very soft, silty CLAY, highly plastic -MARINE DEPOSIT-(CL) 55x110 mm vane raw torque readings: V2A: 102/20 in-lbs V2B: 115/20 in-lbs	
35	V2A		34.0 - 35.0	Su=390/80 psf							
	V2B		35.0 - 36.0	Su=445/80 psf			99				
							111				
							111				
							87				
	9D	24/22	39.0 - 41.0	WOH(12.0)/8/21			87			Gray, very soft, silty CLAY, highly plastic -MARINE DEPOSIT-(CL)	
40								-29.60		40.0	
								-30.60		Medium dense, sandy CLAY, with micaceous weathered rock 41.0	
	R1	54/54	41.4 - 45.9	RQD = 74%			NQ Core			Top of Bedrock at El. -30.6 Gray/white/black, fine to medium grained SCHIST. Planar and/or undulating foliation. Very hard to hard, slightly weathered. Primary joints moderately dipping, very close to close with occasional extremely fractured zones, rough, planar, fresh except occasional discolored and silt coated, tight to open, core barrel jammed at 45.9 ft. Rock Mass Quality=Fair -CAPE ELIZABETH FORMATION- Recovery=100% R1 Core Times (min:sec): 41.4-42.4' (4:00); 42.4-43.4' (4:00); 43.4-44.4' (8:00); 44.4-45.4' (6:00); 45.4-45.9' (5:00)	
45	R2	60/60	45.9 - 50.9	RQD = 53%						Gray/gray-green, fine to medium grained SCHIST. Poorly defined foliation. Very hard to hard, slightly weathered. Primary joints moderately dipping, close, rough, planar, fresh, tight. Secondary joints dipping at high angles, similar to primary except undulating. Secondary chlorite mineralization. Rock Mass Quality=Fair -CAPE ELIZABETH FORMATION- Recovery=100% R2 Core Times (min:sec): 45.9-46.9' (4:00); 46.9-47.9' (4:00); 47.9-48.9' (4:00); 48.9-49.9' (6:00); 49.9-50.9' (6:00)	
50	R3	60/60	50.9 - 55.9	RQD = 82%						Gray/white/black, fine to medium grained SCHIST. Well defined planar foliation dipping at moderate angles. Hard, slightly weathered. Joints dipping at low angles, moderately spaced, rough, planar, moderately weathered surfaces somtimes silt coated, open. Secondary joints at moderate angles with micaceous surfaces. Minor secondary mineralization as possible garnet (top 2 in.) and chlorite. Minimal water loss R1-R3. Rock Mass Quality=Good -CAPE ELIZABETH FORMATION- Recovery=93% R3 Core Times (min:sec): 50.9-51.9' (4:00); 51.9-52.9' (5:00); 52.9-53.9' (5:00); 53.9-54.9' (5:00); 54.9-55.9' (5:00)	
55											
60								-48.90			

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Installed 2-in. diameter solid PVC pipe in completed borehole to facilitate non-destructive testing. PVC pipe was installed from the bottom of the borehole up to existing ground surface.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 1 Viaduct Bridge Rehabilitation
Location: Bath, Maine

Boring No.: BB-BROV-206
PIN: 19273.00

Driller:	MaineDOT	**Elevation (ft.)**	10.4	**Auger ID/OD:**	--
Operator:	C. Giles	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	S. Shay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	HW-300#/24 in. SS-140#/30
Date Start/Finish:	4-02-2014/4-03-2014	**Drilling Method:**	HW Drive	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131890, N393541.6 (See Plan)	**Casing ID/OD:**	SSA-2.25 in. ID; HW-4.0 in. ID	**Water Level*:**	4.5
Hammer Efficiency Factor:	0.867	**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.
60										59.3 **Bottom of Exploration at 59.3 feet below ground surface.**	
65											
70											
75											
80											
85											
90											

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Installed 2-in. diameter solid PVC pipe in completed borehole to facilitate non-destructive testing. PVC pipe was installed from the bottom of the borehole up to existing ground surface.

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Preliminary Phase
Test Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: U.S. Route 1 Viaduct Bridge Replacement
Location: Bath, Maine

Boring No.: BB-BROV-101
PIN: 19273.00

Driller:	Maine Test Borings	**Elevation (ft.)**	27.4	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile Drill B-53 Truck	**Hammer Wt./Fall:**	NW-300#/24 in.-SS 140#/30 in.
Date Start/Finish:	3-26-13/3-26-13	**Drilling Method:**	NW	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131000, N393314 (See Plan)	**Casing ID/OD:**	NW-3.0 in. ID	**Water Level*:**	11.8
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)
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							1		-BITUMINOUS CONCRETE-	
	1D	24/12	1.0 - 3.0	6/7/6/25	13	13	SSA	26.4	1.0 Brown, dry, medium dense, medium to fine SAND, little coarse sand, trace silt, little gravel -FILL-(SW) Note: Probable cobble from approximately 3.0 to 4.3 ft.	
	2D	24/22	4.3 - 6.3	4/6/6/5	12	12		23.1	4.3 Brown-gray, dry to moist, stiff mottled SILT, trace fine sand -MARINE DEPOSIT-(ML)	
5	3D	24/22	6.3 - 8.3	6/6/6/5	12	12		21.1	6.3 Brown-gray, moist, stiff, silty CLAY -MARINE DEPOSIT-(CL)	
	4D V1A	24/22	8.3 - 10.3	1/2/3/3 Su>1940 psf	5	5	4	18.1	Brown, moist, medium stiff, silty CLAY with sand layers 55x110 mm vane raw torque reading: V1A: >500 in/lbs	
10							13		9.3 Brown, clayey, medium to fine SAND, little coarse sand, little gravel -GLACIAL TILL-(SC/SM)	
							42	15.4	12.0	
	R1	59/59	12.3 - 17.2	RQD = 59%			26(2.0")	15.1	-WEATHERED BEDROCK- 12.3 Top of Bedrock at El. 15.1 Gray, medium to fine-grained chlorite, biotite, SCHIST with frequent quartz and pegmatitic intrusions. Moderately hard, slightly weathered with occasional moderately weathered joints, low angle to moderately dipping, one steep secondary joint, very close to close, tight to open Rock Mass Quality=Fair -CAPE ELIZABETH FORMATION- Recovery=100% R1 Core Times (min:sec): 12.3-13.3' (1:45); 13.3-14.3' (2:33); 14.3-15.3' (1:53); 15.3-16.3' (2:26); 16.3-17.2' (3:04)	GTX#300397 qp=2,321 psi
15								10.2	17.2 **Bottom of Exploration at 17.2 feet below ground surface.**	
20										
25										
30										

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. qp=unconfined compressive strength; psf=pounds per square foot; psi=pounds per square inch.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: U.S. Route 1 Viaduct Bridge Replacement
Location: Bath, Maine

Boring No.: BB-BROV-102
PIN: 19273.00

Driller:	Maine Test Borings	**Elevation (ft.)**	17.2	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile Drill B-53 Truck	**Hammer Wt./Fall:**	HW-300#/24 in.-SS 140#/30 in.
Date Start/Finish:	3-26-13/3-26-13	**Drilling Method:**	HW	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131318, N393408 (See Plan)	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	14.2
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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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	24/6	0.5 - 2.5	18/16/7/6	23	23	SSA	16.8	-BITUMINOUS CONCRETE- (0.4)	
									Brown, dry, medium dense, fine to medium SAND, some coarse sand, trace silt, little gravel -FILL-(SW) (2.5)	
	2D	24/14	2.5 - 4.5	5/10/11/16	21	21		14.7	Brown-gray, dry, very stiff, mottled clayey SILT, trace fine sand -MARINE DEPOSIT-(ML)	
	3D	24/22	4.5 - 6.5	9/10/10/10	20	20			Brown-gray, dry, very stiff, mottled clayey SILT, trace fine sand -MARINE DEPOSIT-(ML)	
5										
	4D	24/20	6.5 - 8.5	13/14/20/21	34	34			Brown-gray, dry to moist, hard, clayey SILT, trace fine sand -MARINE DEPOSIT-(ML)	
	5D	20/2	8.5 - 10.2	9/16/28/40(3.0")	44	44			Note: Recovered 2-in. brown, dry, dense, weathered rock.	
10	R1	62/62	10.7 - 15.9	RQD = 85%				7.2	(10.0)	
								6.5	-WEATHERED BEDROCK- (10.7)	
									Top of Bedrock at El. 6.5 Tan-white-gray, medium to fine-grained GNEISS. Moderately hard, slightly weathered, joints low angle to moderately dipping, few steep secondary joints, very close to moderately close, tight to open, occasional silt coatings on joint surfaces. Rock Mass Quality=Good -CAPE ELIZABETH FORMATION- Recovery=100% R1 Core Times (min:sec): 10.7-11.7' (2:08); 11.7-12.7' (1:08); 12.7-13.7' (1:19); 13.7-14.7' (1:31); 14.7-15.9' (1:00)	GTX#300397 qp=1,522 psi
15								1.3	(15.9)	
20										
25										
30										

Bottom of Exploration at 15.9 feet below ground surface.

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. qp=unconfined compressive strength; psf=pounds per square foot; psi=pounds per square inch.

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.

Boring No.: BB-BROV-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: U.S. Route 1 Viaduct Bridge Replacement
Location: Bath, Maine

Boring No.: BB-BROV-103
PIN: 19273.00

Driller:	Maine Test Borings	**Elevation (ft.)**	7.8	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile Drill B-53 Truck	**Hammer Wt./Fall:**	HW-300#/24 in.-SS 140#/30 in.
Date Start/Finish:	3-27-13/3-27-13	**Drilling Method:**	HW	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131808, N393523 (See Plan)	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	14.2
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	24/12	0.5 - 2.5	6/20/30/6	50	50	SSA	7.7	-BITUMINOUS CONCRETE- 0.1	
									Brown, dry, dense, medium to fine SAND, little coarse sand, little gravel, trace silt -FILL-(SW)	
	2D	24/10	2.5 - 4.5	3/2/2/1	4	4		4.8	3.0	
									Dark brown, moist, soft, SILT, little organics (woody/fibrous), trace fine sand -ORGANIC DEPOSIT-(ML)	
	3D	24/24	4.5 - 6.5	1/1/1/1	2	2			Dark brown, moist, soft, SILT with organics, trace fine sand -ORGANIC DEPOSIT-(ML)	
5										
	4D	24/24	6.5 - 8.5	2/2/3/5	5	5			Dark brown, moist, medium stiff, SILT with organics (fibrous), trace fine sand -ORGANIC DEPOSIT-(ML)	
	5D	24/24	8.5 - 10.5	3/5/7/8	12	12		-0.7	8.5	
									Brown-gray, moist, stiff, mottled clayey SILT with organics (fibrous) -MARINE DEPOSIT-(ML)	
10	6D	24/24	10.5 - 12.5	13/12/14/12	26	26	36		Brown-gray, moist, very stiff, mottled clayey SILT to silty CLAY -MARINE DEPOSIT-(ML/CL)	
							35			
	7D	24/12	12.5 - 14.5	4/5/6/7	13	13	34		Note: Attempted field vane shear test at 12.0 ft, could not push. Brown-gray, moist to wet, stiff, mottled clayey SILT to silty CLAY -MARINE DEPOSIT-(ML/CL)	
							43			
							49			
15	8D	24/24	15.0 - 17.0	3/4/5/4	9	9	45		Brown-gray, moist to wet, stiff, mottled clayey SILT to silty CLAY -MARINE DEPOSIT-(ML/CL)	
							43			
	9D	24/18	17.0 - 19.0	1/2/2/2	4	4	40	-9.2	17.0 Gray, moist to wet, medium stiff, silty CLAY (CL) -MARINE DEPOSIT-(CL)	
							38			
	10D	24/24	19.0 - 21.0	WOH/WOH/WOH/ WOH			40		Gray, wet, medium stiff, silty CLAY -MARINE DEPOSIT-(CL)	
20	V1A		19.6 - 20.0	Su=855/20 psf			51		55x110 mm vane raw torque readings: V1A: 220/6 in-lbs	
	U1		21.0 - 23.0				45			C#IP-1 WC=41.9% LL=41 PL=19, PI=22
							39			
	V2A		23.6 - 24.0	Su=700/95 psf			40		55x110 mm vane raw torque readings: V2A: 178/26 in-lbs	
	V2B		24.6 - 25.0	Su=640/115 psf			34		V2B: 165/30 in-lbs	
25	11D	24/24	25.0 - 27.0	WOR/WOR/WOR/ WOR			44		Dark gray, wet, medium stiff, silty CLAY -MARINE DEPOSIT-(CL)	
							40			
							27			
	V3A		28.6 - 29.0	Su=580/80 psf			19		55x110 mm vane raw torque readings: V3A: 150/21 in-lbs	
30	V3B		29.6 - 30.0	Su=580/80 psf			13			

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. psf = pounds per square foot.

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: U.S. Route 1 Viaduct Bridge Replacement
Location: Bath, Maine

Boring No.: BB-BROV-103
PIN: 19273.00

Driller:	Maine Test Borings	**Elevation (ft.)**	7.8	**Auger ID/OD:**	--
Operator:	M. Porter	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	M. Snow	**Rig Type:**	Mobile Drill B-53 Truck	**Hammer Wt./Fall:**	HW-300#/24 in.-SS 140#/30 in.
Date Start/Finish:	3-27-13/3-27-13	**Drilling Method:**	HW	**Core Barrel:**	NQ-1.875 in.
Boring Location:	E1131808, N393523 (See Plan)	**Casing ID/OD:**	HW-4.0 in. ID	**Water Level*:**	14.2
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.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
30	U2		30.0 - 32.0				27		V3B: 150/20 in-lbs Gray, wet, medium stiff, silty CLAY -MARINE DEPOSIT-(CL)	C#IP-3 WC=47.7% LL=38 PL=19, PI=19
							19			
	V4A		32.6 - 33.0	Su=600/40 psf			19		55x110 mm vane raw torque readings: V4A: 154/12 in-lbs V4B: 170/32 in-lbs	
	V4B		33.6 - 34.0	Su=660/115 psf			21			
							24			
35	12D	24/24	35.0 - 37.0	WOR/WOR/WOR/ WOR			23		Dark gray, wet, medium stiff, silty CLAY -MARINE DEPOSIT-(CL)	
							25			
							22			
	V5A		38.6 - 39.0	Su=505/135 psf			25		55x110 mm vane raw torque readings: V5A: 130/34 in-lbs V5B: 127/11 in-lbs	
	V5B		39.6 - 40.0	Su=485/40 psf			27			
40							30			
	U3		41.0 - 43.0				30		Gray, wet, soft to medium stiff, silty CLAY -MARINE DEPOSIT-(CL)	C#IP-2 WC=40.6% LL=37 PL=19, PI=18
							25			
	V6A		43.6 - 44.0	Su=525/115 psf			30		55x110 mm vane raw torque readings: V6A: 137/31 in-lbs V6B: 150/33 in-lbs	
	V6B		44.6 - 45.0	Su=580/135 psf			35			
45							41			
							41			
							51	-39.0	46.8	
							70		Note: Sand and gravel observed in wash water at approximately 46.8 ft. -PROBABLE GLACIAL TILL-	
	13D	0/0	49.0 - 49.0	50(0")				-41.2	49.0	
50	R1	60/60	49.0 - 54.0	RQD = 52%					Top of Bedrock at El.-41.2 Gray, black and white, coarse to medium-grained pegmatitic SCHIST. Moderately hard, fresh to slightly weathered, joints low angle to moderately dipping, very close to close, tight to open, occasional weathered joint surfaces and silt coatings. Rock Mass Quality=Fair Recovery=100% R1 Core Times (min:sec): 49.0-50.0' (2:25); 50.0-51.0' (3:25); 51.0-52.0' (1:58); 52.0-53.0' (2:00); 53.0-54.0' (2:08)	
								-46.2		
55									54.0 **Bottom of Exploration at 54.0 feet below ground surface.**	
60										

Remarks:

1. As-drilled coordinates of test borings determined by MaineDOT and provided in NAD83 (96) ME2000 West Zone coordinate system.
2. Hammer consisted of rope and cathead and safety hammer.
3. psf = pounds per square foot.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

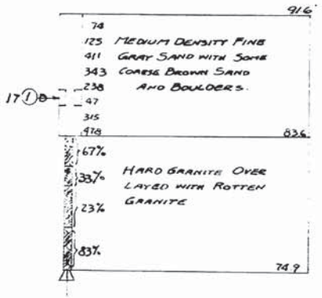
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Historic
Test Boring Logs

F. A. P. No.	STATE	PROJECT NUMBER	SHEET NO	TOTAL SHEETS
	MAINE	U-026-1(8)	10	148

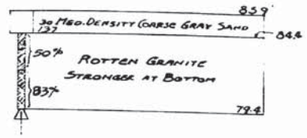
BORING NO. 1
STA 385+95 CENTER ABUT. #1

Elevation



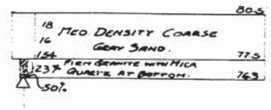
BORING NO. 2
STA. 387+15 CENTER PIER #2

Elevation



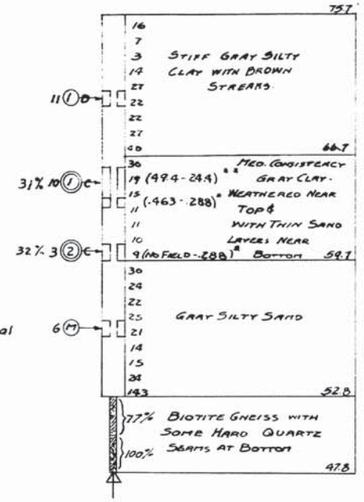
BORING NO. 3
STA. 388+37 CENTER PIER #4

Elevation



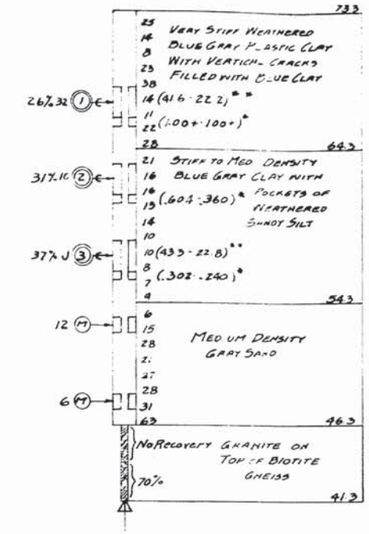
BORING NO. 4
STA 389+51 CENTER PIER #6

Elevation



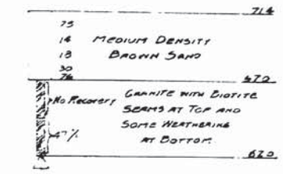
BORING NO. 5
STA 391+22 CENTER PIER #7

Elevation



BORING NO. 6
STA 392+68 CENTER PIER #8

Elevation



BORING NOTES

1. Scales as Noted on Drawings
2. Casing Size as Noted on Drawings.
3. Ground Water Table indicated thus:
4. Number of Blows of 275# Hammer falling 18 inches required to drive Extra Heavy Casing One Foot thus:
5. W, S, or V indicate respectively that the Hole was Washed, Sampled or Vaned ahead of Casing.
6. Location and designation of dry Samples taken in S.&H. sampler #1290s indicated thus:
7. Location and designation of dry samples taken in 2" O.D. 18 Ga. seamless Tubing indicated thus:
8. Location and designation of dry samples taken in 3½ O.D. 16 Ga. seamless tubing indicated thus:

LOG SHEETS — DETAIL SHEETS

40 — Graphical

(1) D — 1D

(1) C — 1C

(1) U — 1U

BORING NOTES

9. Location and designation of 3½ O.D. dry samples taken with piston sampler indicated thus:
10. Location and designation of wash samples indicated thus:
11. Unsuccessful attempts to secure dry sample indicated thus:
12. Location of field vane test indicated thus:
13. Number of blows of 275# hammer falling 15" required to drive spoon or tubing one foot indicated thus
14. Sampling spoon or seamless tubing driven by static load not exceeding ½ ton indicated thus:
15. Sampling spoon or seamless tubing driven by static weight of drill rods and 275# hammer indicated thus:

LOG SHEETS — DETAIL SHEETS

(1) P — 1P

(1) W — 1W

R — M

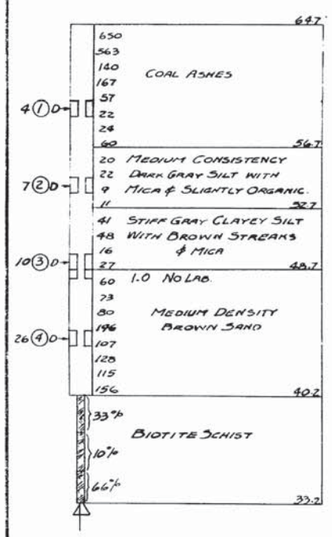
20 — GRAPHICAL

J — J

H — H

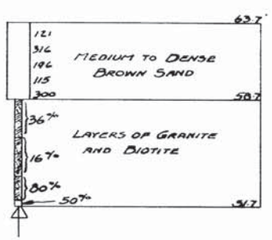
BORING NO. 7
STA 393+40 CENTER PIER #12

Elevation



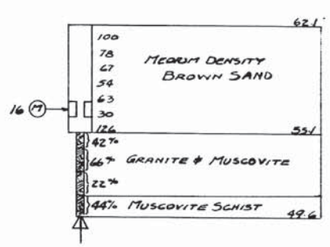
BORING NO. 8
STA 394+04 CENTER PIER #13

Elevation



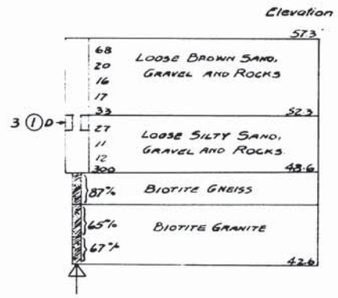
BORING NO. 9
STA 394+68 CENTER PIER #14

Elevation



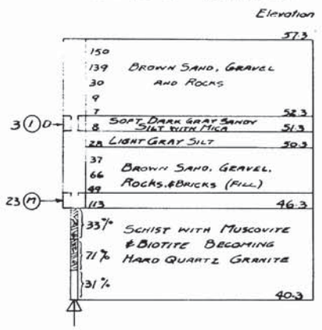
BORING NO. 10
LOCATION "O"
STA 403+12 15' RT. & RAMP

Elevation



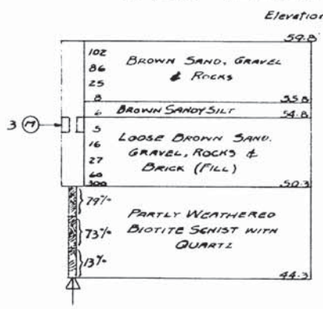
BORING NO. 11
LOCATION "M"
STA 402+59 14.5' RT. & RAMP

Elevation



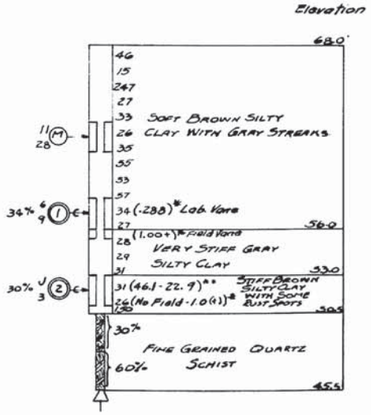
BORING NO. 12
LOCATION "K"
STA 402+01 14' RT. & RAMP

Elevation



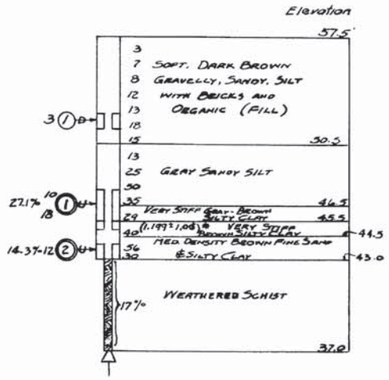
BORING NO. 14
STA 392+13 CENTER PIER #10

Elevation



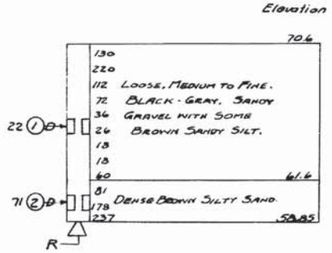
BORING NO. 15
LOCATION "E"
STA 400+34 47' RT. & TRESTLE

Elevation



BORING NO. 23
STA. 401+00 15' RT. & TRESTLE

Elevation



BORING NOTES

16. Where no resistance is specified spoon or tubing was driven by the static weight of drill rods.
17. Natural water contents, given as percent of dry weight are indicated thus:
18. Bottom of boring indicated thus:
19. Refusal of drill rods or casing indicated thus:
20. % recovery of rock core by size E diamond bit thus:

LOG SHEETS — DETAIL SHEETS

31%-20 — Graphical

R — R

71% — 71%

NOTES:

* Field Vane Shear Strength Followed by Laboratory Vane Shear Strength in Tons/sq. ft.

** Liquid Limit Followed By Plastic Limit (Per Cent of Water Content)

SCALE 1" = 5 FEET.

DESIGN - Garrett
TRACE - JONES
CHECK - Langford

WEST APPROACH

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING
AND
WEST APPROACH
BETWEEN
BATH & WOOLWICH
SAGADAHOC COUNTY, MAINE

BORINGS

SHEET 10 OF 148 AUGUSTA, MAINE JAN. 1957

BORING NOTES

		LOG SHEETS	DETAIL SHEETS
1.	Scales as noted on drawings.		
2.	Casing size as noted on drawings.		
3.	Ground water table indicated thus:		
4.	Number of blows of 275# hammer falling 18 inches required to drive extra heavy casing one foot thus:	40	(graphical)
5.	W, S, or V indicate respectively that the hole was washed, sampled, or vaned ahead of casing.		
6.	Location and designation of "dry" samples taken in S&H sampler #1290s indicated thus:	I D	ID
7.	Location and designation of "dry" samples taken in 2" O.D. 18 ga. seamless tubing indicated thus:	I C	IC
8.	Location and designation of "dry" samples taken in 3½" O.D. 16 ga. seamless tubing indicated thus:	I U	IU
9.	Location and designation of 3½" O.D. "dry" samples taken with piston sampler indicated thus:	I P	IP
10.	Location and designation of wash samples indicated thus:	I W	IW
11.	Unsuccessful attempts to secure dry sample indicated thus:	M	M
12.	Location of field vane test indicated thus:		
13.	Number of blows of 275# hammer falling 15" required to drive spoon or tubing one foot indicated thus:	20	(graphical)
14.	Sampling spoon or seamless tubing driven by static load not exceeding ½ ton indicated thus:	J	J
15.	Sampling spoon or seamless tubing driven by static weight of drill rods and 275# hammer indicated thus:	H	H
16.	Where no resistance is specified spoon or tubing was driven by the static weight of drill rods.		
17.	Natural water contents, given as percent of dry weight are indicated thus:	31% 20	graphical
18.	Bottom of boring indicated thus:		
19.	Refusal of drill rods or casing indicated thus:	R	R
20.	% recovery of rock core by size "E" diamond bit thus:	71%	71%

SHEAR AND WATER CONTENT NOTES

Shear Notes:

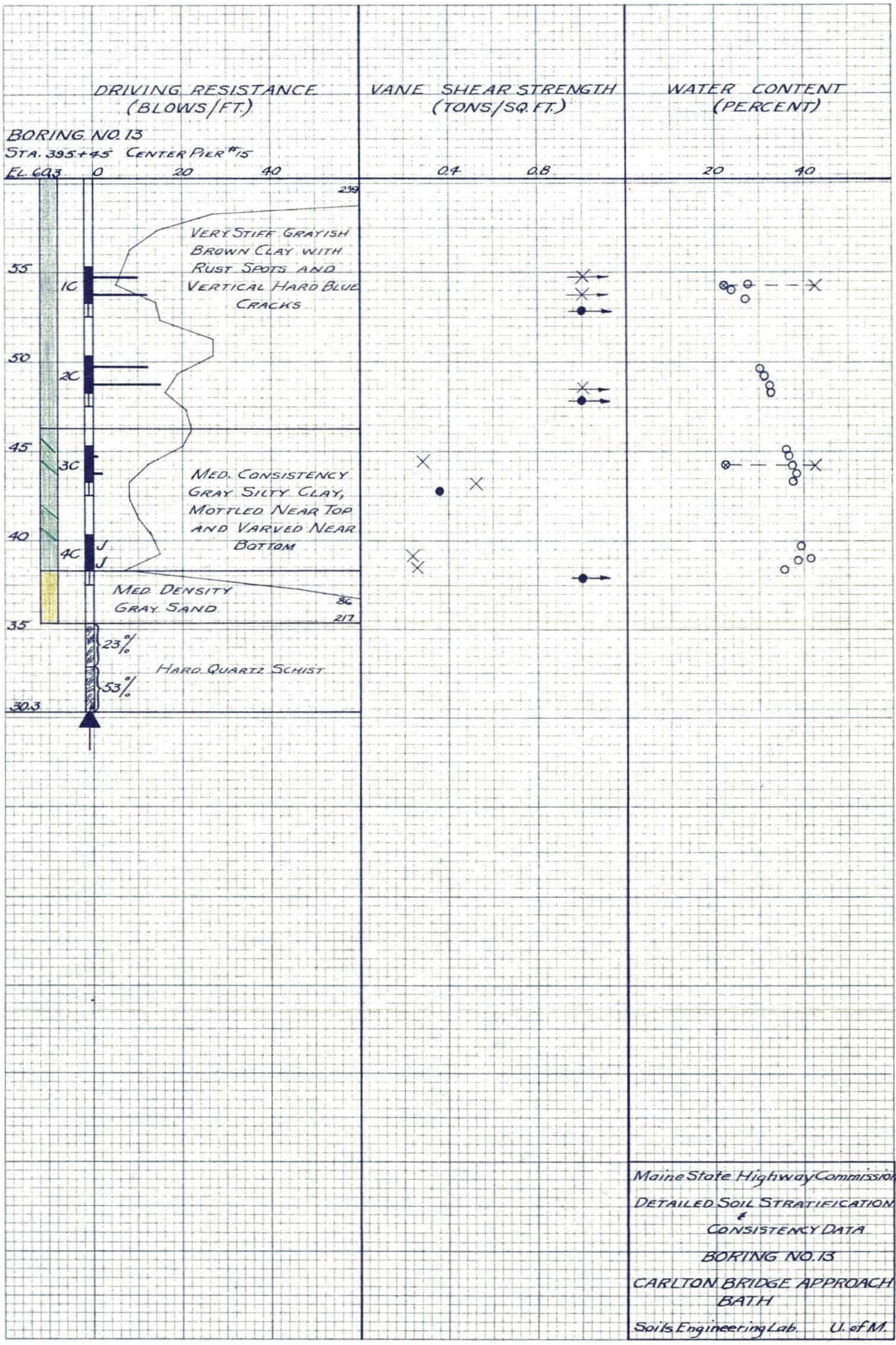
1. Field vane shear strengths indicated thus: ●
2. Laboratory vane shear strengths indicated thus: x
3. One half unconfined compressive strengths indicated thus: o
4. Strengths beyond range of plot indicated at right edge of plot by numerical values and symbols thus: 1.62●
5. Field vane shear strengths in excess of capacity of equipment indicated thus: —●→
6. Laboratory vane shear strengths in excess of capacity of equipment (1.0 T/sf) indicated thus: —x→
7. Field vane shear strengths in excess of capacity of equipment and beyond range of plot indicated at right edge of plot thus: 1.50(+)●
8. Laboratory vane shear strength in excess of capacity of equipment (1.0T/sf) and beyond range of plot indicated at right edge of plot thus: 1.00 (+)x

Water Content Notes:

1. Natural water contents, given as percent of dry weight, are indicated thus: o
2. Plastic and liquid limits are indicated thus: ⊗— — —x
3. Ignition losses are given as percent of dry weight.

NO. 3400-10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

EUGENE DIETZGEN CO.
MADE IN U.S.A.



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10 X 10 PER INCH

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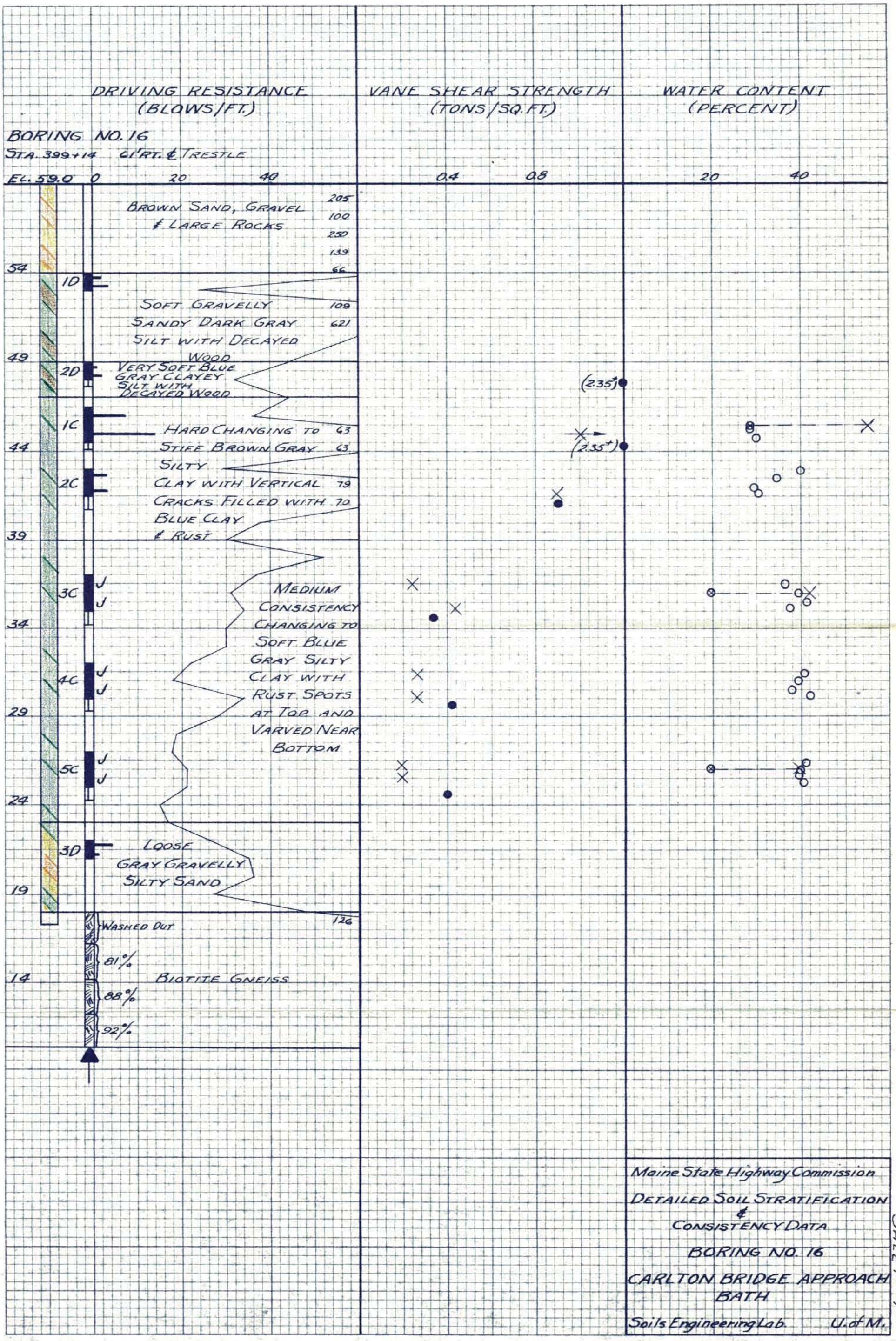
MADE IN U. S. A.



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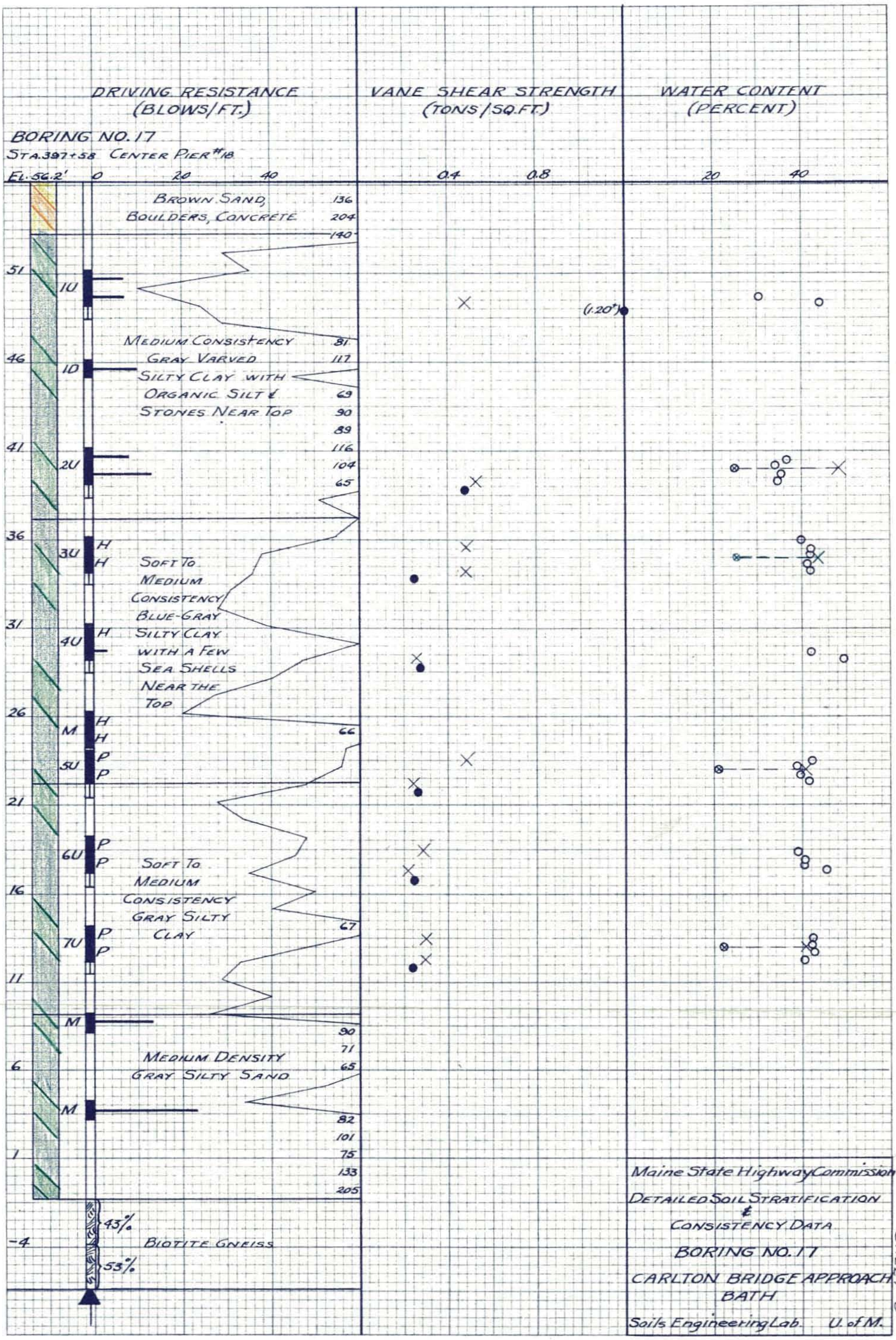
10 X 10 PER INCH

EUGENE DIETZGEN CO.
MADE IN U. S. A.



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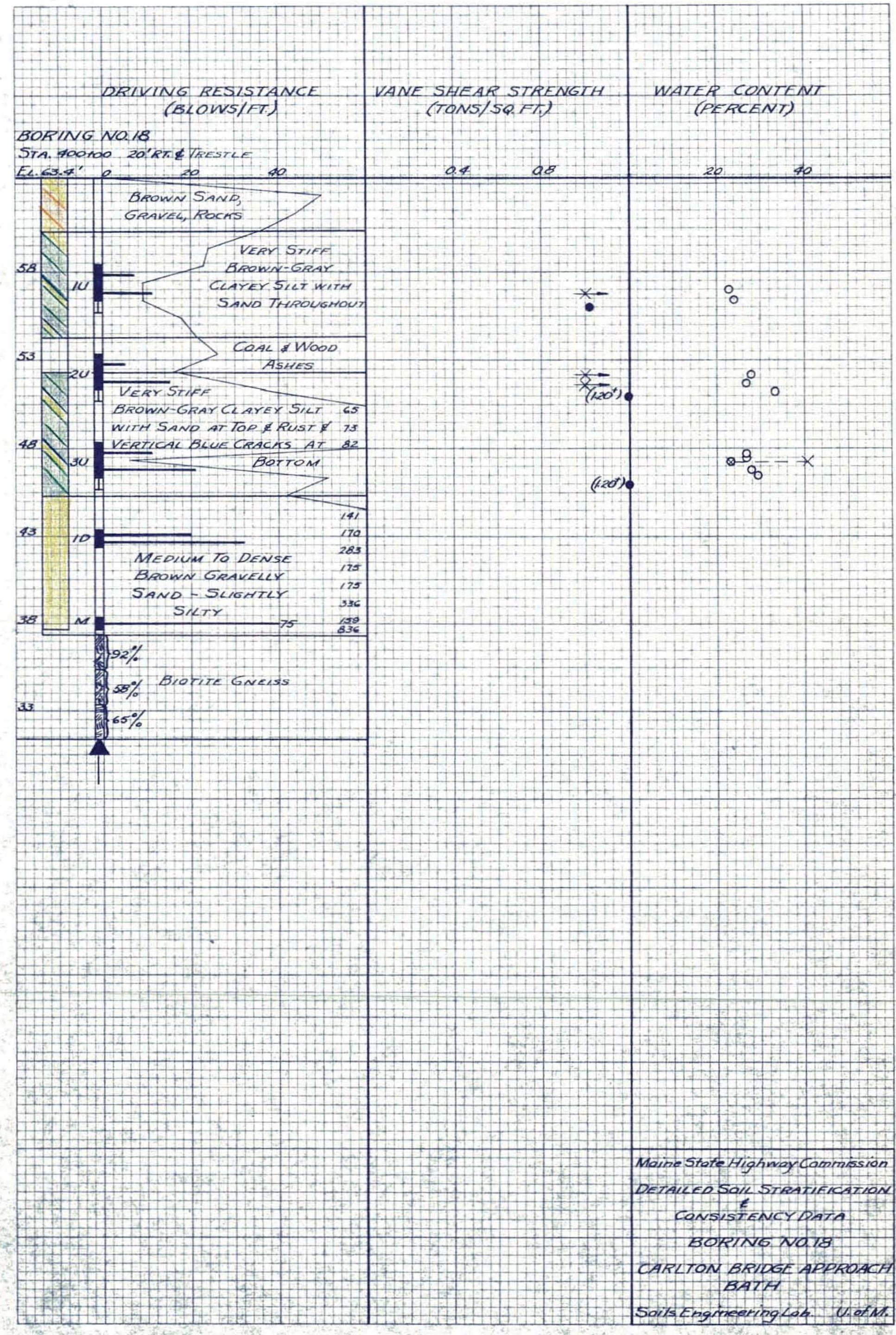

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10 X 10 PER INCH

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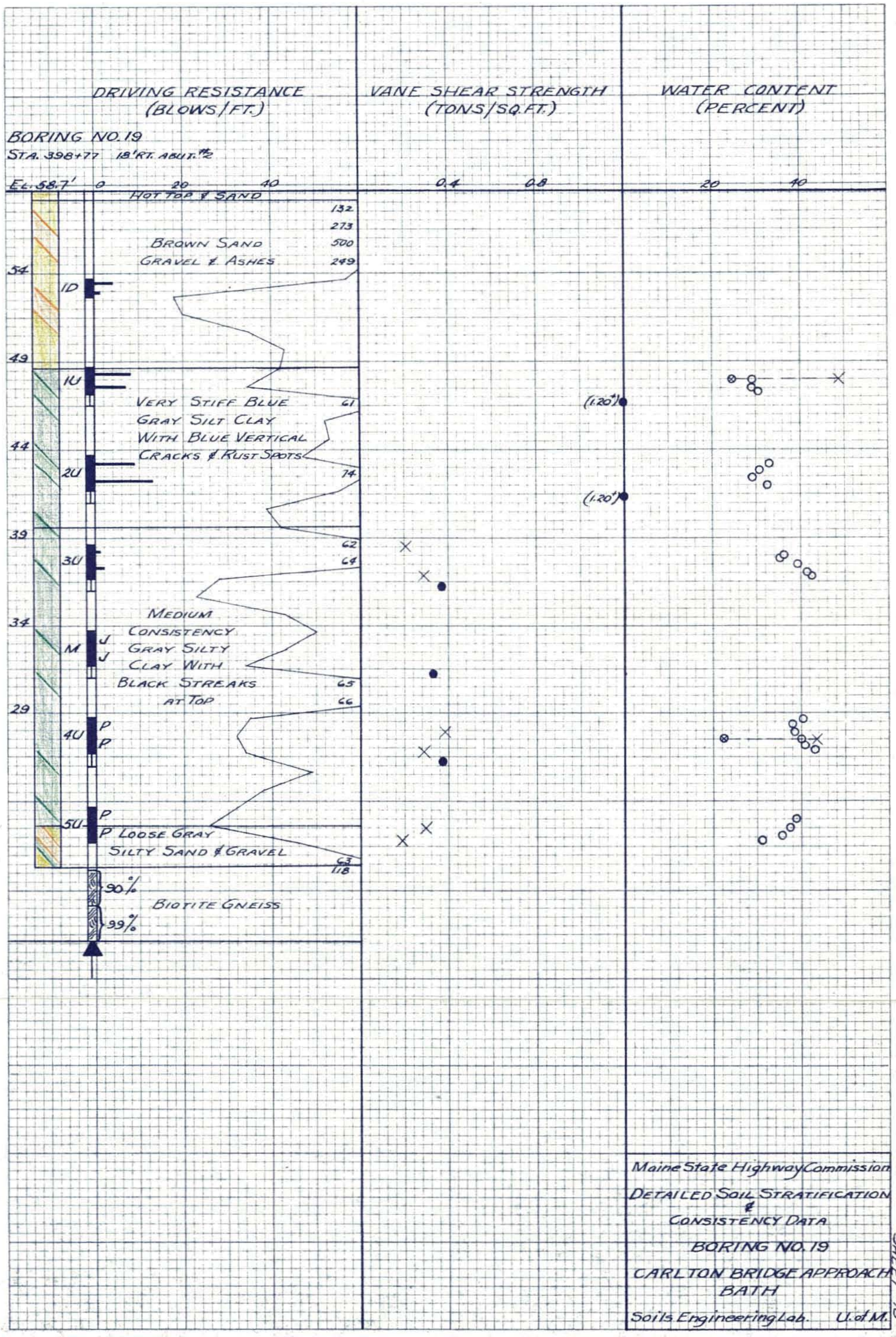
MADE IN U.S.A.



Maine State Highway Commission

DETAILED SOIL STRATIFICATION & CONSISTENCY DATA

BORING NO. 19

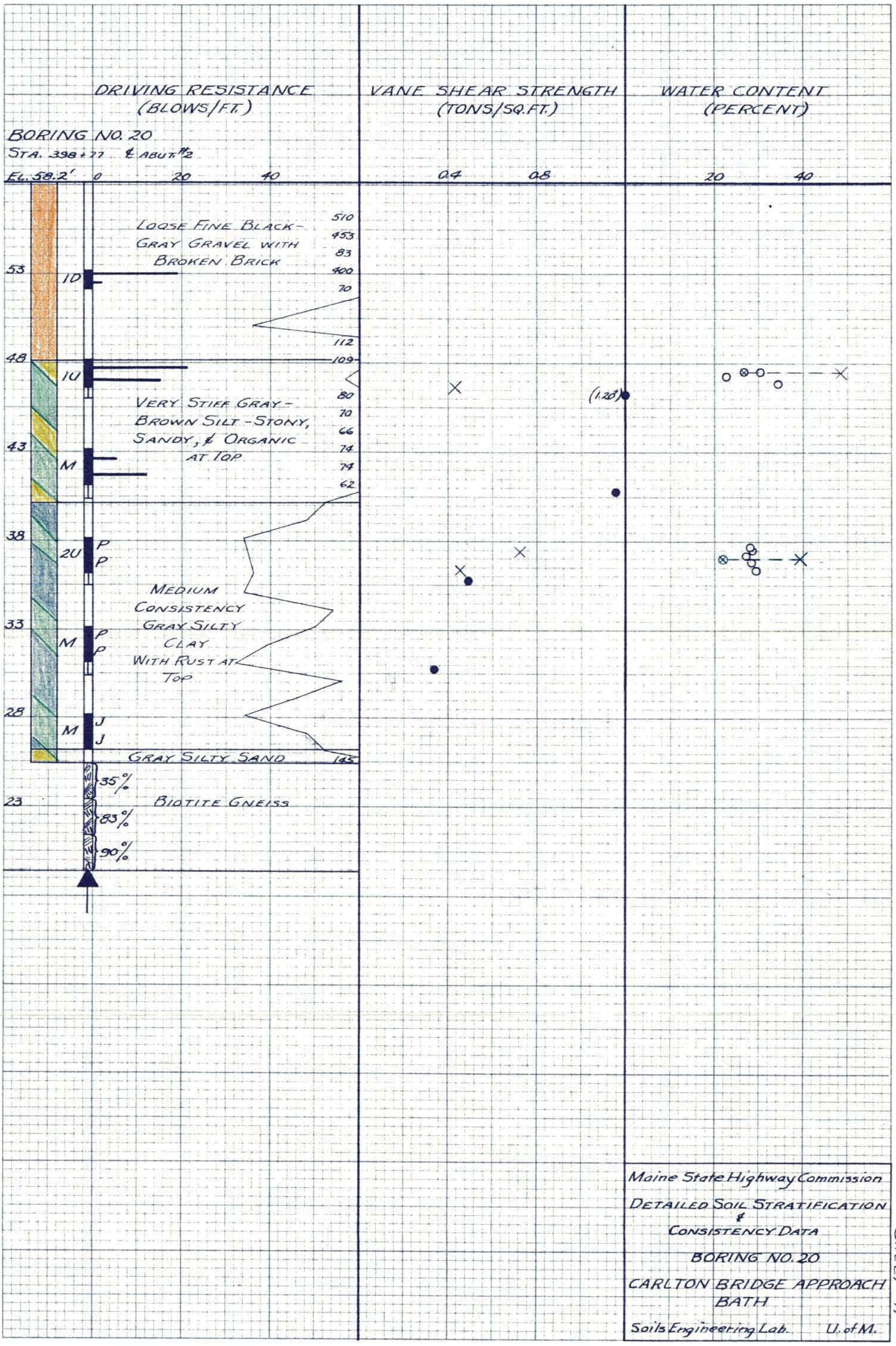
CARLTON BRIDGE APPROACH
BATH

Soils Engineering Lab. U. of M.

SHEET #10

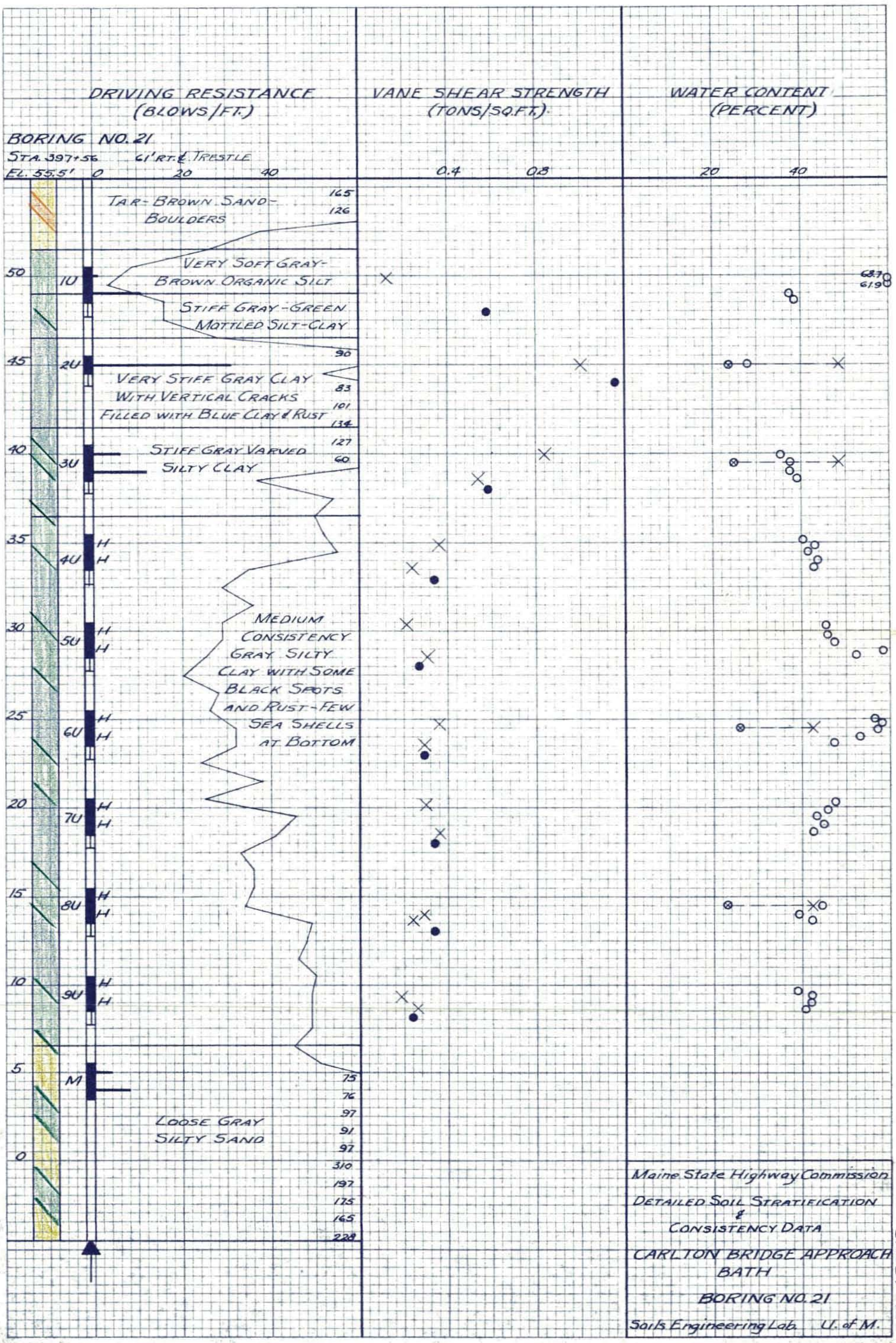
10. 3400-10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

EUGENE DIETZGEN CO.
MADE IN U. S. A.



NO. 3400-10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

EUGENE DIETZGEN CO.
MADE IN U.S.A.



Maine State Highway Commission

DETAILED SOIL STRATIFICATION & CONSISTENCY DATA

CARLTON BRIDGE APPROACH
BATH

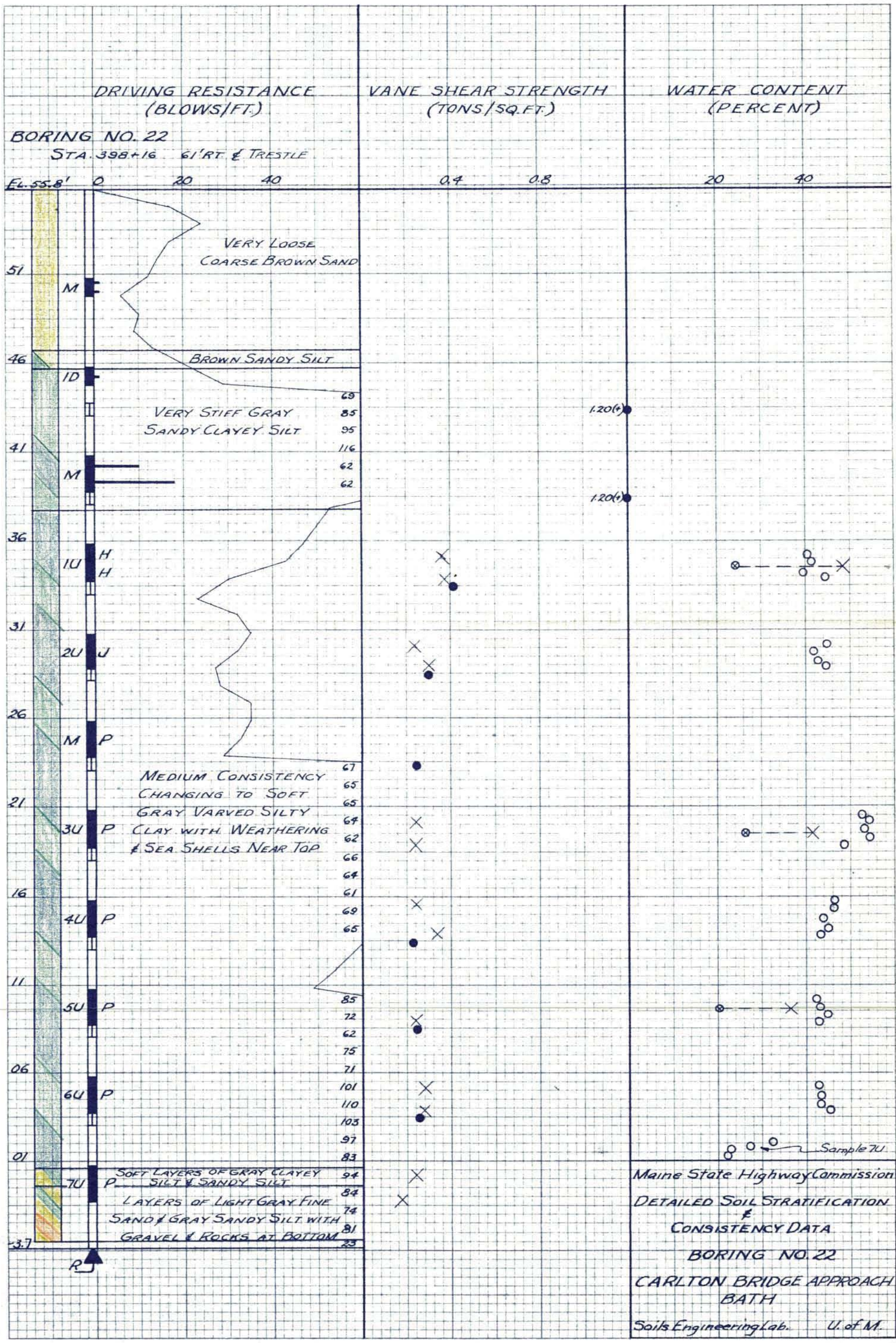
BORING NO. 21

Soils Engineering Lab. U. of M.

SHEET #12

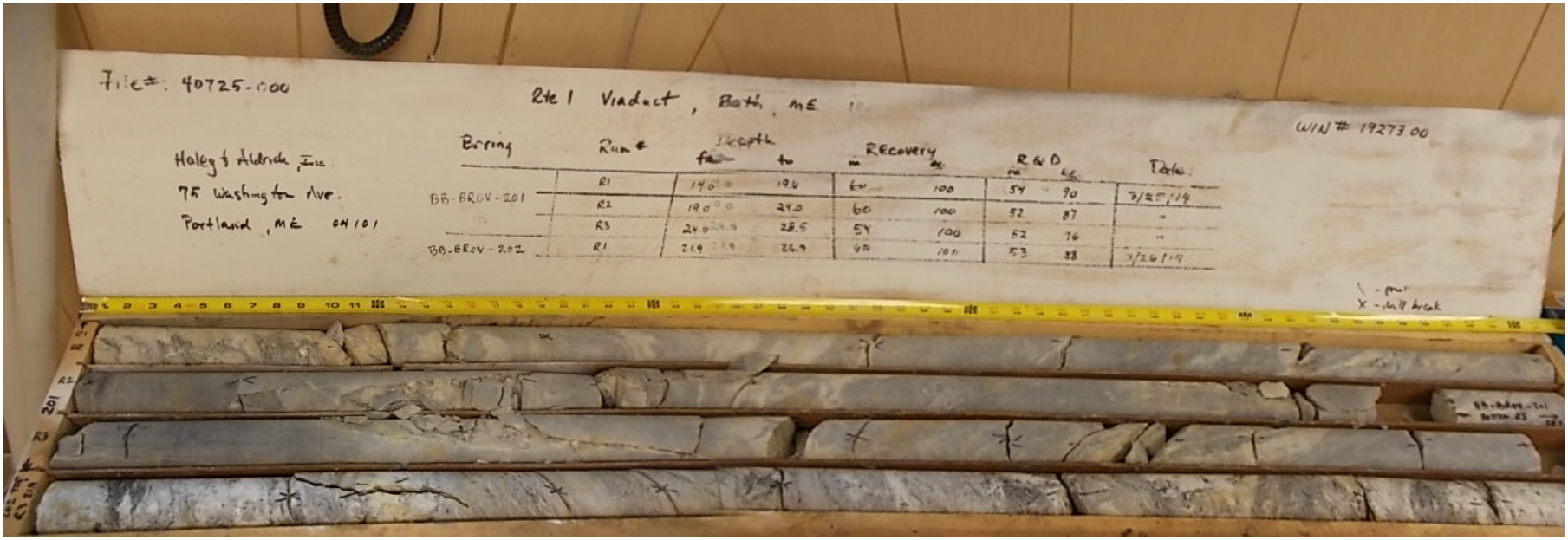
10. 3400-10 DIETZGEN GRAPH PAPER
10 X 10 PER INCH

EUGENE DIETZGEN CO.
MADE IN U.S.A.

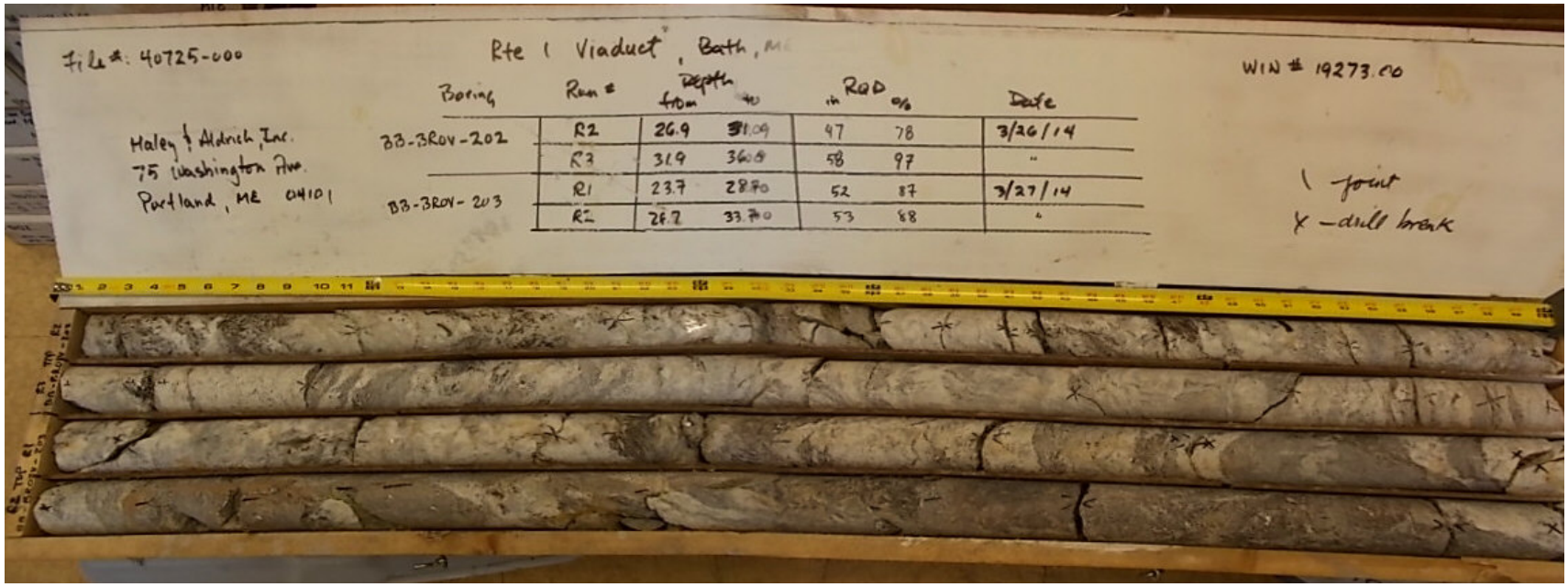


APPENDIX B

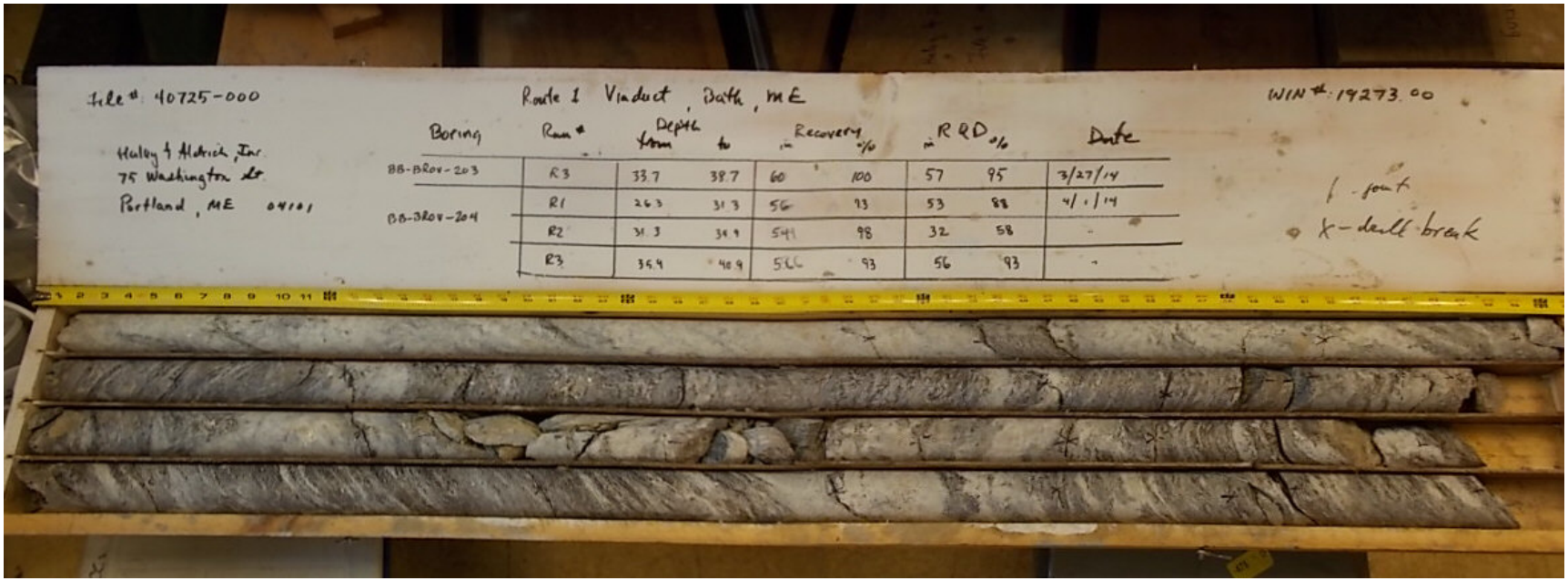
Rock Core Photographs



Photograph No. 1: Collected rock core samples for test borings BB-BROV-201 and BB-BROV-202. Top: BB-BROV-201, Core No. R1 14 ft left to 19 ft right.
Middle/Top: BB-BROV-201, Core No. R2 19 ft left to 24 ft right. Middle/Bottom: BB-BROV-201, Core No. R3 24 ft left to 28.5 ft right. Bottom: BB-BROV-202, Core No. R1 21.9 ft left to 26.9 ft right.



Photograph No. 2: Collected rock core samples for test borings BB-BROV-202 and BB-BROV-203 Top: BB-BROV-202, Core No. R2 26.9 ft left to 31.9 ft right.
Middle/Top: BB-BROV-202, Core No. R3 31.9 ft left to 36.9 ft right. Middle/Bottom: BB-BROV-203, Core No. R1 23.7 ft left to 28.7 ft right. Bottom: BB-BROV-203, Core No. R2 28.7 ft left to 33.7 ft right.



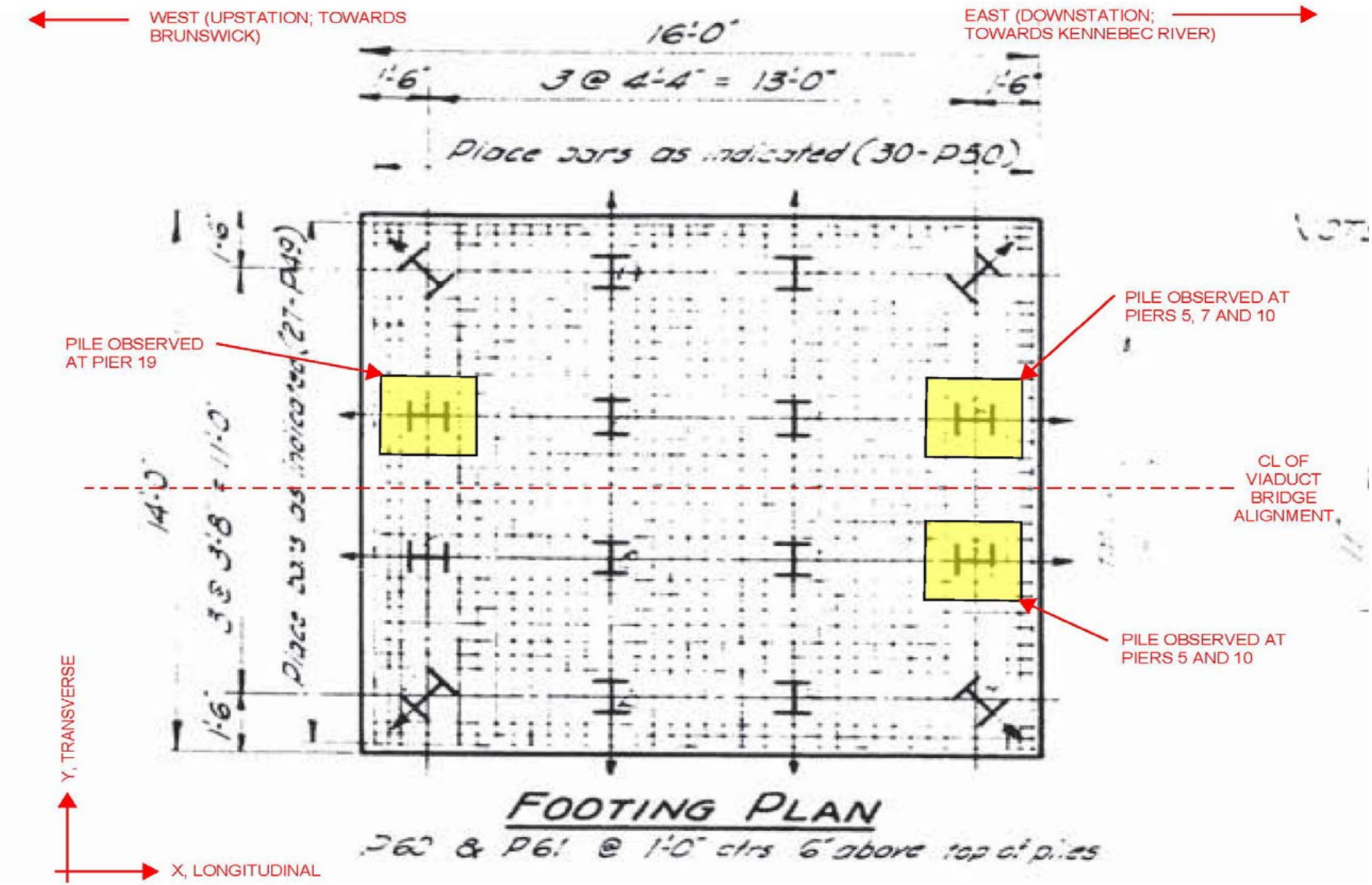
Photograph No. 3: Collected rock core samples for test borings BB-BROV-203 and BB-BROV-204 Top: BB-BROV-203 Core No. R3 33.7 ft left to 38.7 ft right.
Middle/Top: BB-BROV-204 Core No. R1 26.3 ft left to 31.3 ft right. Middle/Bottom: BB-BROV-204 Core No. R2 31.3 ft left to 35.9 ft right. Bottom: BB-BROV-204 Core No. R3 35.9 ft left to 40.9 ft right.

Photograph No. 4: Collected rock core samples for test boring BB-BROV-205 Top: Core No. R1 46.5 ft left to 51.5 ft right.
Middle/Top: Core No. R2 51.5 ft left to 56.5 ft right. Middle/Bottom: Core No. R3 56.5 ft left to 61.5 ft right.

Photograph No. 5: Collected rock core samples for test boring BB-BROV-206 Top: Core No. R1 41.4 ft left to 45.9 ft right.
Middle/Top: Core No. R2 45.9 ft left to 50.9 ft right. Middle/Bottom: Core No. R3 50.9 ft left to 55.9 ft right.

APPENDIX C

Test Pit Documentation


WEST (UPSTATION; TOWARDS BRUNSWICK)
EAST (DOWNSTATION; TOWARDS KENNEBEC RIVER)
16'-0"
1'-6"
3 @ 4'-4" = 13'-0"
1'-6"
Place bars as indicated (30-P50)
PILE OBSERVED AT PIER 19
PILE OBSERVED AT PIERS 5, 7 AND 10
CL OF VIADUCT BRIDGE ALIGNMENT
PILE OBSERVED AT PIERS 5 AND 10
14'-0"
FOOTING PLAN
Y, TRANSVERSE
X, LONGITUDINAL

04/09/14 - 1

Overall view of test pit, looking west.

04/09/14 - 2

Closeup view of test pit with two steel H-piles visible just inside of trench box, left and right, looking west.

04/09/14 - 3

Closeup view of left pile with black staining and no signs of corrosion visible. Pile embedded in moist to wet naturally deposited soil.

04/09/14 - 4

Closeup view of right pile with black staining and no signs of corrosion visible. Pile embedded in moist to wet naturally deposited soil.

04/14/14 - 1

Closeup view of test pit with one steel H-pile visible just inside of trench box, right, looking west.

04/14/14 - 2

Closeup view of right pile embedded in moist to wet naturally deposited soil.

04/14/14 - 3

Closeup view of right pile with slight "chipping" of apparent pile coating and no signs of corrosion visible.

04/14/14 - 4

Closeup view of right pile flange and slight "chipping" of apparent pile coating and no signs of corrosion visible.

04/15/14 - 1

Overall view of test pit, looking west.

04/15/14 - 2

Closeup view of right pile embedded in moist to wet naturally deposited soil with no signs of corrosion visible.

04/15/14 - 3

Closeup view of left pile embedded in moist to wet naturally deposited soil with no signs of corrosion visible.

04/15/14 - 4

Closeup view of left pile flange with no signs of corrosion visible.

04/10/14 - 1

Overall view of test pit, looking northeast.

04/10/14 - 2

Closeup view of test pit with one steel H-pile visible just inside of trench box, left, looking east.

04/10/14 - 3

Closeup view of left pile embedded in saturated fill and naturally deposited soil with pitting visible but no measurable signs of corrosion.

04/10/14 - 4

Closeup view of left pile embedded in saturated fill and naturally deposited soil with pitting visible but no measurable signs of corrosion.

APPENDIX D

Olson Engineering, Inc.
Non-Destructive Testing Report

INFRASTRUCTURE IMAGING AND NDE
ASSESSMENT, MONITORING AND REPAIR

Olson ENGINEERING
Northeast Region
47 Orient Way, Suite 1C
Rutherford, NJ 07070 USA
phone: 201.933.3777
fax: 201.933.8050

FOUNDATION LENGTH INVESTIGATION REPORT
PARALLEL SEISMIC TESTING
ROUTE 1 VIADUCT BRIDGE
BATH, MAINE

Prepared for:
Haley & Aldrich, Inc.
75 Washington Avenue, Suite 203
Portland, ME 04101-2617
Attn: Mr. Bryan C. Steinert, P.E.
Tel: (207) 482-4607

Olson Engineering Job No. 4589A
April 17, 2014

TABLE OF CONTENTS

1.0 INVESTIGATION SCOPE 1
2.0 PARALLEL SEISMIC METHOD 2
3.0 INVESTIGATION RESULTS AND SUMMARY OF FINDINGS 4
4.0 CONCLUSIONS AND RECOMMENDATIONS 7

APPENDIX A - PARALLEL SEISMIC DATA PLOTS

1.0 INVESTIGATION SCOPE

This report presents the Nondestructive Testing (NDT) investigation results for the determination of the unknown lengths of six (6) steel H-piles under piles caps supporting the Route 1 Viaduct Bridge located in Bath, Maine. The viaduct bridge runs east to west and is located west of the Sagadahoc Bridge which connects Bath to Woolwich, Maine. The steel H-piles tested were located below concrete pile caps at Piers 5, 7, 10, 12, 19, and Abutment A2. The foundations were tested with the Parallel Seismic (PS) method. Six (6) boreholes were used for the investigation, each of which was cased with a 2 inch diameter PVC pipe. The boreholes were drilled and cased by Haley & Aldrich and the Maine Department of Transportation prior to the start of NDE.

The investigation was performed on April 8, 2014 by Mr. Aaron Homer, Project Manager of Olson Engineering, Inc. using the Parallel Seismic (PS) method. Hammer impacting was done vertically on the tops of the concrete pile caps. The Parallel Seismic tests were performed at hydrophone receiver depth intervals of nominally 2 ft for the entire water-filled length of the boreholes. For the testing, the hydrophone started at the bottom depth of each casing, and was then moved in 2 foot vertical increments up to the top of the casing. Data was collected at each interval depth. A discussion of the PS method and the investigation results are presented in the following sections.

2.0 PARALLEL SEISMIC METHOD

The Parallel Seismic (PS) method was used to nondestructively evaluate (NDE) the depth of six (6) foundations of the Route 1 Viaduct Bridge located in Bath, Maine. The PS test equipment used in this investigation included a 4 lb hammer, single hydrophone receiver, accelerometer, and a dynamic signal analyzer (Olson Instruments Freedom Data PC), as illustrated in Figures 1 and 2. When the hammer impacted the concrete pile cap, the PC-based signal analyzer was triggered by the accelerometer on the pile cap to capture the time records. A National Instruments digital data acquisition card was used to acquire the data in an Olson Instruments portable Freedom Data PC.

The PS method involves impacting the exposed portion of the foundation or substructure attached to the foundation or a location which, when impacted, couples sufficient energy to the pile to generate a sound or stress wave which travels down the foundation. The wave energy was tracked by a hydrophone receiver suspended in a water-filled, cased borehole. The boreholes were located 2 feet or less from the nearest edge of the concrete pile caps. The PS tests typically involve lowering the hydrophone(s) to the bottom of the borehole, impacting the exposed portion of the foundation structure and recording the hydrophone(s) responses. The hydrophone receiver(s) is then raised approximately 2 feet to the next test elevation. This test sequence is repeated until the top of the casing (or the top of the water level in the casing) is reached.

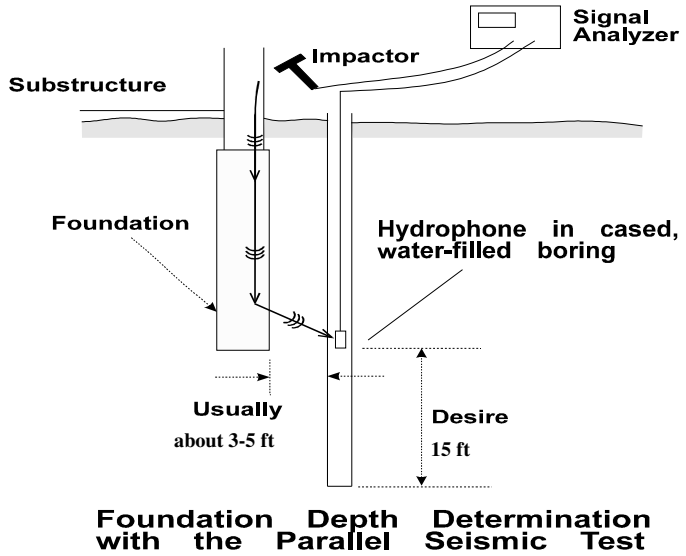


Figure 1 - Parallel Seismic (PS) Method for Unknown Foundation Depth Determination

The pile depth is determined by plotting the hydrophone(s) response from all depths on a single display or page. For soils of constant velocity surrounding the pile, a break in the slope of the line occurs below the bottom of the pile indicating the pile depth. For soils with varying velocities, a break often cannot be identified from the slope of the lines, but the bottom of the pile can be identified by observing the traces of the hydrophone plot to identify changes in the response, such as a reduction in signal amplitude, change in signal frequency, or diffraction/reflection of tube wave energy from the tips of piles (diffraction is frequently the indicator for the pile tip depth of steel H- and sheet pilings).

Figure 2 - Parallel Seismic (PS) setup in the field at the southwest corner of abutment A2 the yellow arrow shows the location of the 2" PVC casing. The 4 lb hammer shown was used for impacting the concrete pile cap, and the accelerometer was grease mounted to the concrete to trigger the system on each hammer blow.

3.0 INVESTIGATION RESULTS AND SUMMARY OF FINDINGS

The Parallel Seismic tests were performed using a 4 lb hammer to impact the tops of each pile cap, with an accelerometer used as the trigger, and a single Olson Instruments hydrophone as a receiver. The test locations were selected by Haley & Aldrich, Inc.

The Parallel Seismic data and results are presented in the summary Table 1 and presented in Figs. A-1 thru A-6 of Appendix A. The PS results presented in Appendix A are from 4 lb hammer hits on the tops of the concrete pile caps located 2 feet or less from the borings. The vertical axes in Figures A-1 thru A-6 represent the depth below the casing top, with each waveform at 2 feet intervals starting from the bottom waveform. The horizontal axis in each plot represents acoustic wave travel time in milliseconds (ms). The results for 5 of the 6 tested pile locations showed relatively clear tip depth indications with good resolution on the actual tip depth and moderate to high confidence in the data results. Based on the results and our past experience, we would expect the accuracy of the tip depth indications to be +/- 1 foot for high confidence data, and +/- 2 feet for moderate confidence data. The data collected for Pier 12, however, resulted in barely analyzable data that had a very low confidence in the tip depth measurement. Thus, the Pier 12 data is considered to be unreliable and therefore unusable.

Testing results on southeast corner of the pile cap of Pier 5 indicate a clear tip depth at 13.8 feet below the top of the PVC casing (top of casing is 1.6 feet above the concrete pile cap). The graph shows a set of wave arrivals at shallow depths with a slope corresponding to the expected pile velocity for a steel H-pile, given the inherent uncertainly in estimating pile velocity from PS data for short piles. This sloping line intersects a set of arrivals typical of saturated soils at the given pile tip depth.

Testing results on the southeast corner of the pile cap of Pier 7 testing indicate a clear tip depth at 18.1 feet below the top of the PVC casing (top of casing is 1.6 feet above the concrete pile cap). The graph shows a set of wave arrivals at shallow depths with a slope corresponding to the expected pile velocity range for a steel H-pile.

Testing results on the northeast corner of the pile cap of Pier 10 indicate a clear tip depth at 24.4 feet below the top of the PVC casing (top of casing is 1.7 feet above the concrete pile cap). The graph shows a set of wave arrivals at shallow depths with a slope corresponding to the expected pile velocity range for a steel H-pile.

Testing results on the southwest corner of the pile cap of Pier 19 indicate a clear tip depth at 25.1 feet below the top of the PVC casing (top of casing is 1.8 feet above the concrete pile cap). There is also visible, however, a relatively clear pile tip depth indication for a second pile at a depth of 42.5 feet below top of casing. There are two figures presented in the appendix, showing both tip depths for apparently two different steel piles. Note that the slope of the arrivals below the upper pile tip as well as the shape of the arrival for the lower pile indicates that the deeper pile is significantly further away from the borehole than the shallow pile, likely 10-15 feet or more laterally.

Testing results on the southwest corner of the pile cap of Abutment A2 indicate a less-clear tip depth at 35.1 feet below the top of the PVC casing (top of casing is 2.1 feet above the concrete pile cap). The data set was relatively noisy and resulted in moderate confidence in the analyzed result.

Table I

Summary of Bridge Foundations

Structure	Approximate Horizontal/Lateral Distance Borehole to Pile cap (in.)	Total Borehole Casing Depth (ft.)	Vertical Distance from Top of PVC to top of Pile Cap (ft.)	Approximate Depth of Pile Tip Below Casing Top (ft.)	Approximate Pile Tip Depth Below Top of Pile Cap (ft.)	PS Data Quality
Pier 5	8	28.5	1.6	13.8	12.2	Good
Pier 7	16	36.5	1.6	18.1	16.5	Good
Pier 10	16	38.7	1.7	24.4	22.7	Good
Pier 12	10	41.5	1.3	Data not usable	N/A	N/A
Pier 19	3	63.3	1.8	25.1 42.5	23.3 40.7	Good
Abutment A2	24	56.3	2.1	35.1	33.0	Moderate

4.0 CONCLUSIONS AND RECOMMENDATIONS

The waveforms for five of the six piles, Pies 5, 7, 10, 19 and Abutment A2, show clear indications of the pile tip depths. The data from Pile 19 shows two apparent pile tip depths, one shallow and one deeper. None of the other piles show this type of result, although it would only be typically expected at deeper pile locations. These tip depths are considered to have a moderate (A2) to high confidence. The data set for Pile 12 was found to be very noisy and irregular, and resulted in data with a very low confidence in the tip depth prediction. Thus, the data for Pier 12 is considered to be unusable for analysis. Note that the boring log for the Pile 12 borehole showed the presence of a significant layer of organics with debris present. The plots for the six tested pile can be found in Appendix A.

CLOSURE

The field NDT investigation was performed in accordance with generally accepted testing procedures. Pile depth data reported is considered to be accurate to ±1 to 2 ft as noted in the report text. Please contact us if you have any questions or require any further information.

Respectfully submitted,

OLSON ENGINEERING, INC.

Aaron Homer
Project Manager

Dennis A. Sack, P.E.
Senior Vice President and Principal Engineer

(1 copy sent as e-mailed pdf document, 1 copy mailed)

APPENDIX A

PARALLEL SEISMIC DATA

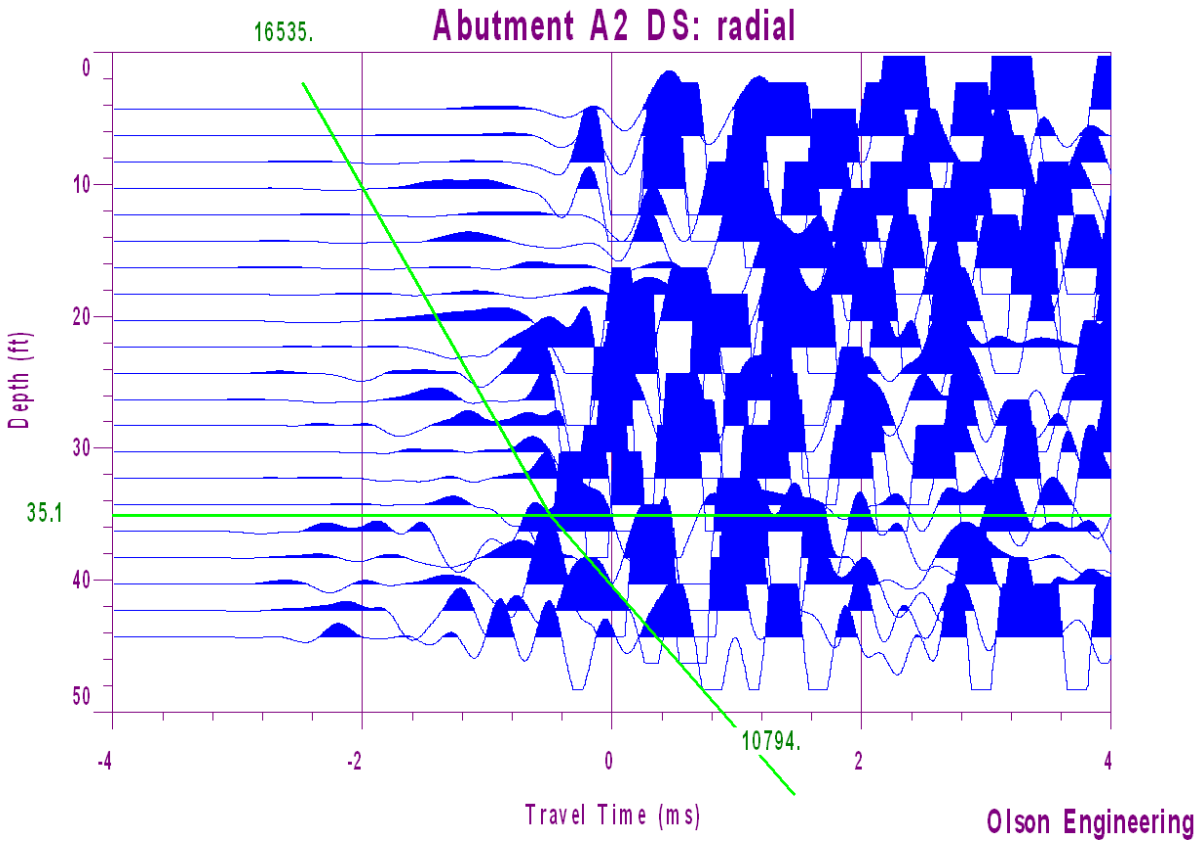


Figure A-1. Results for Abutment 2, Showing Tip Depth of 35.1 ft. Moderate Confidence

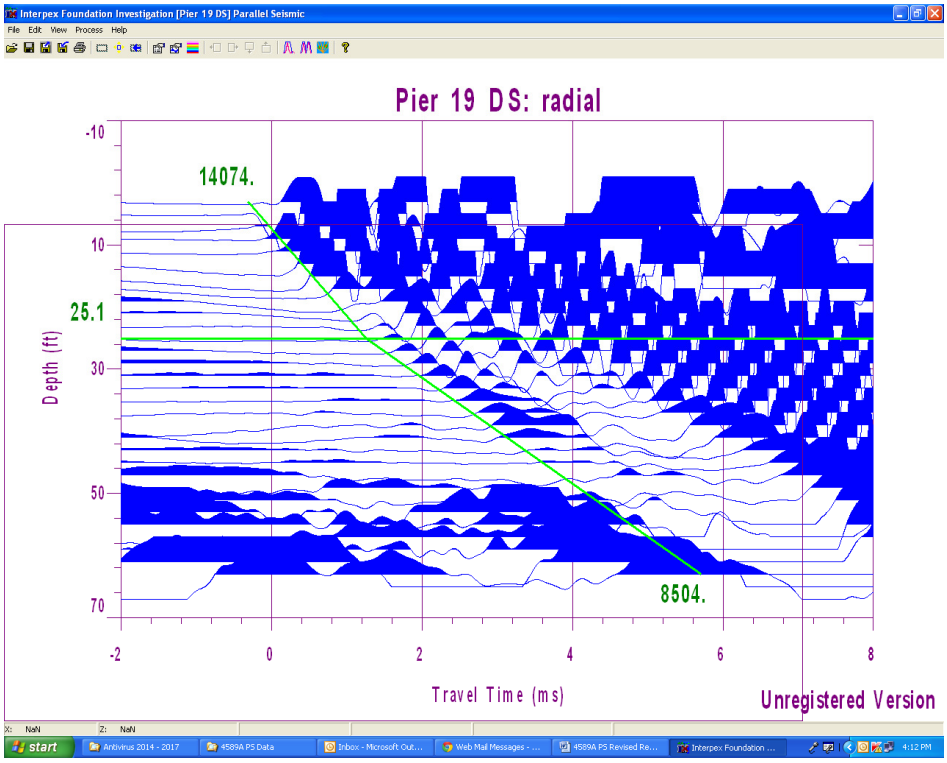


Figure A-2a. Results for Pier 19, Showing Tip Depth of 25.1 ft.

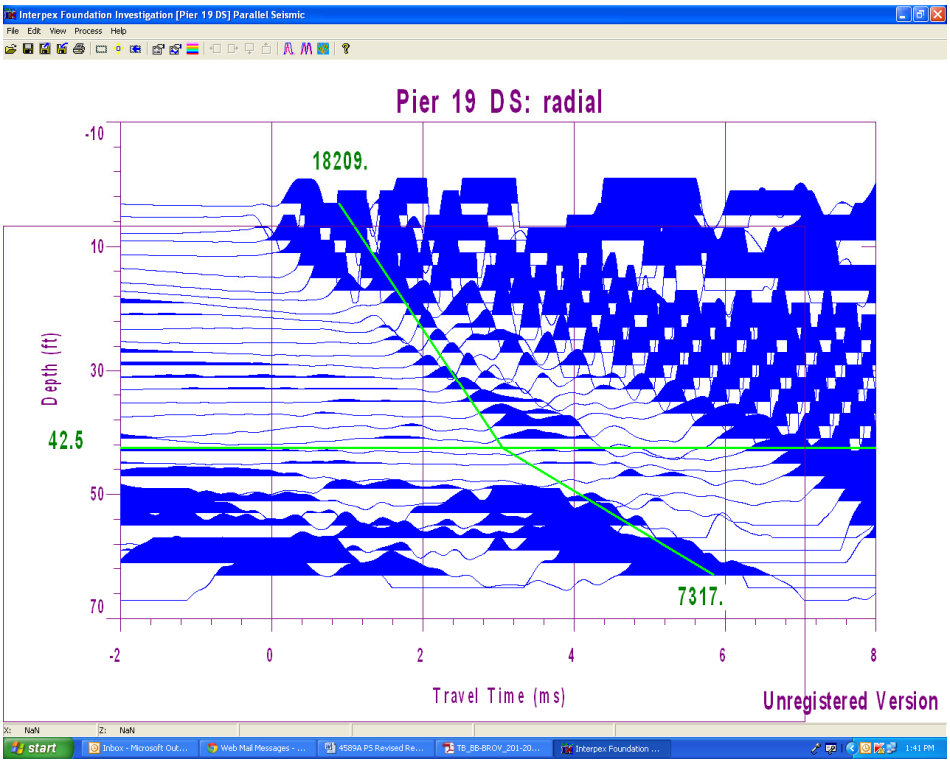


Figure A-2b. Results for Pier 19, Showing Second Tip Depth of 42.5 ft.

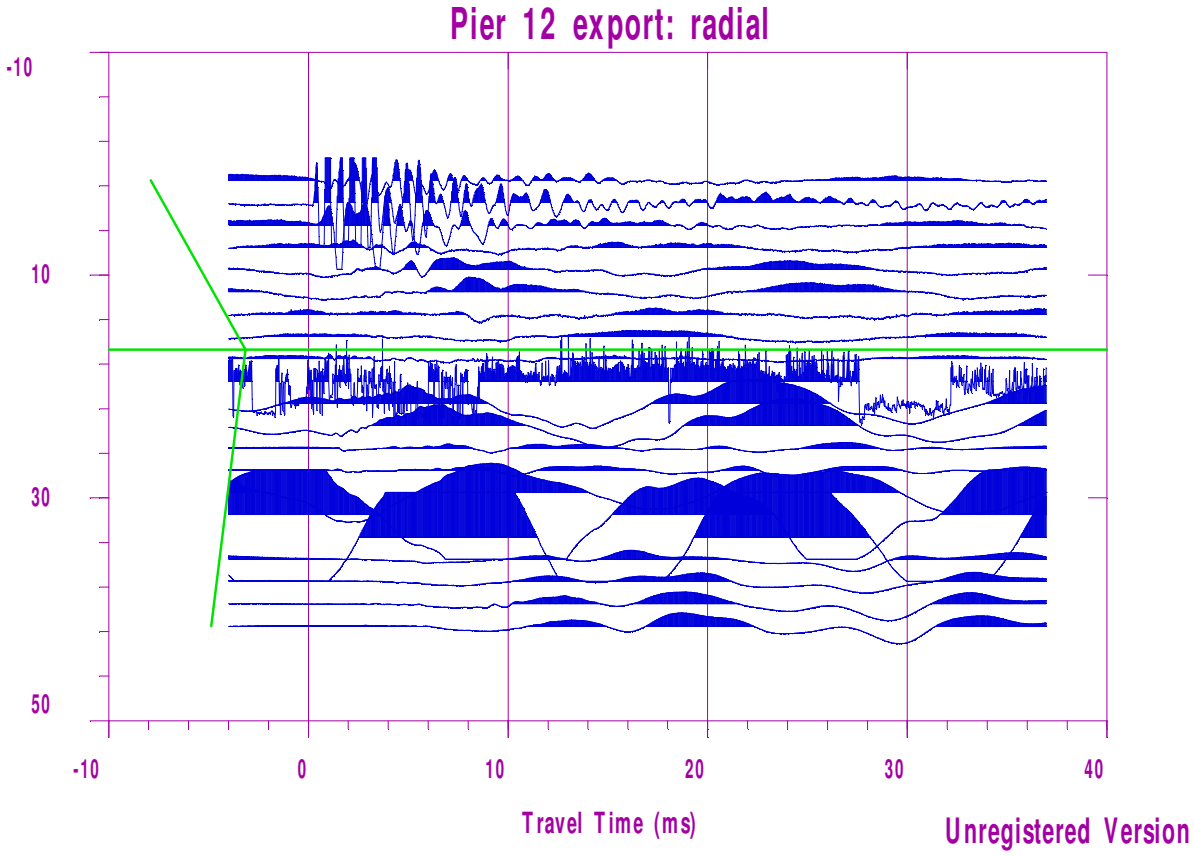


Figure A-3. Results for Pier 12, data not usable.

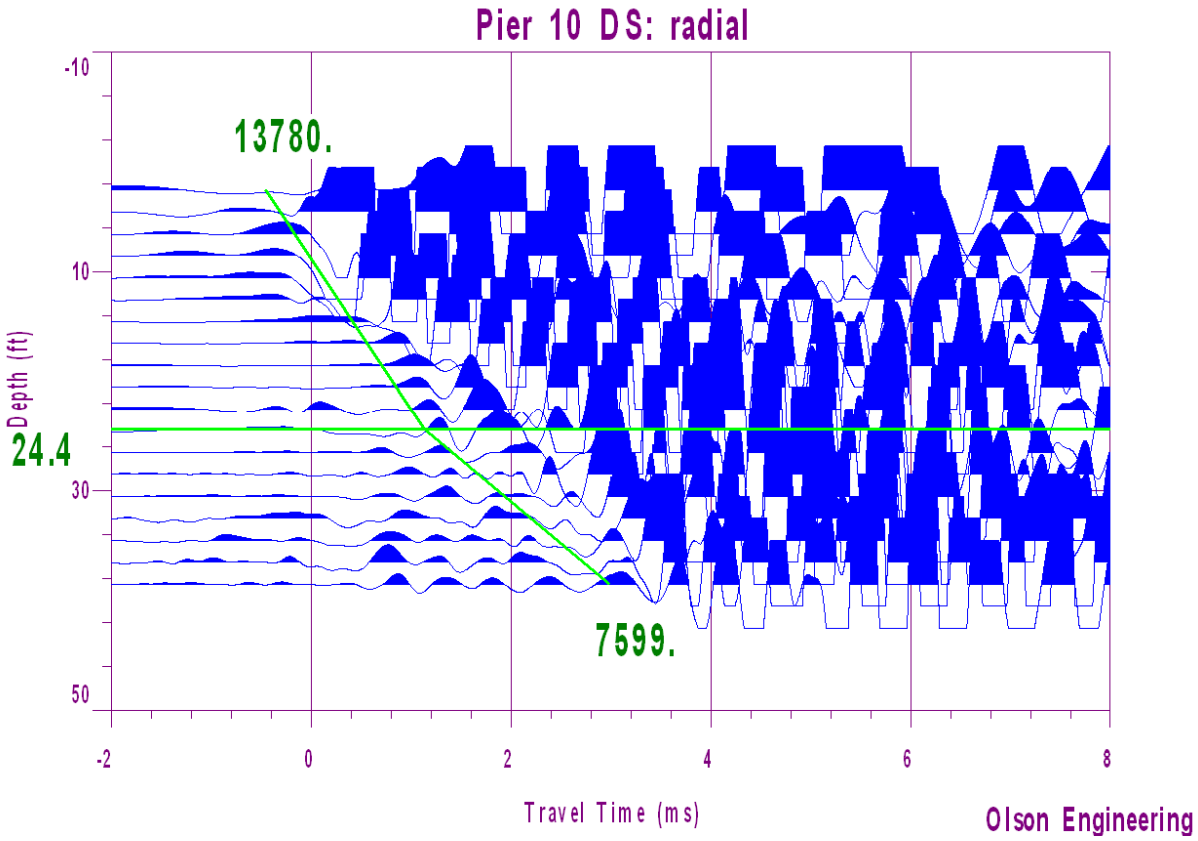


Figure A-4. Results for Pier 10, Showing Tip Depth of 24.4 ft.

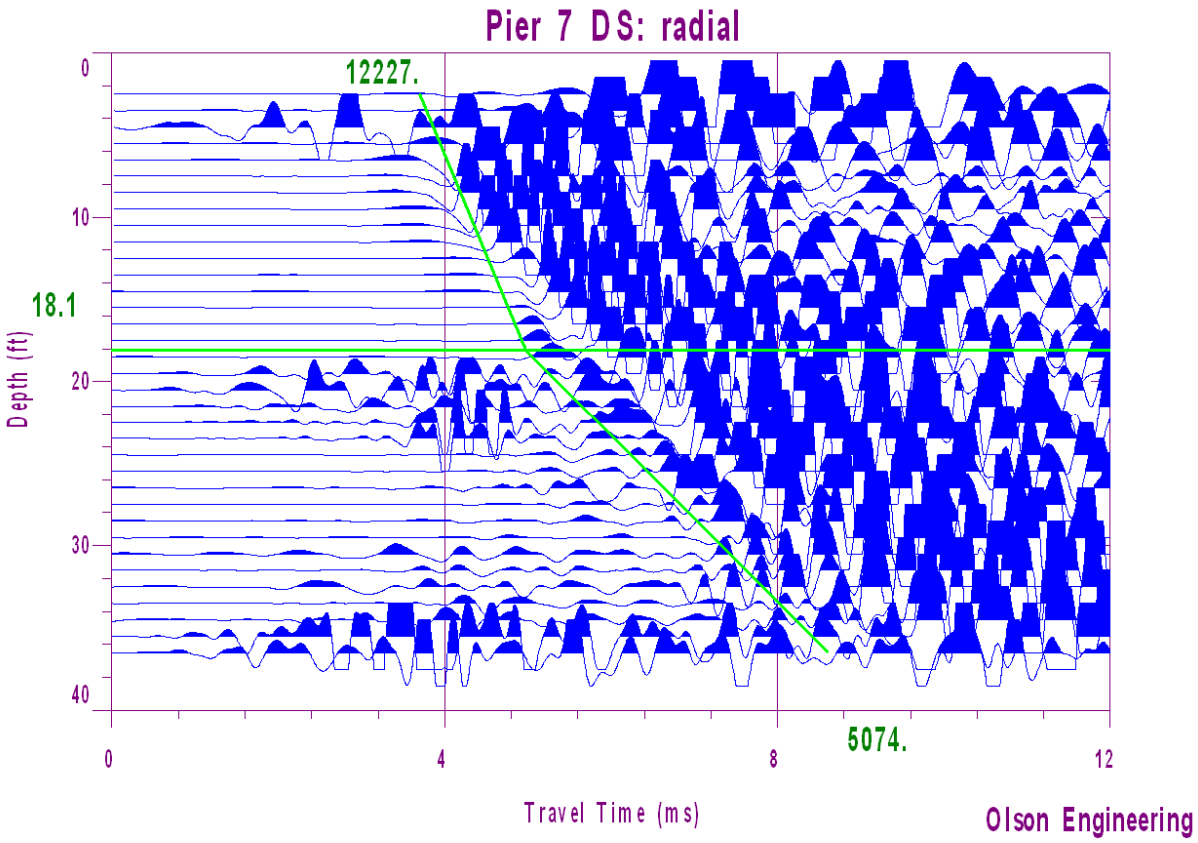


Figure A-5. Results for Pier 7, Showing Tip Depth of 18.1 ft.

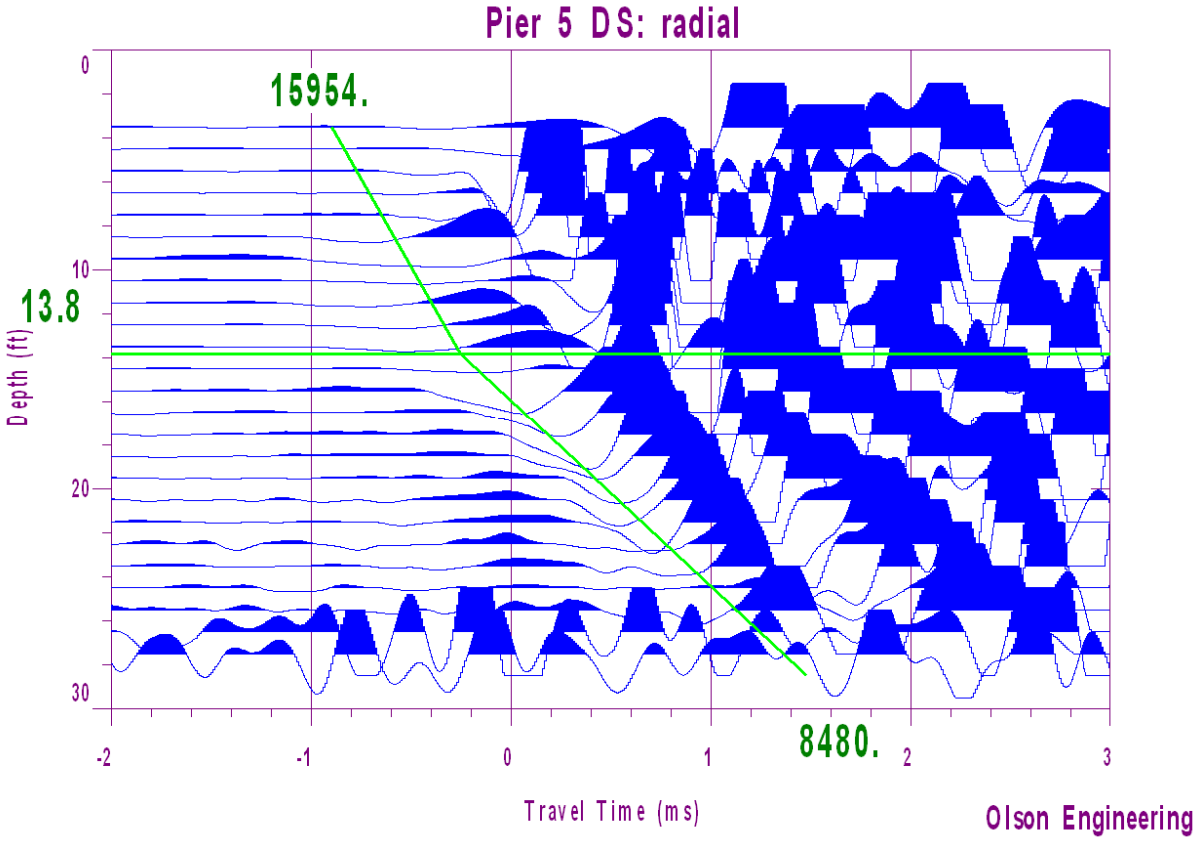


Figure A-6. Results for Pier 5, Showing Tip Depth of 13.8 ft.

APPENDIX E

Historic Viaduct Bridge Information

Survey Field Book
Information

Oct. 2, 1957

		*			
					57.97 ~~10.54~~
BM#1	10.54	116.48			
TP#1	7.34	122.05	1.77	114.71	
TP#2	2.94	124.54	0.45	121.60	
B# 2	6.49	125.11	5.92	~~118.62~~	
TP#1	4.61	121.58	8.14	116.97	
TP#2	8.00	125.93	3.65	117.93	
BM#3	1.36	119.28	8.01	117.92	
				-47.97	
				69.95	
BM#3	4.57	122.49			117.92
TP#1	6.47	125.62	3.34	119.15	
TP#2	4.41	120.90	9.13	116.49	
TP#3	5.27	125.24	0.93	119.97	
BM#2	5.87	124.48	6.63	118.61	118.62
TP#1	3.67	121.82	6.33	118.15	
TP#2	1.13	115.39	7.56	114.26	
BM#1			9.47	105.92	

* To change Elevs. to M.S.L. Subtract 47.97' from above B.M.'s, (As per telephone call 6/22/59 from Mr. Furber.) [illegible] 6/22/59

III T Dumont
Rod. Sandblomb
J. Bruce

BK 26/41
26-1(10)

Cloudy

18

Inside corner shelf at west end of North Pier High Street Bridge

Top of Rock 398+50 - 60' Lt.

395+25 - 19' Lt. top of Rock

Vert. Spk in Trunk of 40" Elm STA 374+27 - 123' Lt.

STA 371+19 - 22' Lt. Top Rock.

STA 367+08 - 20' Rt. Top Rock

STA 363+08 - 64' Rt. Vert. Spk in 30" Maple

364+42 - 20' Rt. top Rock

STA 369+12 - 14' Rt. Top of Rock

STA 369+12 - 14' Rt. Top of Rock.

394+80 - 14' Rt. top Rock

STA. 377+92 - 17 Rt. top Rock

381+15 - 54 Lt.

Maine Department of Transportation

Control Point Description

NAD 83 DATUM -- METRIC

Photogrammetry & Control Unit
(207)624-3493

16 State House Station
Augusta, ME 04333-0016

3837

ME2000 Zone **W** Adjustment **NAD83(1996)** Type **B**

NORTH-2000 **119838.926** EAST-2000 **344572.721** City **BATH** County **SAG**

IMPNORTH-2000 **393171.543** IMPEAST-2000 **1130485.669** GPS **Y**

R=RTK
Y=MDOT(GPS)
F=FEDERAL BASENET
N=CLASSICAL TRAVERSE
C=COOPERATIVE BASENET

NAVD88 ELEV **22.443** IMP ELEV **73.632** Order **2**

Described By **HEN** Date Set **15-NOV-84** Project **01012100**

Description

STATION NAME: 3837 REC Jan 2002 by K.C. Reynolds and Crew. Est in 1984 under PIN 2393.00 Bath. Located in the town of Bath, the station is set in the Easterly sidewalk of the Rte. 209 bridge over U.S. Rte. 1 in front of a plaque reading "Paul L. Davis Memorial Bridge WW II Veteran". The station mark is a standard disk, stamped --- 3837 1984 ---, set in a drill hole in the cement sidewalk on the Easterly side of Rte. 209 overpass bridge over Rte. 1 in Bath. It is: 0.5 m (1.7 ft) East-Southeast of the face of curb 11.9 m (39.2 ft) South-Southwest of a conc. lamp pole at the Northeast corner of the bridge.

	NAD83	ZONE	NAD83 (1992)		
NORTH	**119745.983**	**W**	GPSNORTH	NGVD 29 ELEV	**22.654**
EAST	**927837.912**		GPSEAST	IMP ELEV	**74.324**
SOURCE	**NGVD 29**				

To Station	Grid Distance	Grid Azimuth
BW-70	**1544.656**	**85 41 39.1"**
BW-25	**337.896**	**252 45 22.1"**
CUSTOM	**570.774**	**75 06 14.8"**
	.000	**.0"**

*Latitude: **435438.397** *Longitude: **694912.340** *Not for precision use

***Coordinates Established for Maine Department of Transportation use, Subject to Higher Order Adjustment

***Conversion factor: 1 meter = (39.37in)/(12in/ft) or 1 meter = 3.280833333333 ft. DO NOT ROUND OFF

Bridge Drawings

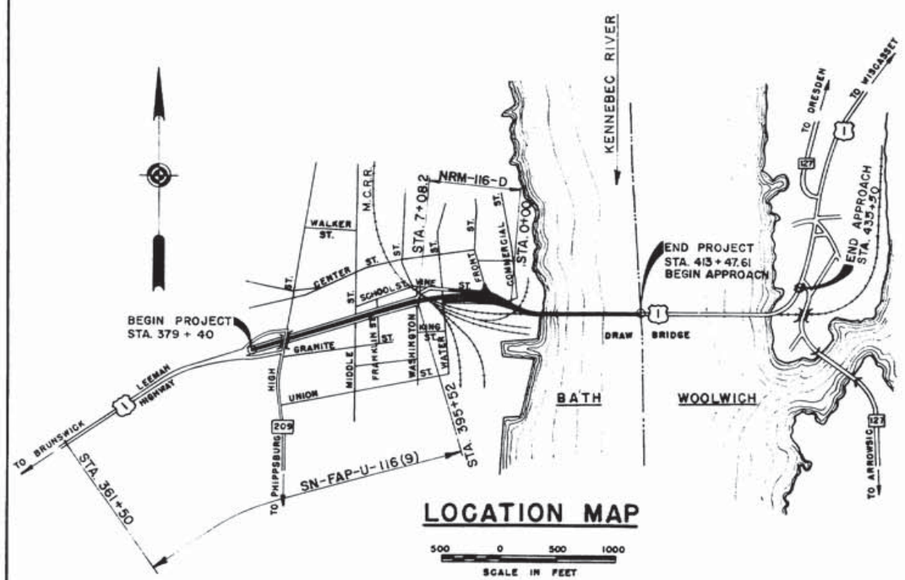
B. P. R. DIV. NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
1	MAINE	U-026-1(8)	1	148

STATE OF MAINE
STATE HIGHWAY COMMISSION

CARLTON BRIDGE WIDENING
AND
WEST APPROACH
BETWEEN
BATH AND WOOLWICH
SAGADAHOC COUNTY, MAINE

FEDERAL AID PROJECT NO. U-026-1(8)

LENGTH OF PROJECT 0.645 MILE



LOCATION MAP

A.D.T. 1959 = 2470

A.D.T. 1979 = 3285

D.H.V. = 330

D = 65 %

T = 6 %

V = 40 MPH

CONCRETE CLASSIFICATION

CONCRETE FILL ..... CLASS "B"
ALL OTHER CONCRETE ..... CLASS "A"

LOADING H20-S16-44

fs = 20000 p.s.i.
fc = 1200 p.s.i.
n = 10

SPECIFICATIONS

MAINE STATE HIGHWAY COMMISSION STANDARD SPECIFICATIONS, REVISIONS OF JAN. 1956, EXCEPT AS MODIFIED ON THESE PLANS AND IN THE SUPPLEMENTAL SPECIFICATIONS.

WEST APPROACH
INDEX OF SHEETS

1 ..... TITLE SHEET
2 ..... STANDARD DETAILS - CATCH BASINS, MANHOLES, ETC.
3 ..... STANDARD DETAILS - GUARD RAIL & FITTINGS, ETC.
4 ..... STANDARD DETAILS - MISCELLANEOUS ITEMS
5 ..... GENERAL PLAN - WEST APPROACH
6-7-8-9 ..... SURVEY
10 ..... BORINGS
11 ..... STRATIGRAPHICAL PROFILE
12 ..... ALIGNMENT AND PROFILE STA. 379 TO 387
13 ..... ALIGNMENT AND PROFILE STA. 387 TO 396
14 ..... ALIGNMENT AND PROFILE STA. 396 TO CARLTON BRIDGE
15 ..... TRESTLE PROFILE STA. 379 TO 388
16 ..... TRESTLE PROFILE STA. 388 TO 397
17 ..... TRESTLE PROFILE STA. 397 TO 403 + 50
18 ..... PROFILE - FRONT ST. TO CARLTON BRIDGE
19 ..... PROFILE - VINE STREET
20 ..... PROFILE - RAMP
21 ..... TYPICAL CROSS SECTIONS
22-23-24-25 CROSS SECTIONS
26 ..... TRAFFIC ISLAND STA. 379 + 50 TO 381 + 50
27 ..... PROW A, AND TRAFFIC ISLANDS STA. 382 + 50 TO HIGH ST.
28 ..... PLAN OF MEDIANS AT PIERS 6-7-9-10-14
29 ..... LAYOUT - VINE AND WATER STREETS
30 ..... LAYOUT - VINE AND FRONT STREETS
31 ..... ROADWAY DRAINAGE
32 ..... RELOCATION OF SIDEWALK AT WASHINGTON STREET
33 ..... ABUTMENT NO. 1
34-35 ..... ABUTMENT NO. 1 - RETAINING WALLS
36 ..... PIERS NO. 1 THRU 11
37 ..... PIERS NO. 12 THRU 19
38 ..... ABUTMENT NO. 2
39-40 ..... NORTH AND SOUTH RETAINING WALLS - ABUTMENT NO. 2
41-42 ..... RAMP RETAINING WALL
43 ..... VINE STREET RETAINING WALL
44 ..... VINE STREET SIDEWALK STA. 399 + 25 TO 399 + 94*
45 ..... PROW B
46-47 ..... DETAILS - T-BEAM SPANS 1 TO 6
48-49 ..... DETAILS - T-BEAM SPANS 7 TO 12
50 ..... RAIL AND RAIL CURB LAYOUT - T-BEAM SPANS 1 TO 12
51 ..... REINFORCING STEEL LAYOUT - T-BEAM SPANS 1 TO 6
52 ..... REINFORCING STEEL LAYOUT - T-BEAM SPANS 7 TO 12
53 ..... EXPANSION DAMS AND BEARINGS - T-BEAM SPANS 1 TO 12
54 ..... DRAINAGE DETAILS - T-BEAM SPANS 1 TO 12
55 ..... DRAINAGE DETAILS - STEEL SPANS 13 TO 20
56 ..... STRUCTURAL STEEL - SPANS 13 TO 20
57 ..... EXPANSION DAMS AND BEARINGS - SPANS 13 TO 20
58 ..... SHEAR CONNECTORS - SPANS 13 TO 20
59 ..... SUPERSTRUCTURE SLABS - SPANS 13 THRU 18
60 ..... SUPERSTRUCTURE SLABS - SPANS 19 & 20
61 ..... BLOCKING FOR SPANS 13 THRU 20, OFFSETS FOR SLAB SPANS 19 & 20
62 ..... RAIL LAYOUT - SPANS 13 THRU 20
63 ..... ALUMINUM RAIL - TYPICAL DETAILS, PYLON CAP DETAIL
64 ..... REINFORCING STEEL - ABUTMENT NO. 1 & RETAINING WALLS, VINE STREET RETAINING WALL & PROWS
65 ..... REINFORCING STEEL - ABUTMENT NO. 2 & RETAINING WALLS
66 ..... REINFORCING STEEL - RAMP RETAINING WALL, LIGHT POST BASES
67 ..... REINFORCING STEEL - PIERS 1 THRU 11
68 ..... REINFORCING STEEL - PIERS 12 THRU 19
69 ..... REINFORCING STEEL - SUPERSTRUCTURE, TRESTLE SPANS 1 THRU 20
70 ..... CONDUIT AND LIGHTING LAYOUT
71 ..... CONDUIT DETAIL AT PIER 6, AND SPAN 18
72 ..... CONDUIT DETAIL AND CONCRETE LIGHT BASES
73 ..... CONDUIT DETAIL AND HANGERS
74 ..... LIGHT STANDARDS AND ANCHOR BOLT DETAIL
75 ..... CONDUIT DETAIL - BATH AND WOOLWICH ABUTMENTS

USE SHEET NO. 70 FOR BOTH WEST APPROACH & CARLTON BRIDGE WIDENING
USE SHEETS NO. 73, 74, & 75 FOR CARLTON BRIDGE WIDENING

NOTE – FOR GENERAL PLAN - CARLTON BRIDGE WIDENING & INDEX OF SHEETS SEE SHEET NO. 76

APPROVED:
MAINE CENTRAL RAILROAD
J. W. Higgins
CHIEF ENGINEER

APPROVED:
MAINE STATE HIGHWAY COMMISSION
David H. Stevens
CHAIRMAN
Harold B. Emery
Carl Lawrence Gray
CHIEF ENGINEER

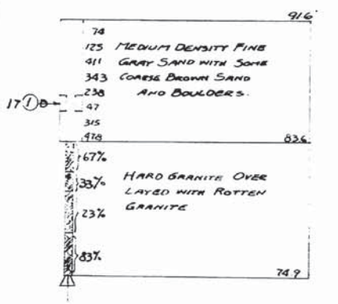
DEPARTMENT OF COMMERCE
BUREAU OF PUBLIC ROADS
REGION 1

APPROVED:

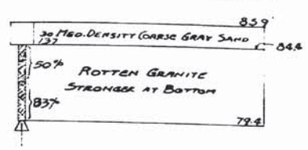
DIVISION ENGINEER ..... DATE

F. A. & B. Sta. No.	STATE	PROJECT NUMBER	SHEET NO	TOTAL SHEETS
-	MAINE	U-026-1(8)	10	9/48

BORING NO. 1
STA 385+95 CENTER ABUT. #1

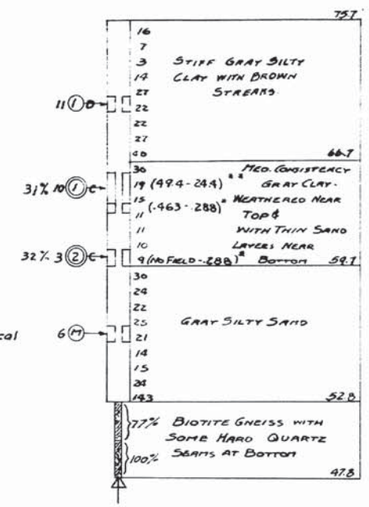


BORING NO. 2
STA. 387+15 CENTER PIER #2

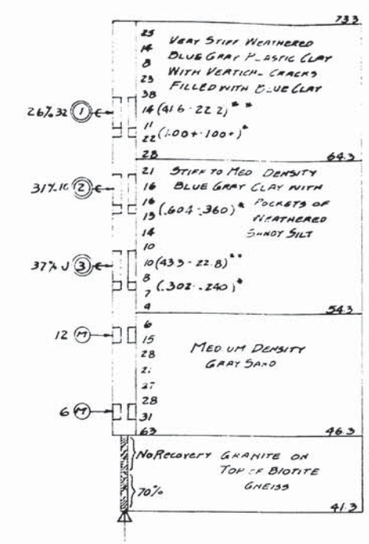


BORING NO. 3
STA. 388+37 CENTER PIER #4

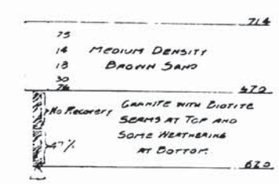
BORING NO. 4
STA 389+51 CENTER PIER #6



BORING NO. 5
STA 391+22 CENTER PIER #7

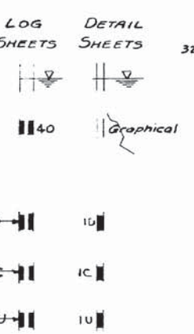


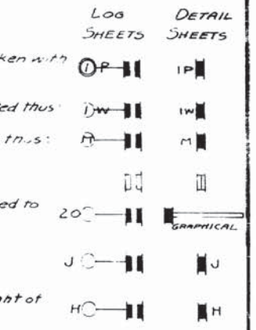
BORING NO. 6
STA 392+65 CENTER PIER #8



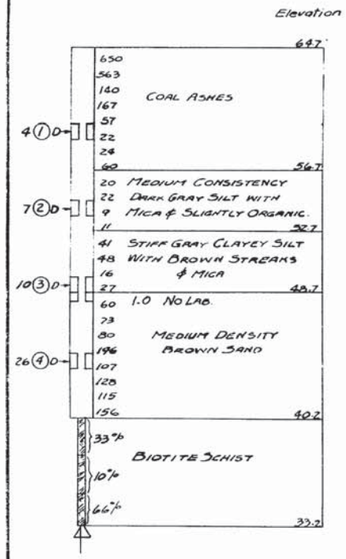
BORING NOTES

1. Scales as Noted on Drawings
2. Casing Size as Noted on Drawings.
3. Ground Water Table indicated thus:
4. Number of Blows of 275# Hammer falling 18 inches required to drive Extra Heavy Casing One Foot thus:
5. W, S, or V indicate respectively that the Hole was Washed, Sampled or Vaned ahead of Casing.
6. Location and designation of dry Samples taken in S.&H. sampler #1290s indicated thus:
7. Location and designation of dry samples taken in 2" O.D. 18 Ga. seamless Tubing indicated thus:
8. Location and designation of dry samples taken in 3½ O.D. 16 Ga. seamless tubing indicated thus:
9. Location and designation of 3½ O.D. dry samples taken with piston sampler indicated thus:
10. Location and designation of wash samples indicated thus:
11. Unsuccessful attempts to secure dry sample indicated thus:
12. Location of field vane test indicated thus:
13. Number of blows of 275# hammer falling 15" required to drive spoon or tubing one foot indicated thus:
14. Sampling spoon or seamless tubing driven by static load not exceeding ½ ton indicated thus:
15. Sampling spoon or seamless tubing driven by static weight of drill rods and 275# hammer indicated thus:

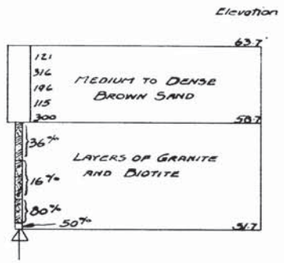




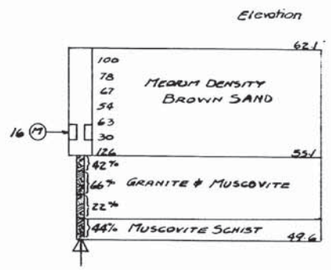
BORING NO. 7
STA 393+40 CENTER PIER #12



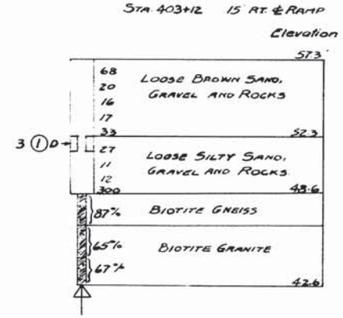
BORING NO. 8
STA 394+04 CENTER PIER #13



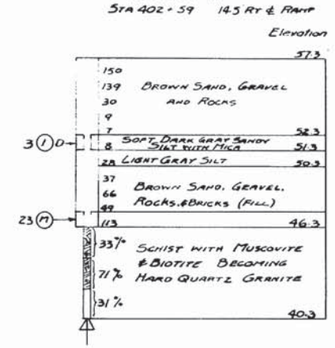
BORING NO. 9
STA 394+68 CENTER PIER #14



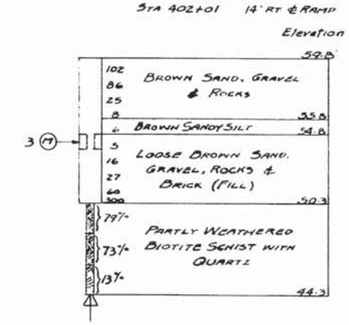
BORING NO. 10
LOCATION "O"
STA 403+12 15' RT. & RAMP



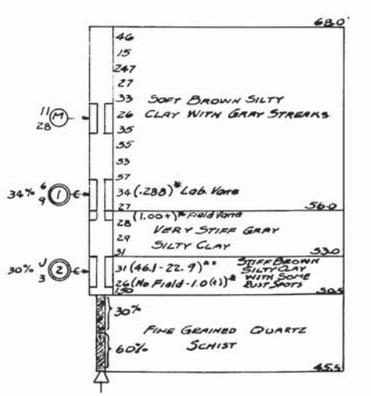
BORING NO. 11
LOCATION "M"
STA 402+59 14.5' RT. & RAMP



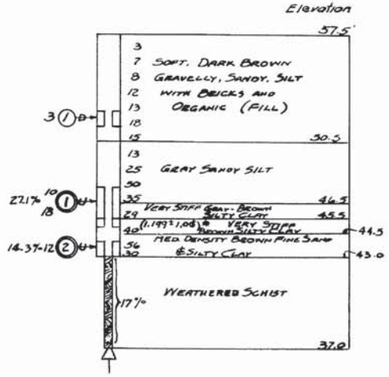
BORING NO. 12
LOCATION "K"
STA 402+01 14' RT. & RAMP



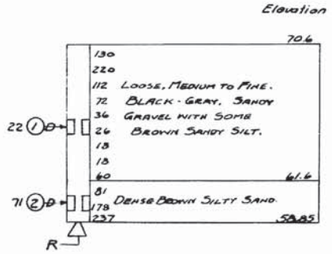
BORING NO. 14
STA 392+13 CENTER PIER #10



BORING NO. 15
LOCATION "E"
STA 400+34 47' RT. & TRESTLE

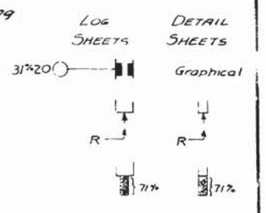


BORING NO. 23
STA. 401+00 15' RT. & TRESTLE



BORING NOTES

16. Where no resistance is specified spoon or tubing was driven by the static weight of drill rods.
17. Natural water contents, given as percent of dry weight are indicated thus:
18. Bottom of boring indicated thus:
19. Refusal of drill rods or casing indicated thus:
20. % recovery of rock core by size E diamond bit thus:



NOTES:

* Field Vane Shear Strength Followed by Laboratory Vane Shear Strength in Tons/sq. ft.

** Liquid Limit Followed By Plastic Limit (Per Cent of Water Content)

SCALE 1" = 5 FEET.

DESIGN - Garrill
TRACE - JONES
CHECK - Langford

WEST APPROACH

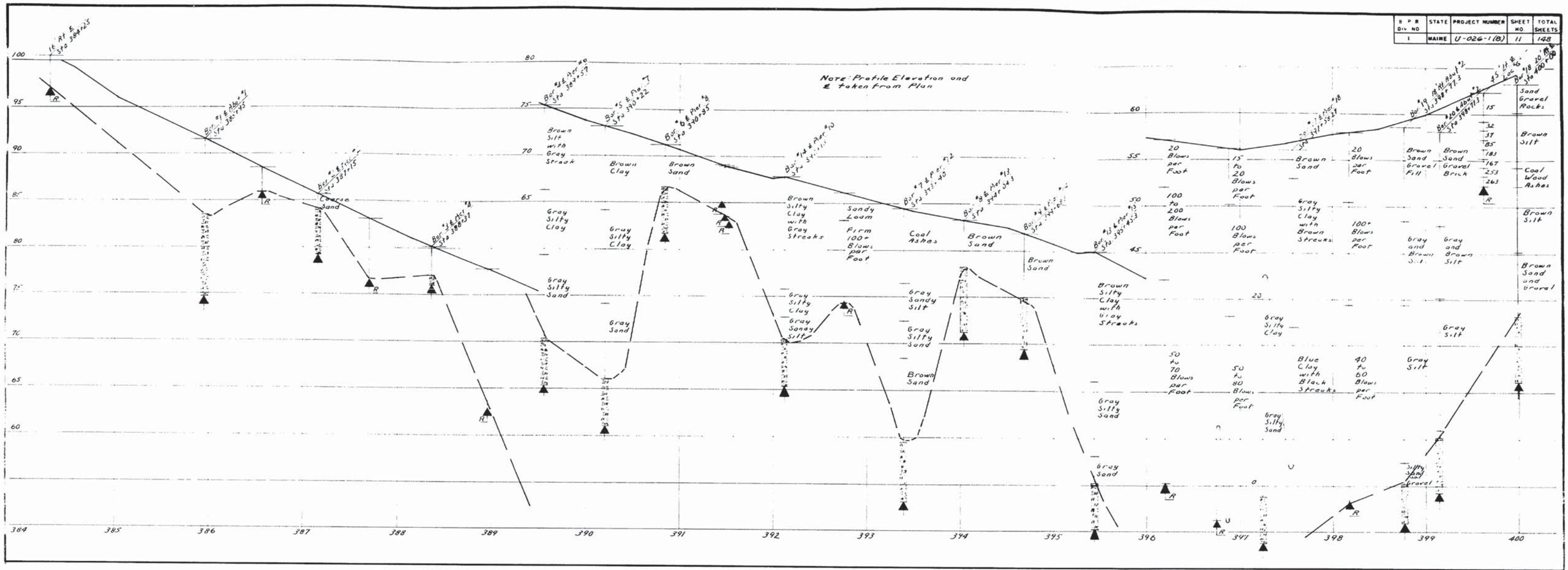
STATE HIGHWAY COMMISSION
BRIDGE DIVISION

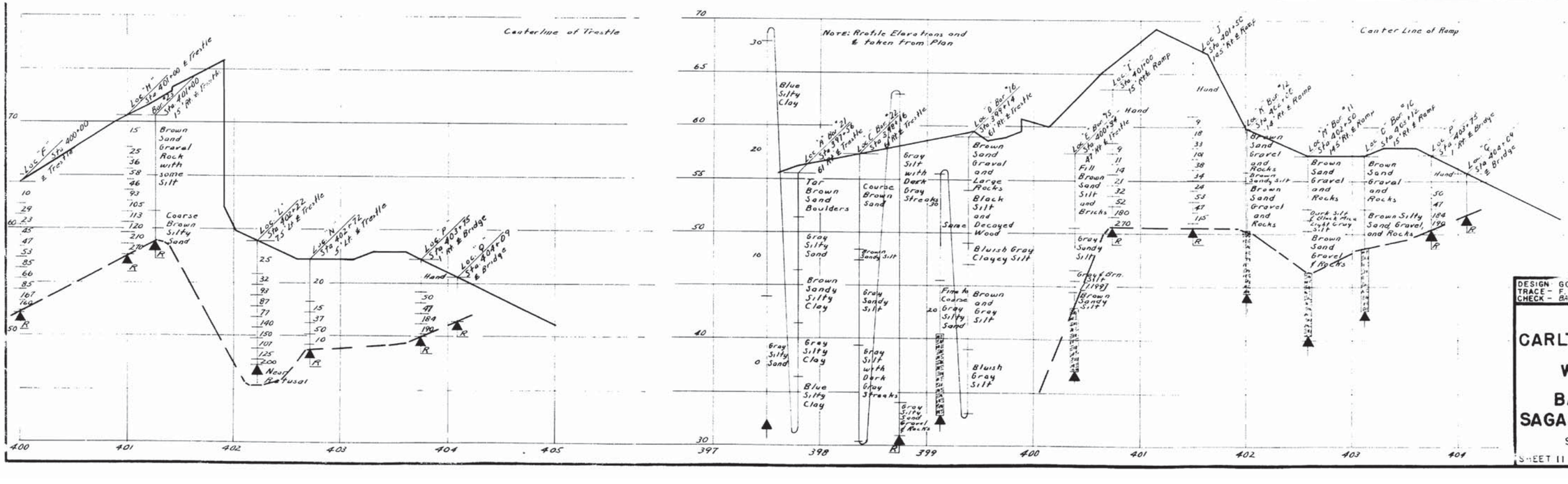
CARLTON BRIDGE WIDENING AND WEST APPROACH BETWEEN BATH & WOOLWICH SAGADAHOC COUNTY, MAINE

BORINGS

SHEET 10 OF 148 AUGUSTA, MAINE JAN. 1957

B P R Div NO	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	U-026-1(8)	11	148





DESIGN - GORRILL
TRACE - F. SMITH
CHECK - BARNES

WEST APPROACH

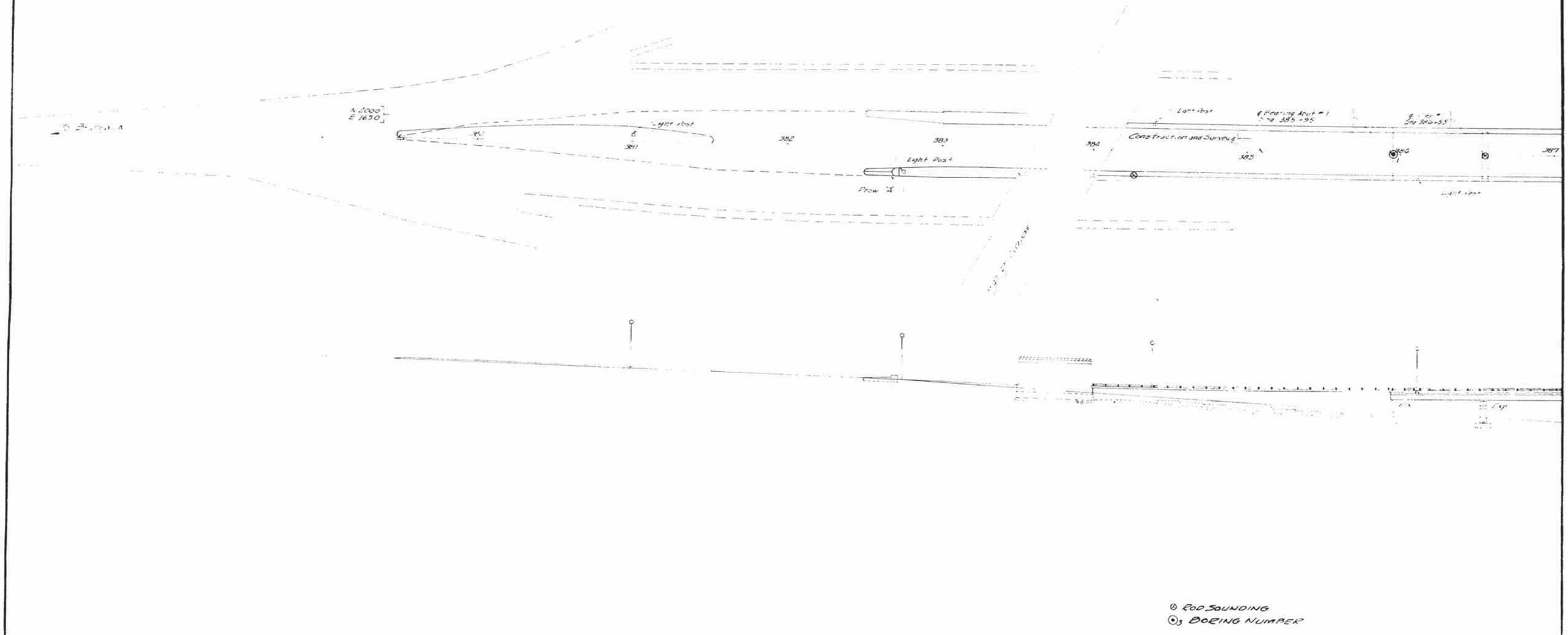
STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING AND WEST APPROACH BETWEEN BATH & WOOLWICH SAGADAHOC COUNTY, MAINE

STRATIGRAPHICAL PROFILE

SHEET 11 OF 148 AUGUSTA, MAINE JAN. 1957

B P R DIV. NO	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	U-026-1(8)	12	148



DESIGN - EVERETT
TRACE - WENTZEL
CHECK - TURNER

WEST APPROACH

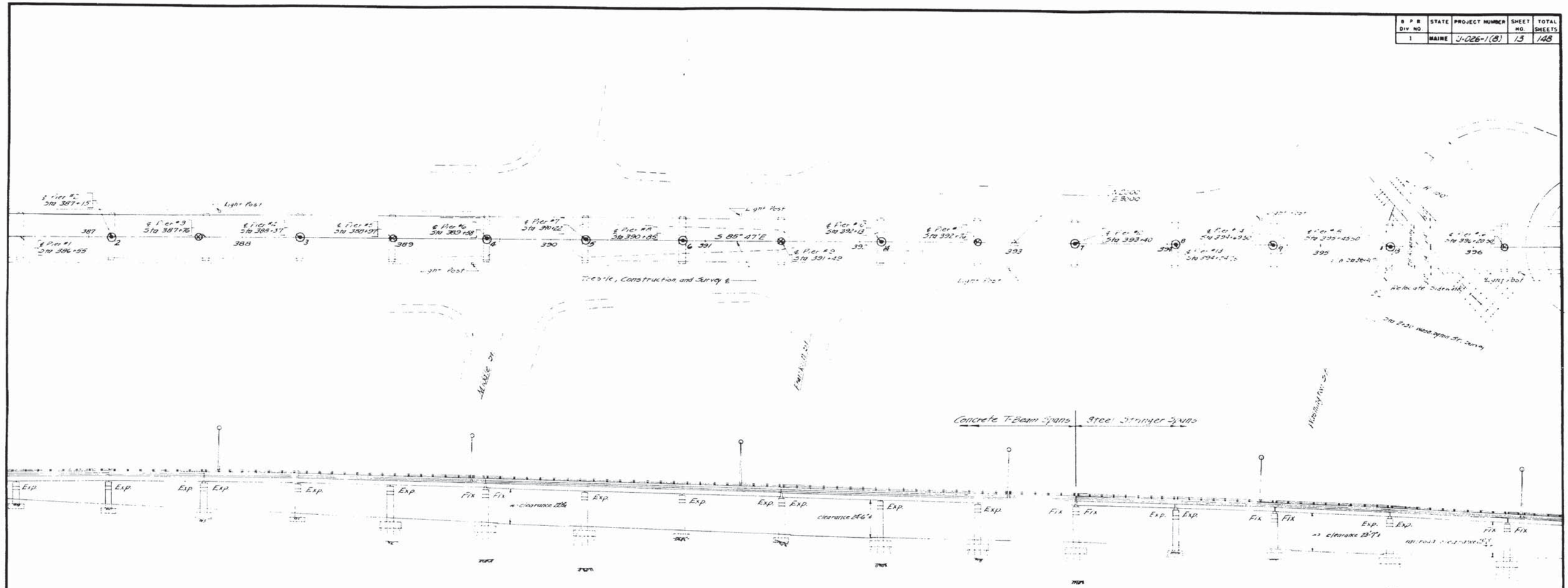
STATE HIGHWAY COMMISSION
BRIDGE DIVISION

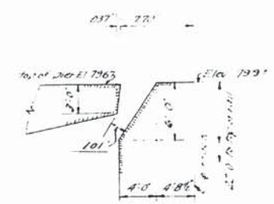
CARLTON BRIDGE WIDENING
AND
WEST APPROACH
BETWEEN
BATH & WOOLWICH
SAGADAHOC COUNTY, MAINE

ALIGNMENT AND PROFILE
STA. 379 TO STA. 387

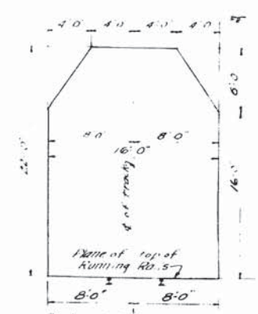
SHEET 12 OF 148 AUGUSTA, MAINE JAN. 1957

B P R DIV NO	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	J-026-1(8)	13	148

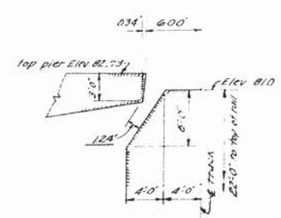




Railroad Clearance at Pier #16



Railroad Clearance Diagram



Railroad Clearance at Pier #15

DESIGN - EVERETT
TRACE - WENTZEL
CHECK - TURNER

WEST APPROACH

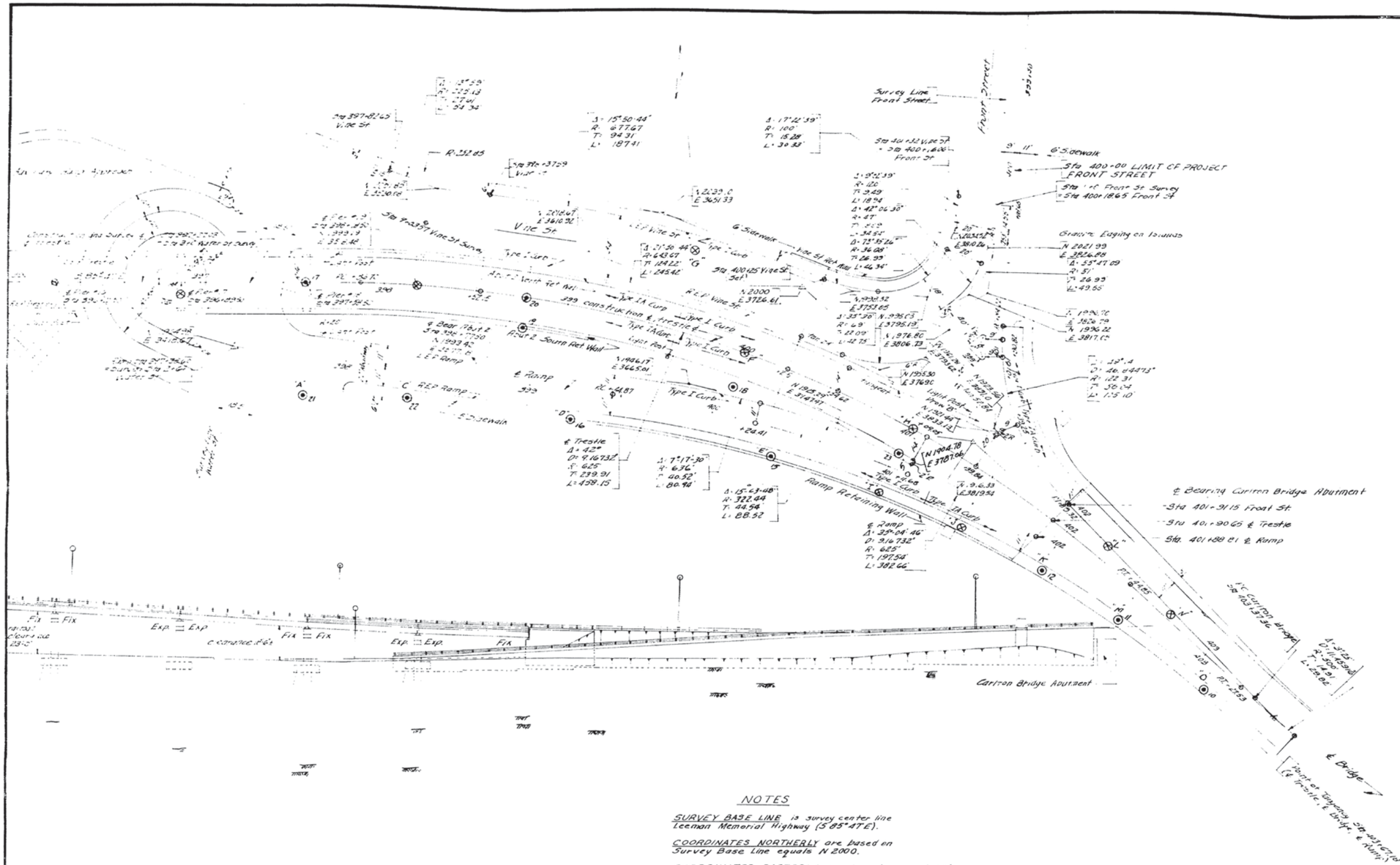
STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING
AND
WEST APPROACH
BETWEEN
BATH & WOOLWICH
SAGADAHOC COUNTY, MAINE

ALIGNMENT AND PROFILE
STA. 387 TO STA. 396

SHEET 13 OF 148 AUGUSTA, MAINE JAN. 1957

B P R Div NO	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	U-026-1(8)	14	148



NOTES

SURVEY BASE LINE is survey center line Leeman Memorial Highway (S 85°47' E).

COORDINATES NORTHERLY are based on Survey Base Line equals N 2000.

COORDINATES EASTERLY are based on line at right angles to Survey Base Line at Sta. 393+00 equals E 3000.

All curves are arc length.
Scale: 1 inch = 30 feet

Construction center line and center line of trestle are the same.

Construction center line and survey center line are the same to Sta. 397+06.70, P.C. of curve.

DESIGN - EVERETT
TRACE - WENTZEL
CHECK - TURNER

WEST APPROACH

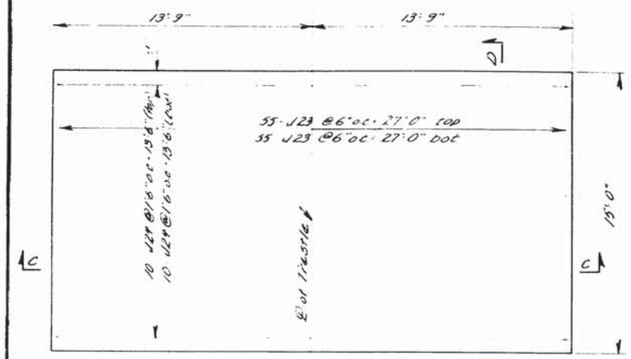
STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING
AND
WEST APPROACH
BETWEEN
BATH & WOOLWICH
SAGADAHOC COUNTY, MAINE

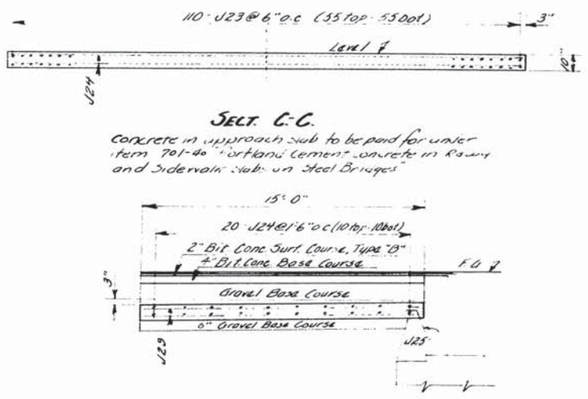
ALIGNMENT AND PROFILE
STA. 396 TO CARLTON BRIDGE

SHEET 14 OF 148 AUGUSTA, MAINE JAN. 1957

B P R DIV NO	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	U-026-1 (A)	33	148



PLAN - APPROACH SLAB



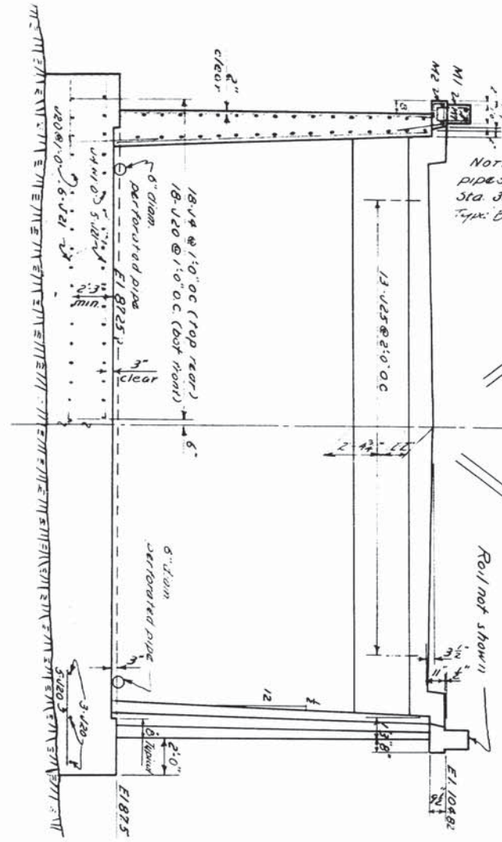
SECT. C-C.

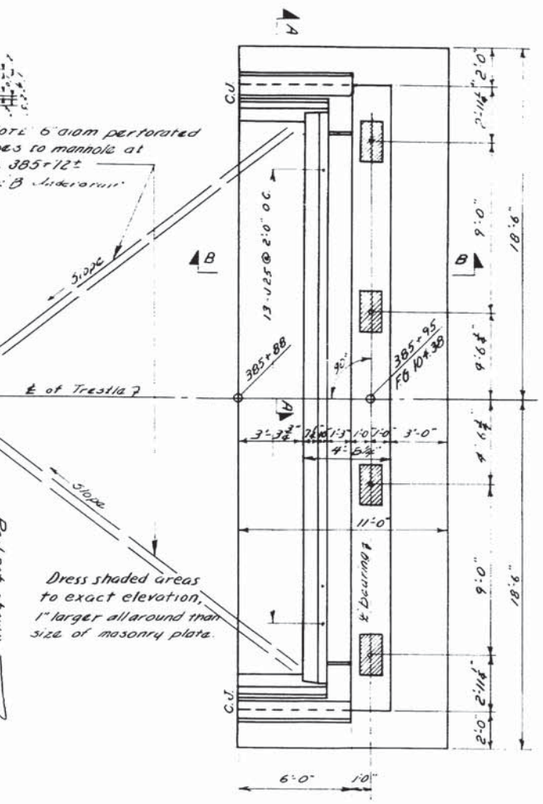
Concrete in approach slab to be paid for under item 701-40 "Portland Cement Concrete in Rdwy and Sidewalk Slab on Steel Bridges"

SECT. D-D.

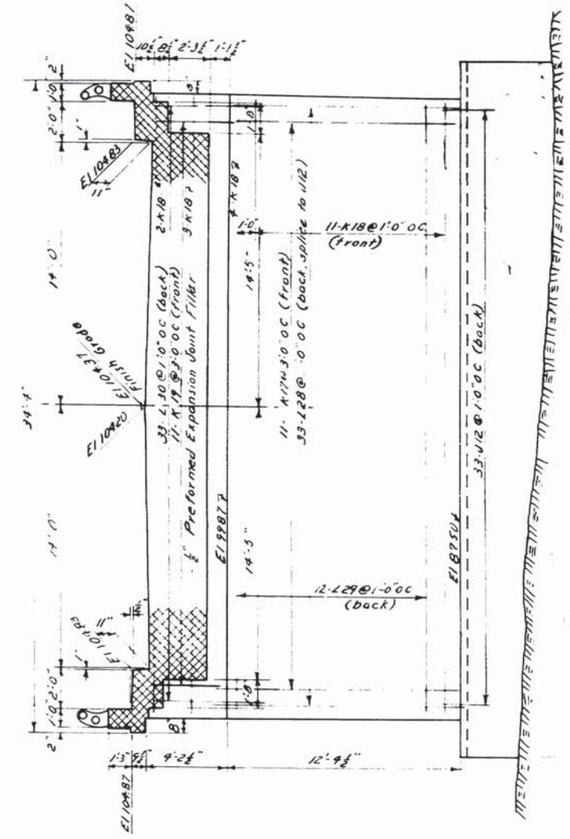
SECTION A-A

HALF REAR ELEVATION

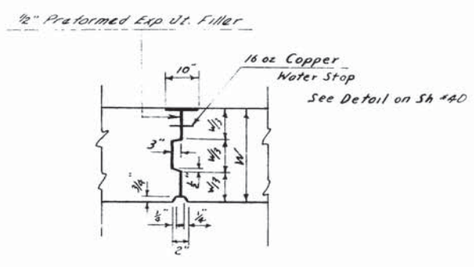




PLAN

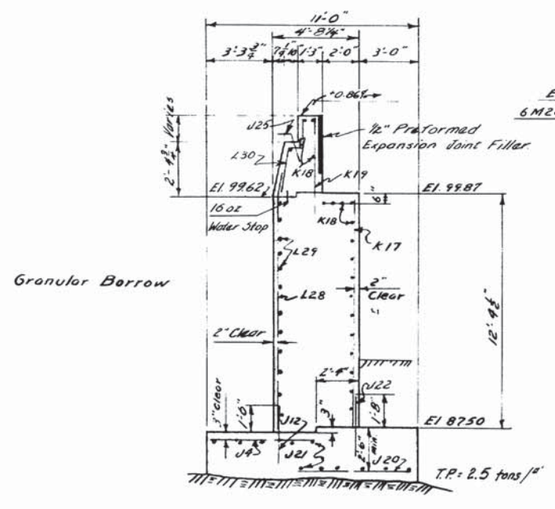


FRONT ELEVATION

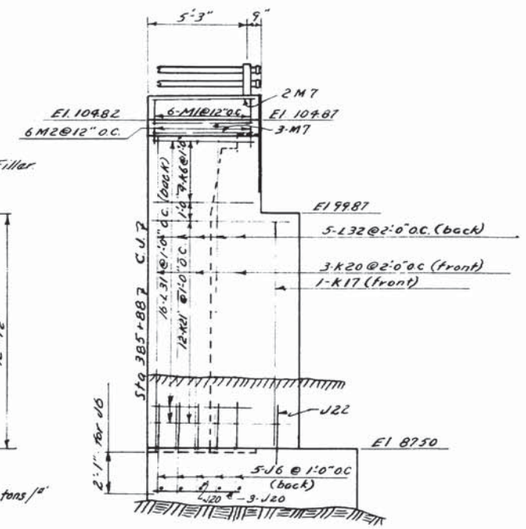


VERT GROOVE AND KEY AT CONSTRUCTION JOINTS

Cover constructruction joints on the back side with two layers of heavy roofing, 10" wide. Coat the surface of the concrete and the contact surface of roofing with a suitable grade of roofing cement as applied. The area to be covered shall be recessed 3/8". Do not construct Key in Coping and Rail Curb. Cover the horizontal construction joint on the rear of the breastwall at El. 99.62 in the same manner as described above. - Omit groove.



SECTION B-B



END ELEVATION

DESIGN - SHAILER
TRACE - E.V. SMITH
CHECK - [illegible]

WEST APPROACH

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING AND WEST APPROACH

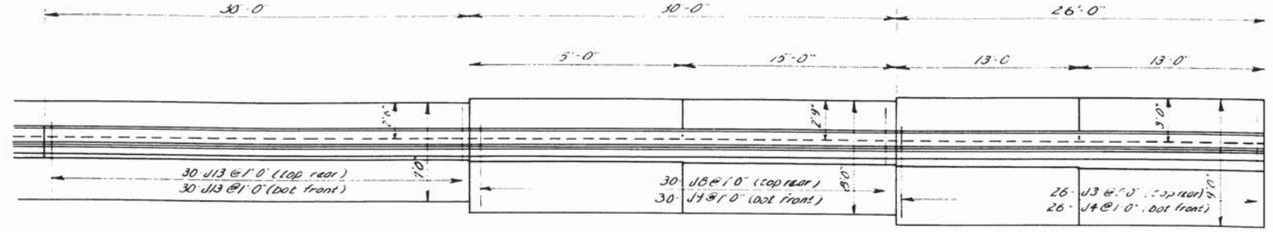
BETWEEN

BATH & WOOLWICH
SAGADAHOC COUNTY, MAINE

ABUT. NO. 1

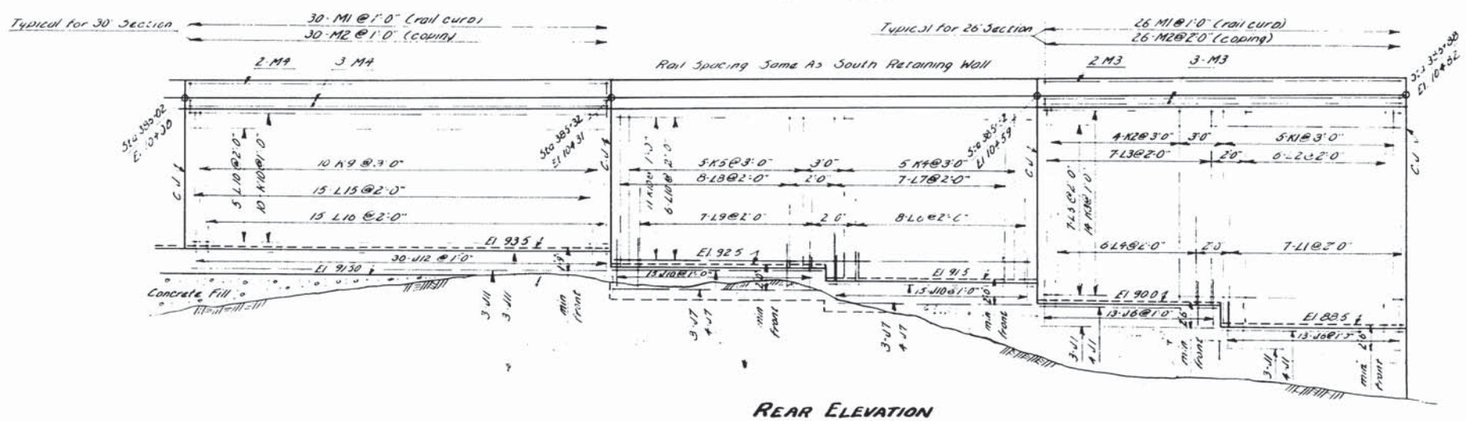
SHEET 33 OF 148 AUGUSTA, MAINE JAN. 1957

B P R DIV NO	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	U-026-1 (8)	34	148



PLAN

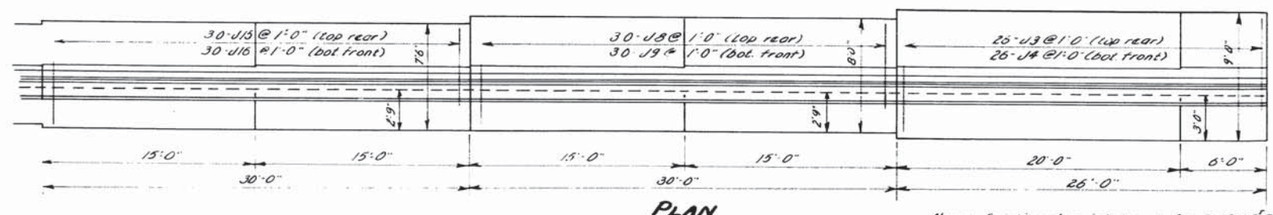
Rail not shown



REAR ELEVATION

NORTH RETAINING WALL

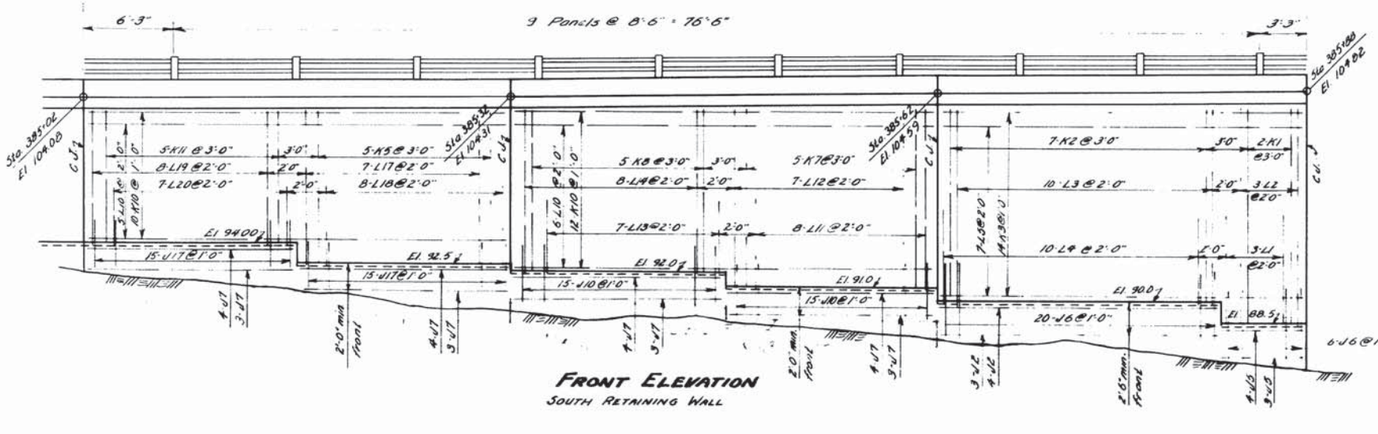
Rail not shown



PLAN

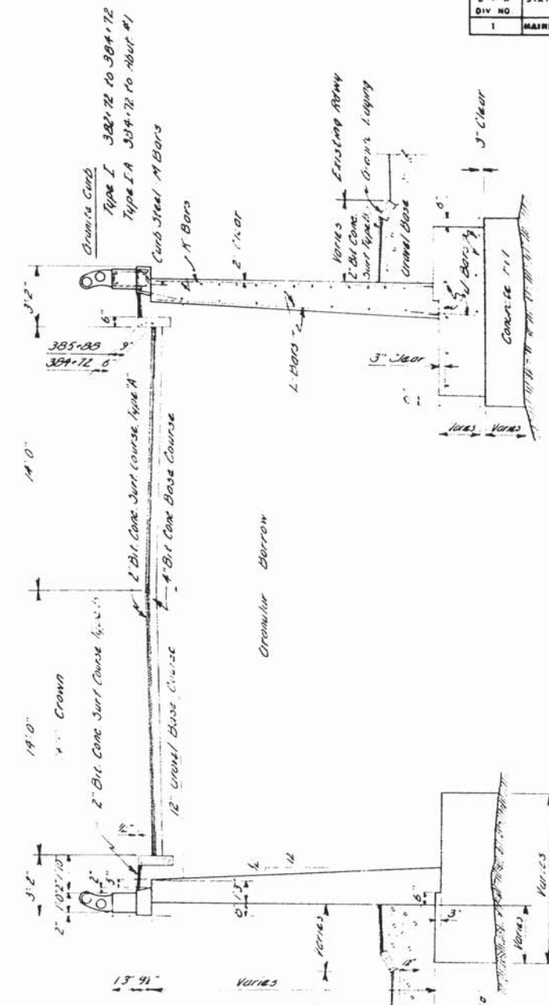
Rail not shown

Note: Existing drop inlets at Sta 385+65 Rt. and Sta. 385+89 Lt with connecting 12" R.C. Pipe to remain in place.



FRONT ELEVATION

SOUTH RETAINING WALL



TYPICAL TRANSVERSE SECTION

Note: See Sheet #40 for Construction Joint and Water Stop Details.

Key To Reinforcing Steel Location

J-Bars - Footing

L-Bars - Rear of Wall

K-Bars - Front of Wall

M-Bars - Rail Curb and Coping

DESIGN - SHAILER
TRACE - SHAILER
CHECK - Hamilton

WEST APPROACH

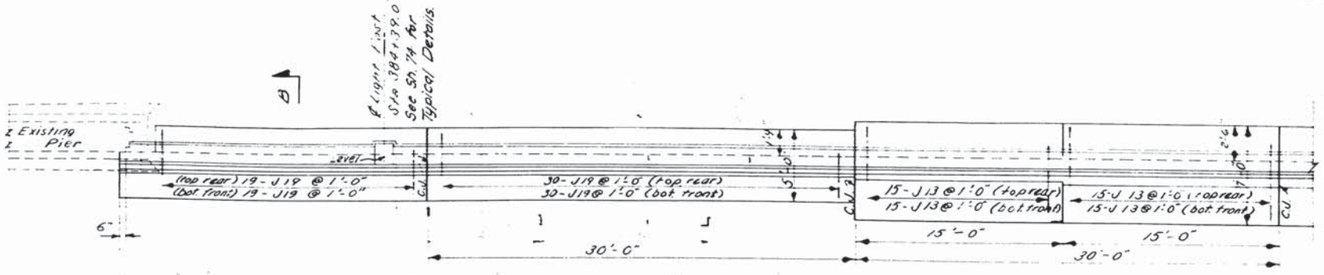
STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING AND WEST APPROACH BETWEEN BATH & WOOLWICH SAGADAHOC COUNTY, MAINE

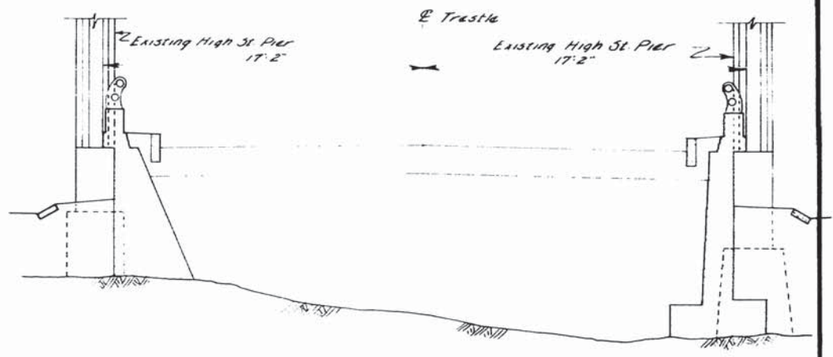
ABUT. NO. 1 RETAINING WALLS

SHEET 34 OF 148 AUGUSTA, MAINE JAN. 1957

B P R DIV NO	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	U-026-1 (8)	35	148

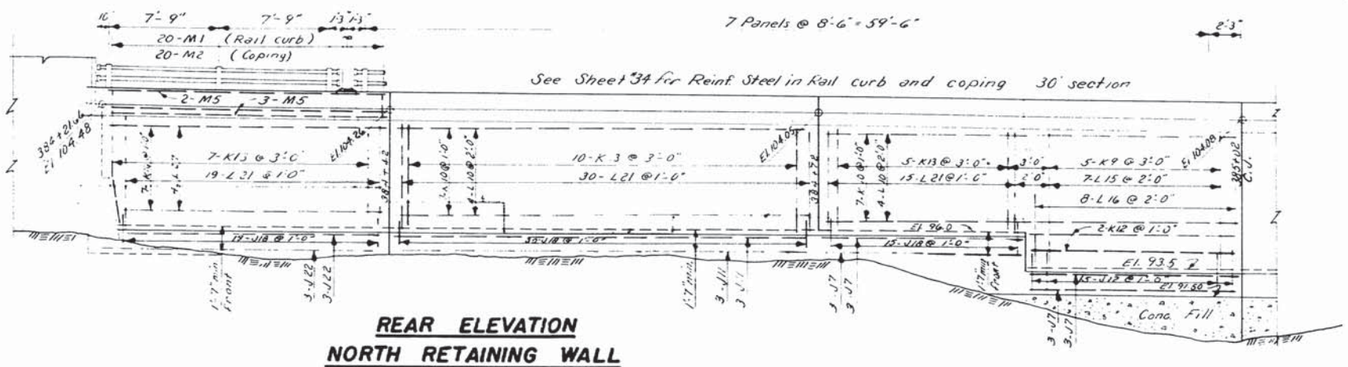


PLAN

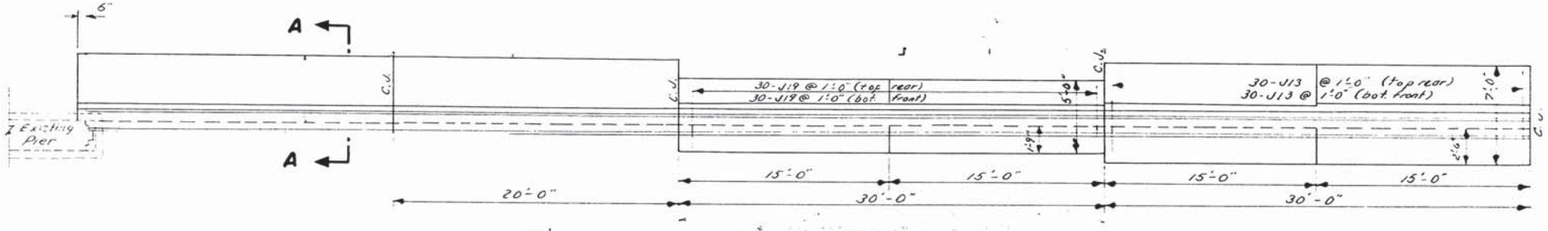


SECTION B-B

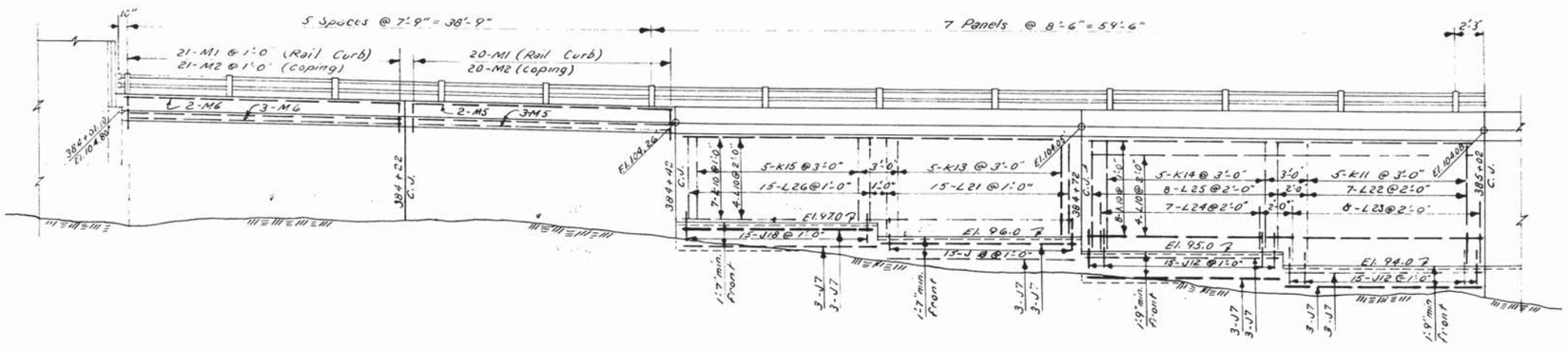
Note: Break the bond between the Existing Piers and the new concrete with a coating of roofing cement.



REAR ELEVATION
NORTH RETAINING WALL

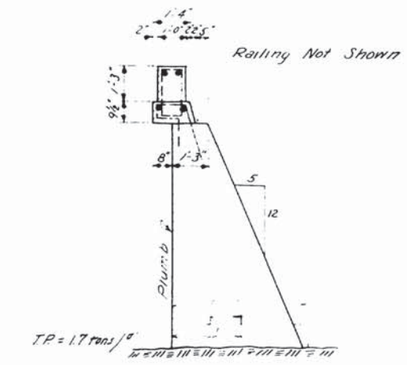


PLAN



FRONT ELEVATION
SOUTH RETAINING WALL

Note: Ledge to be excavated where necessary to obtain minimum depth footing.



SECTION "A-A"

Typical from Existing Pier to Sta. 384+42, South Retaining Wall only.

DESIGN - SHAILER
TRACE - C.M.
CHECK - Gamble

WEST APPROACH

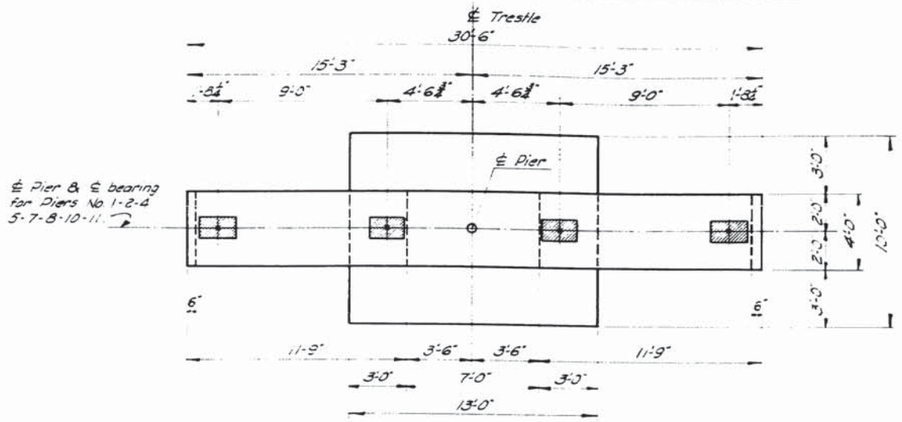
STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING
AND
WEST APPROACH
BETWEEN
BATH & WOOLWICH
SAGADAHOC COUNTY, MAINE

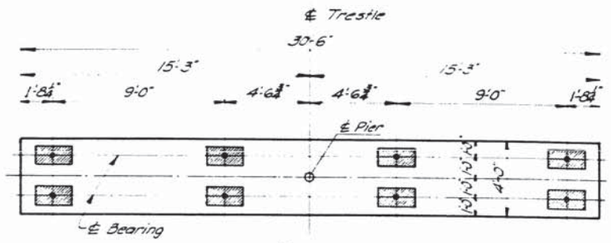
ABUTMENT NO. 1 RETAINING WALLS

SHEET 35 OF 148 AUGUSTA, MAINE JAN. 1957

S P R DIV NO	STATE	PROJECT NUMBER	SHEET NO	TOTAL SHEETS
1	MAINE	U-026-1 (8)	36	148



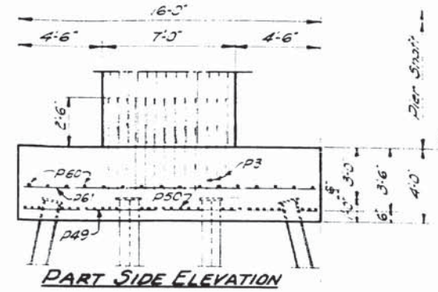
PLAN



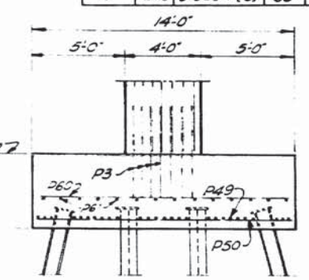
PLAN

Showing location of bearing plates for Piers No. 3-6-9.

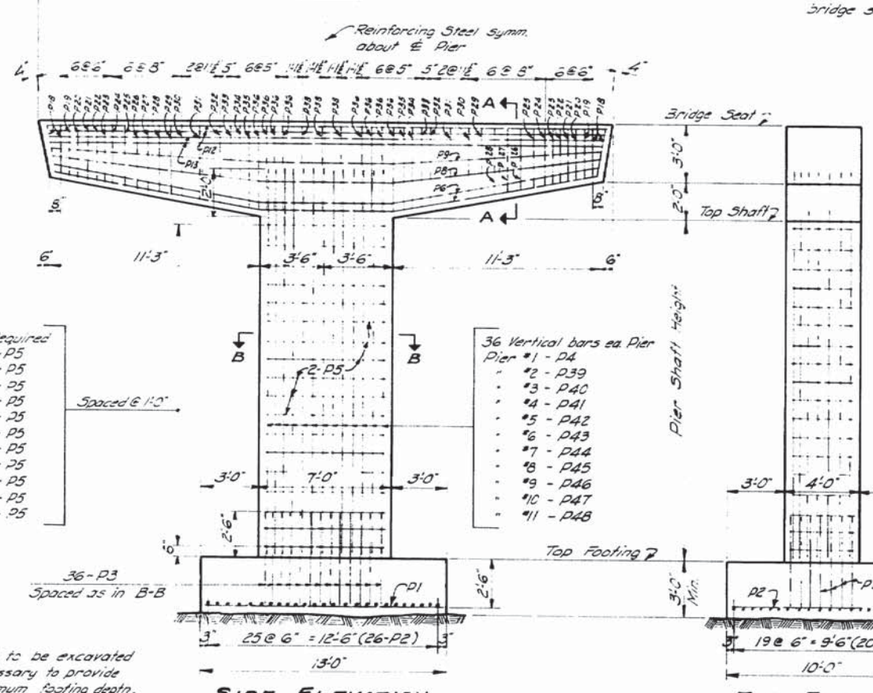
NOTE: Shaded bearing areas to be dressed 1" larger all around than masonry plates to exact bridge seat elevations.



PART SIDE ELEVATION



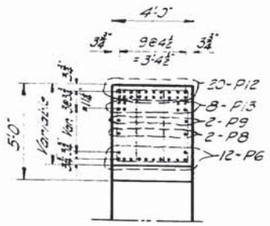
PART END ELEVATION



NOTE: Rock to be excavated if necessary to provide 3'-0" minimum footing depth.

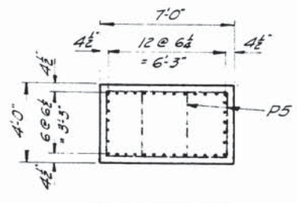
SIDE ELEVATION

END ELEVATION

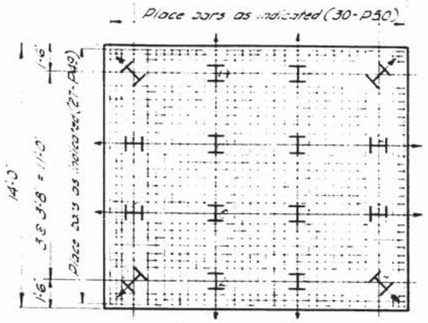


SECTION A-A

Place reinforcing steel in bridge seats to clear anchor bolts.



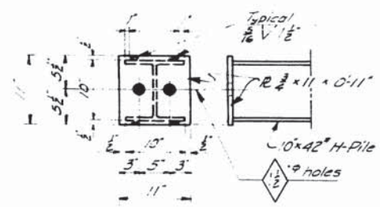
SECTION B-B



FOOTING PLAN

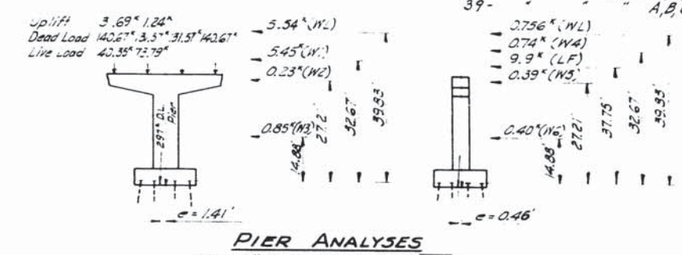
P60 & P61 @ 1'-0" ctrs 6" above top of piles

NOTE: Piles shown ⊢⊣ to be battered 3" ft in direction indicated by arrow.



PILE CAP DETAILS

16-Required for each pier
63- " " Walls O, P, Q, R, V.
39- " " " A, B, C, D, E, F.

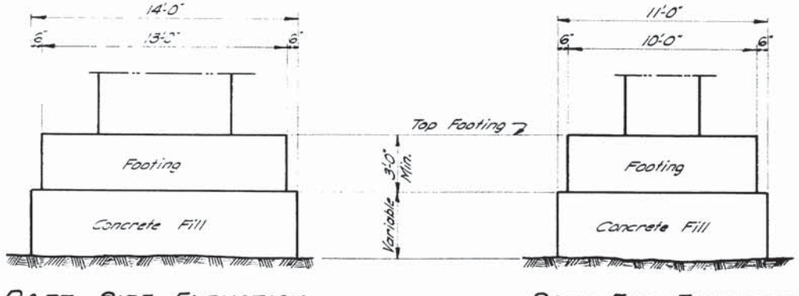


PIER ANALYSES

WL = Wind on moving Live Load
W1, W4 = Wind on superstructure
W2, W5 = Wind on pier cap
W3, W6 = Wind on pier shaft
L.F. = 5% of Live Load
— Group III Loading Wind Skew 15°

Maximum load on corner pile = 92.4ᴷ
Allowable load = 6ᴷ × 12.35 × 125% = 92.6ᴷ

NOTE: Pier footing on ledge.
Maximum pressure at corner = 13.7ᴷ



PART SIDE ELEVATION

PART END ELEVATION

Pier No.	Station ℄ Pier	Bridge Seat Elev.	Top Pier Shaft Elev.	Height of Pier Shaft	Top of Footing Elev.	Bottom of Footing Elev.	Footing Depth	Steel H-Piles 10×42# Req'd.	Pile cut-off Elev.	Est. Length of Piles	Remarks
1	386+55	100.28	95.28	9'-7"	85.69	82.69	3'-0"	—	—	—	Rock excavation required
2	387+15	100.45	95.45	12'-7"	82.86	79.86	3'-0"	—	—	—	" " "
3	387+76	100.22	95.22	14'-9"	80.47	77.47	3'-0"	—	—	—	Concrete fill required
4	388+37	99.61	94.61	16'-11"	77.69	74.69	3'-0"	—	—	—	Rock excavation required
5	388+97	98.63	93.63	18'-4"	75.29	71.29	4'-0"	16	72.29	15'-0"	16-Pile Caps required
6	389+58	97.38	92.38	19'-6"	72.88	68.88	4'-0"	16	69.88	25'-0"	16- " " "
7	390+22	95.86	90.86	19'-11"	70.94	66.94	4'-0"	16	67.94	30'-0"	16- " " "
8	390+85	94.45	89.45	20'-7"	68.86	65.86	3'-0"	—	—	—	Rock excavation required
9	391+49	93.01	88.01	21'-1"	66.92	63.92	3'-0"	—	—	—	" " "
10	392+13	91.58	86.58	21'-2"	65.41	61.41	4'-0"	16	62.41	20'-0"	16-Pile Caps required
11	392+76	90.16	85.16	21'-3"	63.91	59.91	4'-0"	16	60.91	15'-0"	16- " " "

DESIGN – HAMILTON
TRACE – CLARK
CHECK – A.B.P.

WEST APPROACH

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING AND WEST APPROACH

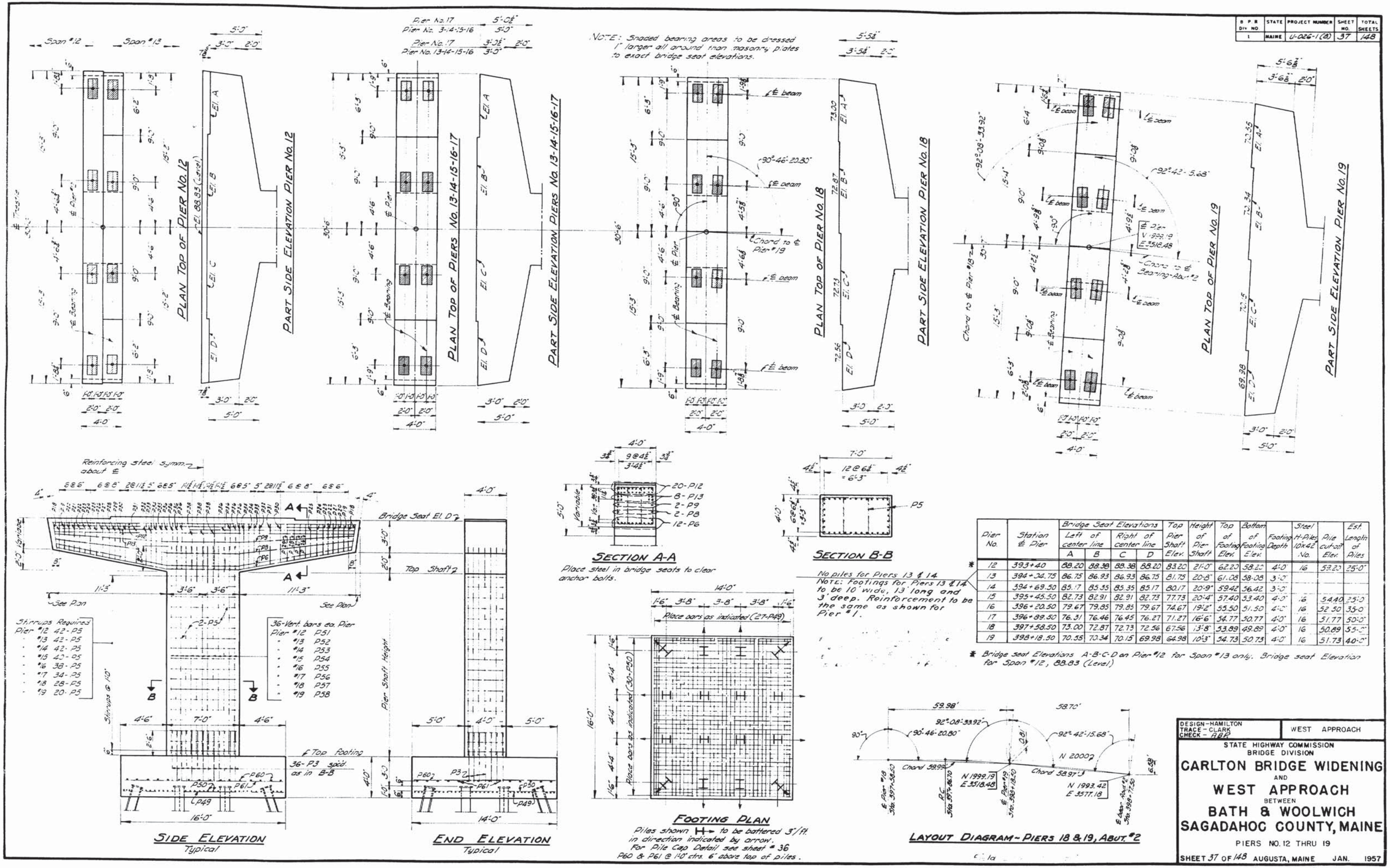
BETWEEN

BATH & WOOLWICH
SAGADAHOC COUNTY, MAINE

PIERS NO. 1 THRU 11

SHEET 36 OF 148 AUGUSTA, MAINE JAN. 1957

B.P.R. DIV. NO.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	U-026-1(8)	37	148



Pier No.	Station ℄ Pier	Bridge Seat Elevations Left of center line A	B	Right of center line C	D	Top Pier Shaft Elev.	Height of Pier Shaft	Top of Footing Elev.	Bottom of Footing Elev.	Footing Depth	Steel H-Piles 10x42 No.	Pile cut-off Elev.	Est. Length of Piles
* 12	393+40	88.20	88.38	88.38	88.20	83.20	21'-0"	62.20	58.20	4'-0	16	53.20	25'-0"
13	394+34.75	86.75	86.93	86.93	86.75	81.75	20'-8"	61.08	58.08	3'-0			
14	394+69.50	85.17	85.35	85.35	85.17	80.17	20'-9"	59.42	56.42	3'-0			
15	395+45.50	82.73	82.91	82.91	82.73	77.73	20'-4"	57.40	53.40	4'-0	16	54.40	25'-0
16	396+20.50	79.67	79.85	79.85	79.67	74.67	19'-2"	55.50	51.50	4'-0	16	52.50	35'-0
17	396+89.50	76.31	76.46	76.45	76.27	71.27	16'-6"	54.77	50.77	4'-0	16	51.77	50'-0"
18	397+58.50	73.00	72.87	72.73	72.56	67.56	13'-8"	53.89	49.89	4'-0	16	50.89	55'-0"
19	398+18.50	70.35	70.34	70.15	69.98	64.98	10'-3"	54.73	50.73	4'-0	16	51.73	40'-0"

* Bridge seat Elevations A-B-C-D on Pier #12 for Span #13 only. Bridge seat Elevation for Span #12, 88.83 (Level)

DESIGN - HAMILTON
TRACE - CLARK
CHECK - ABP

WEST APPROACH

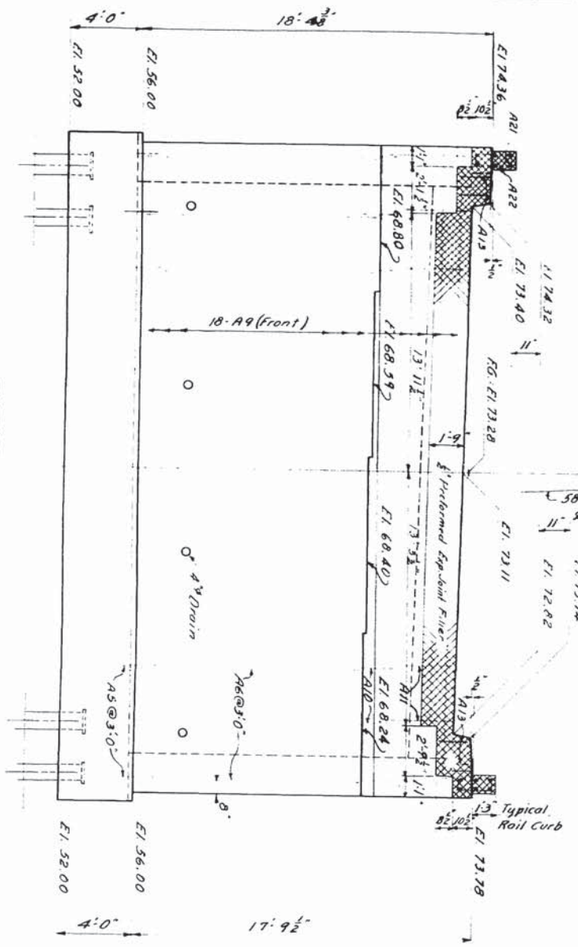
STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING AND WEST APPROACH BETWEEN BATH & WOOLWICH SAGADAHOC COUNTY, MAINE

PIERS NO. 12 THRU 19

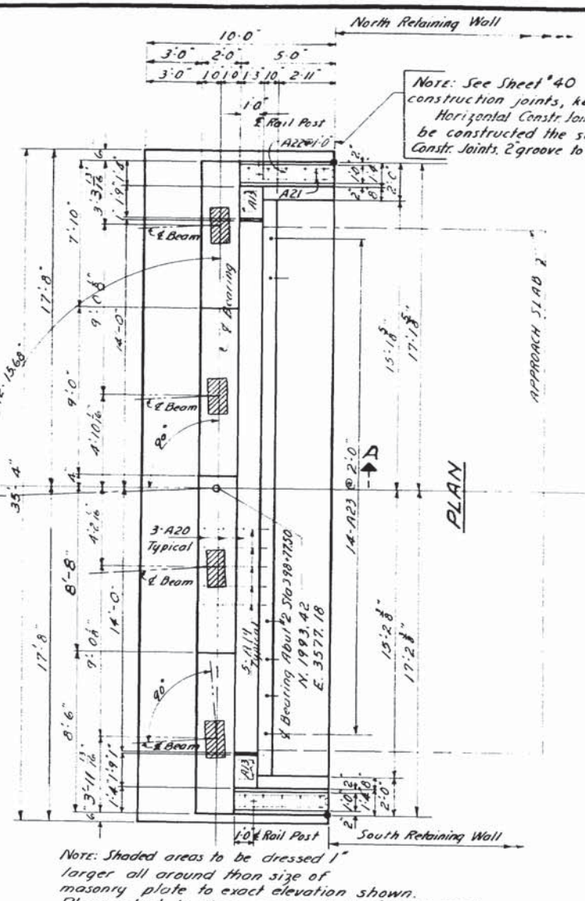
SHEET 37 OF 148 AUGUSTA, MAINE JAN. 1957

B P R Div. No.	STATE	PROJECT NUMBER	SHEET NO.	TOTAL SHEETS
1	MAINE	U-026-1 (8)	38	148



FRONT ELEVATION

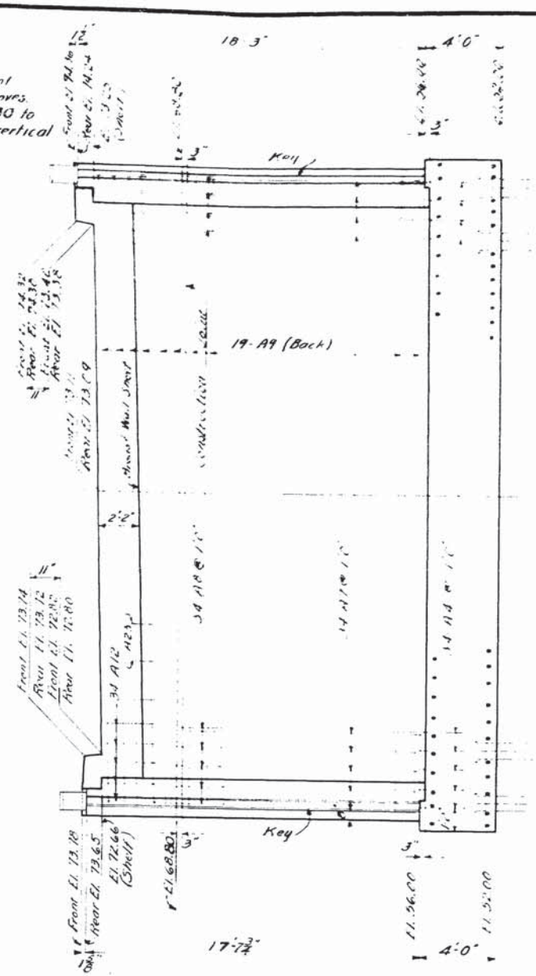
NOTE: Exact location of 4" drains to be determined by the Engineer in the field. Place stone conforming to Stone for French Drains, Special all abutment drains. To be paid for as Granular Backfill.



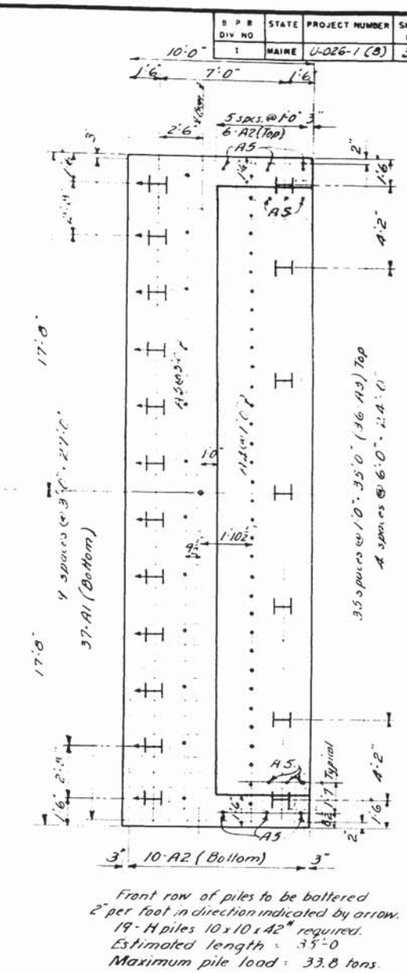
Note: See Sheet #40 for detail of construction joints, key and 2" grooves. Horizontal Constr. Joint at El. 68.80 to be constructed the same as the vertical Constr. Joints. 2" groove to be omitted.

PLAN

Note: Shaded areas to be dressed 1" larger all around than size of masonry plate to exact elevation shown. Place steel to clear anchor bolts (A19 & A20)



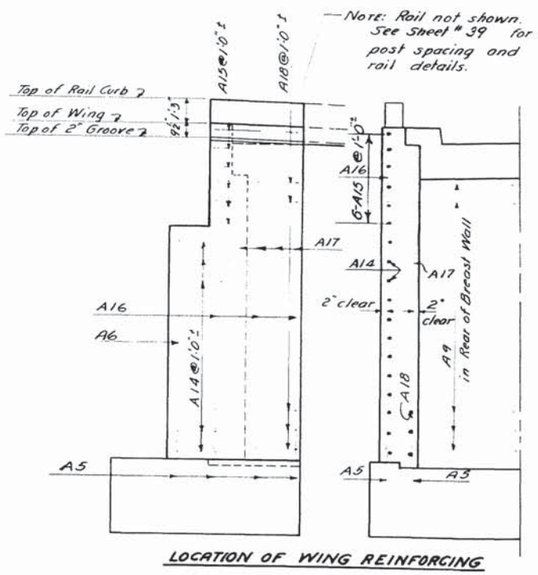
REAR ELEVATION



FOOTING PLAN-PILE AND STEEL LAYOUT

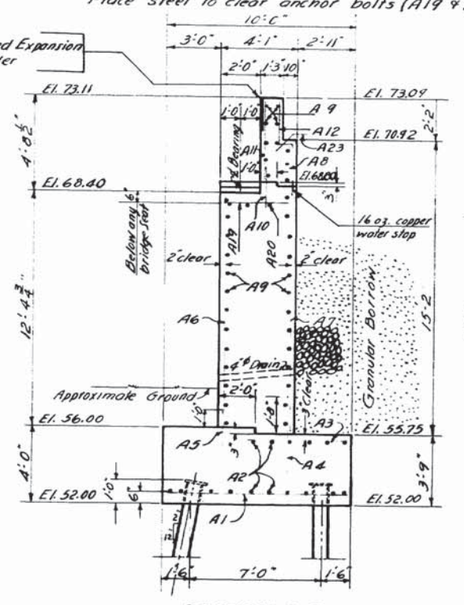
Bend Alinere necessary to clear rail bars.

Front row of piles to be battered 2" per foot in direction indicated by arrow.
19-H piles 10x10x42# required.
Estimated length = 35'-0
Maximum pile load = 33.8 tons.



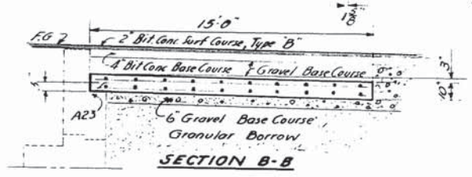
Note: Rail not shown. See Sheet # 39 for rail post spacing and rail details.

LOCATION OF WING REINFORCING

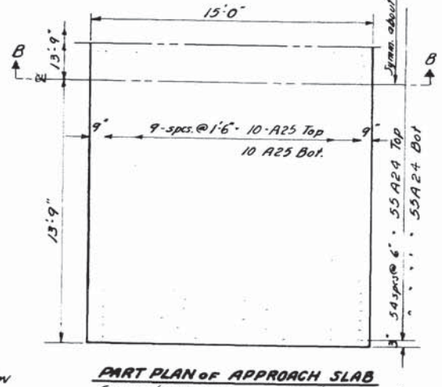


SECTION A-A

Note: Bars A19 & A20 to be placed 6" below bridge seat elevation at beam bearing areas.

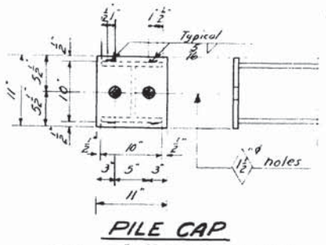


SECTION B-B



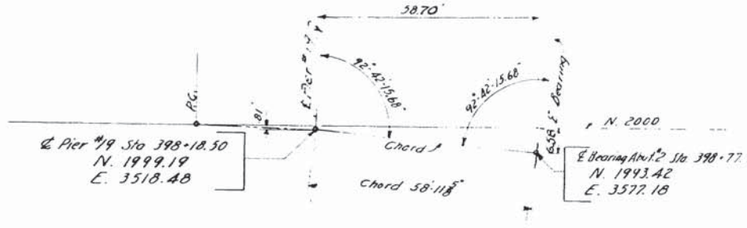
PART PLAN OF APPROACH SLAB

Concrete in approach slab to be paid for under Item 701-40 "Portland Cement Concrete in Roadway & Sidewalk Slabs on Steel Bridges"



PILE CAP

Plate 11" x ¾" x 0'-11" - 19 req'd.



LAYOUT ABUTMENT NO. 2

DESIGN - HAMILTON
TRACE - SAVAGE
CHECK -

WEST APPROACH

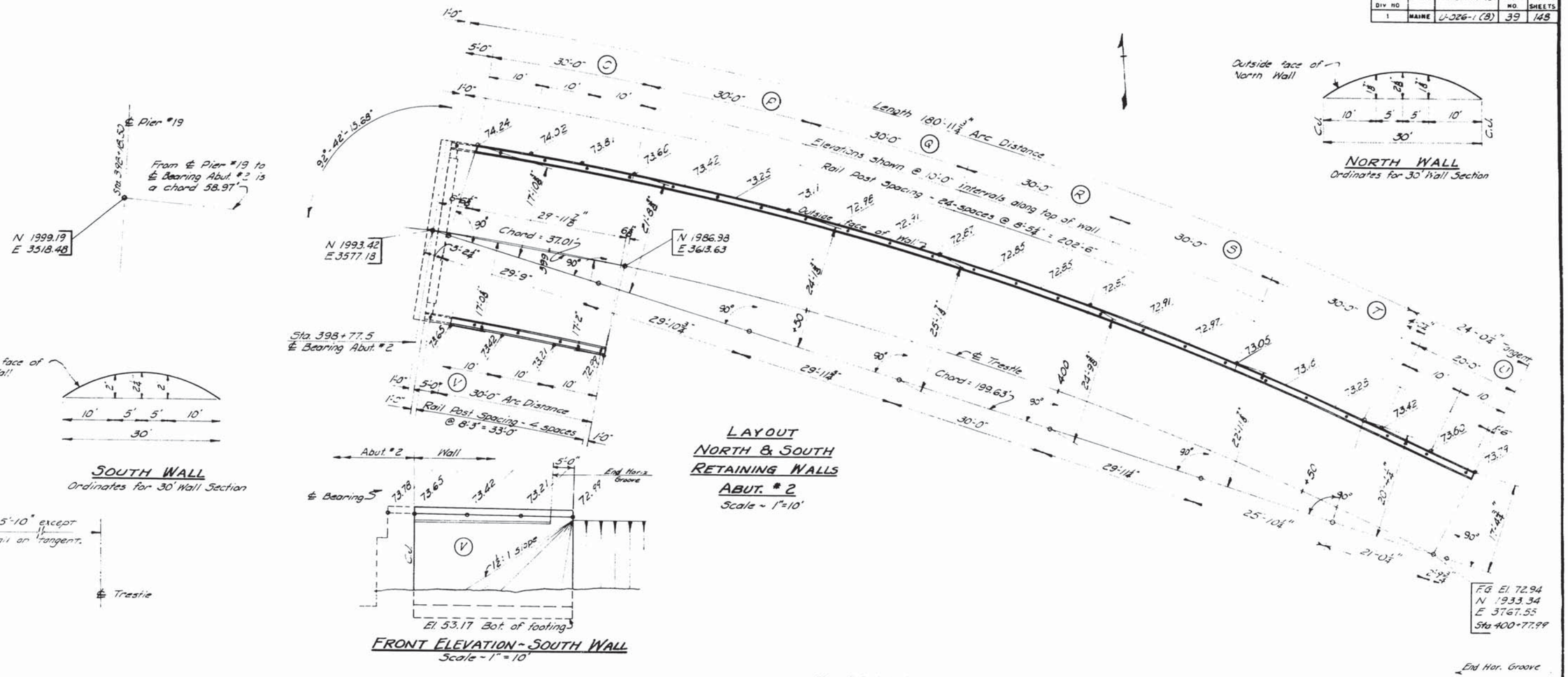
STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING AND WEST APPROACH BETWEEN BATH & WOOLWICH SAGADAHOC COUNTY, MAINE

ABUTMENT NO. 2

SHEET 38 OF 148 AUGUSTA, MAINE JAN. 1957.

B P R DIV NO	STATE	PROJECT NUMBER	SHEET NO	TOTAL SHEETS
1	MAINE	U-326-1 (8)	39	148



DESIGN - HAMILTON
TRACE - CLARK
CHECK - [illegible]

WEST APPROACH

STATE HIGHWAY COMMISSION
BRIDGE DIVISION

CARLTON BRIDGE WIDENING
AND
WEST APPROACH
BETWEEN
BATH & WOOLWICH
SAGADAHOC COUNTY, MAINE

NORTH AND SOUTH RETAINING WALLS
ABUTMENT NO. 2

SHEET 39 OF 148 AUGUSTA, MAINE JAN. 1957

As-Built Pile Documentation

CALCULATIONS

STATE HIGHWAY COMMISSION, AUGUSTA, MAINE

Book 9 of 9

Town Bath-Woolwich, Carlton Bridge Widening and West Approach Bridge

Contractor (s) _____

Engineer (s) _____ See Book 1

Work started _____ Suspended _____

Resumed _____ Completed _____

General index, Page 1 of Book 1

Field changes, Page 137 of Book 9

Materials used in project, Page 141 of Book 9

GENERAL NOTES

Notebooks should contain the following:

Layout of project, date & name of participants.

Computations for final quantities on all contract items. Extensive computations may be made on letter size paper, the sheets numbered and references made in notebook under the proper item that they are in the letter file.

Records of all samples and tests, noting source (manufacturer, name of pit and city or town).

Record of materials actually used in project. Note if sand or gravel was washed.

Account and description of all work done on a "Force Account Basis".

Location of right of way markers, if placed.

Recommended concrete mixes and mixes used.

Description and sketches of changes in plans.

If project is suspended, record work to be completed when work is resumed.

Disposal of salvaged material from old bridge.

NOTEBOOKS ARE TO BE PROMPTLY TURNED IN TO THE

COMPUTATION of QUANTITIES

No.	Item	Quantity Rdwy.	Quantity W.A.	Quantity CBW	Price & Unit	Est. Quant.	Final Quant.	Final Amount	Over or Under	Calcul. & details
202-5	Rem. trees, 9-24	9			$50.00 ea.	7	9	450.00	+100.00	B5, P8
202-6	Rem. trees, over 24	1			100.00 ea.	2	1	100.00	−100.00	B5, P8
203-9	Earth excav.	1447			1.00 cy	2230	1447	1447.00	−783.00	B-9, P49
203-10	Rock excav.	365			10.00 cy	270	365	3650.00	+950.00	B-9, P50
204-10	St. ca. excav., dr.	458			1.50 cy	500	458	687.00	−63.00	B9, P38
204-11	St. rock excav., dr.	81			15.00 cy	55	81	1215.00	+390.00	B9, P39
204-12	St. ca. exc., abuts	2917	249	137	1.25 cy	2680	3303	4128.75	+778.75	B-9, P30
204-13	St. rock exc., abuts	175	24		10.00 cy	302	199	1990.00	−1030.00	B9, P41
204-14	St. ca. exc., piers		1171	155	1.25 cy	1340	1326	1657.50	− 17.50	B-9, P35
204-15	St. rock exc., piers		152	13	12.00 cy	89	165	1980.00	+912.00	B9, P43
205-9	Granular borrow	8360	1000	300	1.00 cy	6365	9660	9660.00	+3295.00	B9, P55
302-7	Gravel base	2636			1.50 cy	3160	2636	3954.00	−524.00	B9, P62
304-9	Bit. conc. base	1665			14.00 ton	1740	1665	23,310.00	−1050.00	B-9, P127
306-8	Levelling course			5	14.00 ton	4	5	70.00	+ 14.00	B-9, P127
308-5	Overhaul, in pl.	18,715			0.10 sy m	15,700	18,715	1871.50	+301.50	B9, P61
308-6	Overhaul, pit	27,872	3200	960	0.08 sy m	31,900	32,032	2562.56	+10.56	B9, P55
311-6	Calcium chloride	5.55			100.00 ton	5	5.55	555.00	+55.00	B5, P10
404-11	Gravel surface	0			10.00 cy	25	None			
404-28	Bit. conc. surf. "A"	900	400	900	14.00 ton	2285	2200	30,800.00	−1190.00	B-9, P127
404-29	Bit. conc. surf. "B"	400			14.00 ton	400	400	5600.00	—	B-9, P127
506-6	Tack coat	5		175	0.40 gal	1165	180	72.00	− 394.00	B-9, P-127
506-7	Membrane waterproof		3963		2.00 s.y.	3960	3963	7926.00	+ 6.00	B9, P-125
603-10	12" RCP pipe	484			3.50 l.f.	460	484	1694.00	+84.00	B-4, P144

GENERAL SUMMARY

No	Item	Quantity Rdwy	Quantity W.A.	Quantity C.B.W.	Price & Unit	Est. Quant.	Final Quant.	Final Amount	Over or Under	Calcul. & Details
605-12	Drop inlet, Type A	1			$300.00/ea	1	1	300.00	—	B-6, P52
605-14	Catch basins, Type A	2			350.00/ea.	2	2	700.00	—	B5, P52
605-16	Catch basins, Type C	13.25			300.00/ea.	13	13.25	3975.00	+75.00	do
605-19	Manholes, Type A	2.375			350.00/ea	2	2.375	831.25	+131.25	do
605-22	12" Traps	2			70.00/ea	2	2	140.00	—	do
605-23	Adj. DI, CB, & MH.	6			70.00/ea	6	6	420.00	—	do
606-10	Underdrain, Type S	139			2.50/l.f.	100	139	347.50	+97.50	B-9, P-123
606-11	12" Underdr., Type C	285			5.00/l.f.	280	285	1425.00	+25.00	B-9, P-123
701-33	PCC. abuts. + retwls	1559	278	139	54.00/cy	1901	1976	106,704.00	+4050.00	B-6, P-17
701-35	PCC. piers		1224	137	62.00/cy	1345	1361	84,382.00	+992.00	B-6, P-73
701-40	PCC. rdwy & SW. slabs		571	937	79.00/cy	1510	1508	119,132.00	-158.00	B-6, P-85
701-41	PCC. superstr.		1348		85.00/cy	1369	1348	114,580.00	-1785.00	B-6, P-82
701-47	Portland cement	2765	5292	1845	6.00/bbl	9310	9902	59,412.00	+3552.00	B-7, P-32
701-50	Concrete fill	295	66		22.00/cy	44	361	7942.00	+6974.00	B-6, P-132
702-103A	Str. steel, del., W.A.		547,400		0.19/lb.	544,300	547,400	104,006.00	+589.00	B-9, P-63
702-103B	Str. steel, del., CBW.			114,358	0.34/lb.	111,700	114,358	38,881.72	+903.72	B-9, P-69
702-104A	Str. steel, erect., W.A.		547,400		0.04/lb.	544,300	547,400	21,896.00	+124.00	B-9, P-63
702-104B	Str. steel, erect., CBW.			112,492	0.39/lb	111,700	112,492	43,871.88	+308.88	B-9, P-69
702-105	Carlton Br. rail			100%	22,500.00/LS.	100%	100%	22,500.00	—	B-9, P129
703-9	Exp. plates, del.		1030	104	2.30/lb.	1140	1134	2608.20	-13.80	B-9, P-63,69
703-10	Exp. plates, pla.		1030	104	1.00/lb.	1140	1134	1134.00	-6.00	B-9, P-63,69
705-13	Reinf. steel, del.	134,453	788,683	178,347	0.10/lb	1,121,000	1,101,463	110,146.30	-1953.70	B-9, P-77
705-14	Reinf. steel, pla.	134,453	788,683	178,347	0.0325/lb	1,121,000	1,101,463	35,797.55	-634.95	B-9, P-77
705-17	Shear connectors		90%	10%	2500.00/LS.	100%	100%	2500.00	—	B-9, P-129

No	Item	Quantity Rdwy.	Quantity W.A.	Quantity C.B.W.	Price & Unit	Est. Quant.	Final Quant.	Final Amount	Over or Under	Calcs & details
708-16	St. H-beam piles	4662	5551		$8.00/l.f.	10,560	10,249	81,992.00	-2488.00	B-9-P122
801-8	Rem. ex. conc. substr.	15		9	20.00/cy	22	24	480.00	+40.00	B-9-P116
801-9	Rem. ex. conc. superstr.			621	29.50/cy	675	621	18,319.50	-1593.00	B-9, P-112
801-10	Rem. ex. conc. rail	196			4.00/l.f.	190	196	784.00	+24.00	B-9, P-118
801-11	Rem. bit. conc. surf.	174		7048	1.75/sy.	7265	7222	12,638.50	-75.25	B-9, P-124
802-10	Maint. of traffic			100%	58,000.00/LS	100%	100%	58,000.00	—	B-9, P129
805-8	Drainage, superstr.		100%		31,000.00/LS.	100%	100%	31,000.00	—	B-9, P-129
805-9	Aluminum rail	1101	2555	173	8.00/l.f.	3829	3829	30,632.00	—	B5-P53
805-10	Pipe rail	72.2			12.00/l.f.	133	72.2	866.40	-729.60	B5-P53
901-8	Granite curb Ty. I	2214			4.50/l.f.	2200	2214	9963.00	+63.00	B5-P50
901-9	Gran. curb, Ty. I cir.	149			6.00/l.f.	130	149	894.00	+114.00	B5-P50
901-14	Gran. edging	2622			3.30/l.f.	2640	2622	8652.60	-59.40	B5-P48
901-15	Gran. edging, cir.	158			6.00/l.f.	165	158	948.00	-42.00	B5-P49
901-16	Gran. curb, Ty. 1A	509			5.50/l.f.	520	509	2799.50	-60.50	B5-P51
905-29	Guard rail, Type G	175			4.00/l.f.	175	175	700.00	—	B5-P60
905-34	End wings	4			10.00/ea.	4	4	40.00	—	B5-P60
905-37	Guard posts, Type G	18			15.00/ea.	18	18	270.00	—	B5-P60
908-9	Loam borrow	148			5.00/cy	50	148	740.00	+490.00	B9-P126
909-7	Sodding	439			1.00/sy.	325	439	439.00	+114.00	B-5, P-58
910-10	Seeding, pkwy. mix	5.5			50.00/un.	1	5.5	275.00	+225.00	B5-P56
914-6	Proj. markers	1			25.00/ea.	2	1	25.00	-25.00	B5-P60
915-6	R/W monuments	1			20.00/ea.	10	1	20.00	-180.00	0.0
917-6	Traffic officers			13,313.5	2.00/m.h.	7000	13,313.5	26,627.00	+12,627.00	B-8 all
927-1	Lighting system	13.42%	23.42%	63.16%	43,573.00/LS	100%	100%	43,573.00	—	B-9, P.129

No.	Item	Quantity Rdwy.	Quantity W.A.	Quantity C.B.W.	Proc. & Unit	Est. Quant.	Final Quant.	Final Amount	Over or Under	Calcul. & details
927-2	Pylon caps	4			$350.00/ea.	4	4	1400.00	—	B-9, P-130
927-3	Rais. & reloc. pylons	100%			600.00/LS.	100%	100%	600.00	—	B-9, P-130
927-4	Light sys., stairs	100%			850.48/LS.	100%	100%	850.48	+850.48	B-9, P-130
701-51	P.C.C. stairs + plat.	22.9			105.00/cy	23	22.9	2404.50	+2404.50	B-9, P-136
805-11	Stair rail, 2-bar	47.8			13.16/l.f.	47	47.8	628.57	+628.57	B-5, P-59
805-12	Stair rail, 3-bar	91.9			18.25/l.f.	92	91.9	1677.18	+1677.18	B-5, P-59
	Filling station & tank removal	100%			2000.00/LS.	—	100%	2000.00	+2000.00	B-9, P-131
	Pile cut-offs	300	359.58		3.00/71/l.f.	—	659.58	1983.42	+1983.42	B-5, P-58
	Relaying sewer at Pier 12		100%		—	—	—	994.13	+994.13	B-9, P-131
	Lowering sewer + water lines at Drake's Ins.	100%			—	—	—	143.10	+143.10	B-9, P-132
	Relocating Light #26			100%	—	—	—	468.21	+468.21	B-9, P-132
	Adjusting counterweights			100%	—	—	—	1500.44	+1500.44	B-9, P-133
	Widening temporary bridge			100%	—	—	—	5600.00	+5600.00	B-9, P-133
	Waterproofing Carlton Bridge			300	20.00/gal.	—	300	6000.00	+6000.00	B-9, P-134
	Flashing, bobbers & sign anchor bolts	100%			—	—	100%	1380.00	+1380.00	B-9, P-134
	Vine street curb extension	100%			—	—	—	2921.06	+2921.06	B-9, P-135
	Total Roadway							$301,218.15	$+33,440.90	
	Total West Approach							646,881.72	−5411.08	
	Total Carlton Bridge Widening							402,142.43	+23,469.73	
	Grand Total							$1,350,242.30	+51,499.55	

Steel H-beam Piles - 112# / ft.

Str	No. Piles	Length Range	Total Length	Caps	Total Pay length
Pier #5	12	9.3 - 13.9	128.2	12	164.2
Pier #6	16	9.3 - 28.0	311.1	16	359.1
Pier #7	16	15.7 - 37.2	388.2	16	424.2
Pier #10	16	9.8 - 18.7	215.1	16	263.1
Pier #11	8	5.3 - 16.0	58.1	8	82.1
Pier #12	18	15.0 - 22.5	339.9	18	393.9
Pier #15	16	8.8 - 44.0	282.9	16	330.9
Pier #16	16	28.1 - 44.0	546.7	16	594.7
Pier #17	16	34.3 - 87.0	745.2	16	793.2
Pier #18	16	34.7 - 63.7	795.7	16	843.7
Pier #19	16	31.3 - 46.2	601.7	16	649.7
Ramp Ret Wall					
Sect "A"	4	53.2 - 55.8	219.7	4	231.7
" B	7	55.0 - 62.2	403.2	7	424.2
" C	7	54.4 - 65.4	415.4	7	436.4
" D	7	44.7 - 66.3	396.1	7	417.1
" E	7	33.0 - 50.1	283.3	7	304.3
" F	7	31.3 - 41.8	261.8	7	282.8
" G	11	28.0 - 41.1	397.3	11	430.3
" H	11	19.9 - 32.0	283.7	11	316.7
" I	10	8.5 - 19.0	139.3	10	169.3
Abut #2	19	24.7 - 44.8	630.8	19	687.8

Steel H-Beam Piles - 42#/ft.

Str	No. Piles	Length Range	Total Length	Caps	Total Pay Length
Abut #2 Ret. Wall					
Sect O	16	22.0-28.6	402.3	16	450.3
Sect P	11	25.3-27.9	291.5	11	324.5
Sect Q	11	9.3-28.4	189.6	11	222.6
Sect R	11	6.7-18.5	123.3	11	156.3
Sect V	16	23.2-33.7	417.9	16	495.9
			Total for Units		10,249.00
		Total West Approach	5586.6		
		Total Roadway	4662.4		
		Total Item	10,249.0		

Total Length

164.2
259.1
424.2
263.1
82.1
393.9
330.9
594.7
793.2
843.7
649.7

231.7
424.2
436.4
417.1
304.3
282.8
430.3
316.7
169.3
687.8

- Field Changes. -

- Abutment #1 Retaining Walls.

The section of the north wall between sta. 384+20± and 384+87 and the south wall between sta. 384+42 and 384+72 was changed to a gravity mass section like that of the south wall west of Sta 384+42. This was done to eliminate excavation of up to 4' of ledge, which would have been necessary in order to keep the footings below ground. Due to the low height of wall required it resulted in no essential change in volume of concrete.

At section between 384+87 and 385+88 of the north wall and between sta 385+32 and 385+62 of the south wall ledge was found up to 7' below the footings. The overburden consisted of silty clay and sand and as there was evidence of ground water and resulting instability, it was removed and concrete fill placed to the designed footing elevations. This required 166 cy. of concrete fill.

- Piers.

Footing of Pier 3 was raised 3" to clear existing concrete pipe.
Four piles in Pier 5 were replaced with concrete fill where the ledge was too high to drive piles.
Eight piles in Pier 11 were replaced with concrete fill where ledge was 1' to 4' below the footing.
Footing of Pier 14 was raised 6" to eliminate need to excavate ledge which would without doubt result in overbreakage.

- Ramp Retaining Wall.

At Walls G, H, I and J the existing soil was found unsatisfactory to support these walls. The footings of Walls G, H, and I were thickened to 3 FT. and piles driven to ledge.
Pile arrangement was the same as walls of equal height elsewhere on project.
Wall J, which would have been partly on ledge at the designed footing elevation was founded on concrete fill on ledge.
45 cy of concrete fill was required.
At Walls L & M footings were

raised to clear ledge, which was about 5' higher than indicated by borings.

— Abutment #2 Retaining Walls
Footings of walls S and T were raised to clear ledge. Concrete fill was placed under parts or all of walls S, T and U to attain uniform bearing.

— Vine Street Retaining Wall
All of this wall was founded on ledge. Ledge was found to be higher at the easterly end than was indicated on the plans.

— Pipe Rail
The pipe rail intended to be installed in front of the garage building on Vine St. between Sta 399+35 and 399+95 was eliminated at a saving of about $600.00. The building foundation was raised by the owner to an elevation above that of the adjacent sidewalk eliminating the need of a ditch and rail. The drop inlet at the

westerly corner of the building was relocated to Sta. 399+92± at the uphill corner of the building and modified to take care of drainage on that side.

- Gravel Base -

The existing median west of the High Street overpass was found to contain gravel base under a thin layer of loam. This effected a saving of several hundred cubic yards of both gravel base and excavation from that originally estimated.

All field changes are shown on the plans, together with final ledge profiles and other construction details.

Materials Used in Project

- Concrete
  Cook & Co., Portland
  Transit mix, haul 32 mi
  Course aggregate - crushed rock from Cook & Co. quarry.
  Fine aggregate - from Cumberland Sand and Gravel Co.
  Cement - Dragon.
  Type II A in bags used when added on project. Type II in bulk when added at plant + 3/4 oz. Darex.
  St. Lawrence Cement was tested and failed
- Reinforcing steel
  Bancroft + Martin Rolling Mills Co.
- Structural steel (including piles)
  Supplied and fabricated by Bancroft Martin Rolling Mills Co.
- Bituminous Concrete
  Supplied and laid by Northeast Paving Corp., Westbrook.
- Aluminum Rail
  Supplied by Bancroft & Martin
  Manufactured by Capitol Products Co.

- Pipe Rail
  Supplied & fabricated by Bancroft & Martin Rolling Mills Co.
- Corrugated Metal Pipe
  Bancroft & Martin Rolling Mills Co.
- Concrete Pipe & drainage structure block
  Maine Cement Products Co.
- Granite curb and edging
  Hocking Granite Co., Clark Island.
- Gravel base
  Harriman Pit, Woolwich.

APPENDIX F

Calculations

LOAD INFORMATION FROM FIGG

FIGG	**Project** Approach Viaduct to Sagadahoc Bridge	**Date** 06/02/14
	Project Number 2215.02	**Designed** ENH
	Description Bottom of Footing Loads for Geotechnical Evaluation (Multi-T, Maine Modified Live Load)	**Checked**

PARAMETERS				
w_{DC} (k/ft)	7.54	N_{brg}	3	
w_{DW} (k/ft)	2.12	1 + IM	1.33	
$P_{sw,cap}$ (k)	77.8	m_1	1.20	
$P_{sw,ftg1}$ (k)	69.3	m_2	1.00	
$P_{sw,ftg2}$ (k)	134.4	d_1 (ft)	11.67	(distance from CL of pier to outside bearing)
V_{CT} (k)	150	d_2 (ft)	5.67	(distance from top of pier column to top of bearing)
μ	0.10			

										UNFACTORED											
														STRENGTH I LIMIT STATE				ALL OTHER LIMIT STATES			
														LC 1*		LC 2**		LC 1*		LC 2**	
Pier No.	L_{trib} (ft)	Group	Trans. CT Load?	Col. Height (ft)	Ftg. Depth (ft)	Fill Depth (ft)	V_y***	$P_{sw,col}$ (k)	$P_{SW,total}$ (k)	P_{DC} (k)	P_{DW} (k)	M_{CT} (k*ft)	M_l*** (k*ft)	P_{LL+IM} (k)	M_t (k*ft)	P_{LL+IM} (k)	M_t (k*ft)	P_{LL+IM} (k)	M_t (k*ft)	P_{LL+IM} (k)	M_t (k*ft)
1	60.0	1	N	9.58	3.00	3.17	S	19.3	40	187	640	127	1675	1057	172	1891	287	1433	147	1616	245	1224
2	60.5	1	N	12.58	3.00	3.17	S	19.5	53	200	656	128	1675	1243	152	1670	253	1265	129	1424	216	1079
3	61.0	1	N	14.75	3.00	2.83	S	19.6	62	209	669	129	1625	1377	155	1700	258	1288	132	1454	220	1101
4	60.5	1	N	16.92	3.00	2.92	S	19.5	71	218	674	128	1638	1497	153	1681	255	1273	131	1436	218	1088
5	60.5	1	N	18.33	4.00	2.92	S	19.5	77	289	745	128	1788	1638	152	1675	254	1269	130	1431	217	1084
6	62.5	2	Y	19.50	4.00	2.92	S	20.1	82	294	765	133	1788	1759	158	1740	264	1318	135	1487	225	1127
7	63.5	2	Y	19.92	4.00	2.67	E	20.5	84	296	775	135	1750	1819	161	1766	268	1338	137	1510	229	1144
8	63.5	2	N	20.58	3.00	2.58	E	18.8	86	234	712	135	1588	1650	160	1762	267	1335	137	1506	228	1141
9	64.0	2	Y	21.08	3.00	2.83	E	18.2	89	236	718	136	1625	1624	162	1781	270	1349	138	1523	231	1154
10	63.5	2	Y	21.17	4.00	2.67	E	2.7	89	301	780	135	1750	250	160	1763	267	1336	137	1507	228	1142
11	63.5	2	N	21.25	4.00	2.50	E	12.7	89	301	780	135	1725	1178	160	1762	267	1335	137	1505	228	1140
12	64.4	2	N	21.00	4.00	2.58	E	20.2	88	300	786	137	1738	1858	164	1802	273	1365	140	1541	233	1167
13	64.8	2	N	20.67	3.00	2.67	E	22.0	87	234	722	137	1600	1936	164	1809	274	1370	140	1543	234	1169
14	70.4	3	Y	20.75	3.00	2.75	S	22.7	87	234	765	149	1613	2003	186	2045	310	1549	158	1743	264	1321
15	75.5	3	Y	20.33	4.00	2.67	S	24.3	85	298	867	160	1750	2187	201	2206	334	1671	171	1883	285	1426
16	72.0	3	Y	19.17	4.00	1.83	S	23.2	81	293	836	153	1625	2007	190	2085	316	1580	162	1777	269	1346
17	69.0	3	Y	16.50	4.00	1.58	S	22.2	69	282	802	146	1588	1743	180	1981	300	1501	154	1691	256	1281
18	64.5	2	Y	13.67	4.00	3.17	S	20.8	57	270	756	137	1825	1456	165	1818	276	1378	141	1551	235	1175
19	59.5	1	N	10.25	4.00	3.33	S	19.2	43	255	704	126	1850	1147	170	1873	284	1419	145	1599	242	1211

NOTES:

* Load Combination 1: One lane of live load (lane A loaded). Strength I limit state uses Maine Modified Live Load. All other limit states use AASHTO HL-93.

** Load Combination 2: Two lanes of live load (lanes A & B loaded). Strength I limit state uses Maine Modified Live Load. All other limit states use AASHTO HL-93.

*** Longitudinal load at each bearing due to CR + SH + TU or FR depending on bearing type (E = elastomeric or S = sliding).

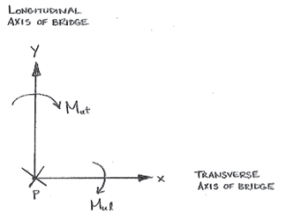
Fill depth per 1957 plans.

P = Axial Load

M_t = Transverse Moment (about longitudinal axis)

M_l = Longitudinal Moment (about transverse axis)

V = Shear



FIGG	**Project** Approach Viaduct to Sagadahoc Bridge	**Date** 06/02/14
	Project Number 2215	**Designed** ENH
	Description Bottom of Footing Loads for Geotechnical Evaluation (Multi-T, Maine Modified Live Load)	**Checked**

Strength I		Service I	
$\gamma_{DC,MIN}$	0.90	γ_{DC}	1.00
$\gamma_{DC,MAX}$	1.25		
$\gamma_{DW,MIN}$	0.65	γ_{DW}	1.00
$\gamma_{DW,MAX}$	1.50		
γ_{LL+IM}	1.75	γ_{LL+IM}	1.00
γ_{FR}	1.00	γ_{FR}	1.00
$\gamma_{CR+SH+TU}$	1.00	$\gamma_{CR+SH+TU}$	1.00

	STRENGTH I LIMIT STATE (MIN)								STRENGTH I LIMIT STATE (MAX)							
	LC 1				LC 2				LC 1				LC 2			
Pier No.	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)
1	959	3310	1057	58	1160	2507	1057	58	1291	3310	1057	58	1492	2507	1057	58
2	940	2922	1243	59	1117	2214	1243	59	1278	2922	1243	59	1455	2214	1243	59
3	957	2974	1377	59	1137	2253	1377	59	1301	2974	1377	59	1481	2253	1377	59
4	958	2942	1497	59	1136	2229	1497	59	1303	2942	1497	59	1481	2229	1497	59
5	1021	2931	1638	59	1198	2220	1638	59	1391	2931	1638	59	1568	2220	1638	59
6	1052	3045	1759	60	1236	2306	1759	60	1432	3045	1759	60	1617	2306	1759	60
7	1066	3091	1819	62	1253	2342	1819	62	1451	3091	1819	62	1639	2342	1819	62
8	1009	3084	1650	56	1196	2336	1650	56	1373	3084	1650	56	1560	2336	1650	56
9	1018	3117	1624	55	1207	2361	1624	55	1385	3117	1624	55	1574	2361	1624	55
10	1070	3086	250	8	1257	2338	250	8	1457	3086	250	8	1644	2338	250	8
11	1070	3083	1178	38	1257	2335	1178	38	1457	3083	1178	38	1644	2335	1178	38
12	1083	3154	1858	61	1274	2389	1858	61	1474	3154	1858	61	1665	2389	1858	61
13	1027	3166	1936	66	1219	2398	1936	66	1397	3166	1936	66	1589	2398	1936	66
14	1111	3578	2003	68	1328	2711	2003	68	1506	3578	2003	68	1722	2711	2003	68
15	1235	3860	2187	73	1469	2925	2187	73	1675	3860	2187	73	1909	2925	2187	73
16	1183	3649	2007	70	1404	2765	2007	70	1605	3649	2007	70	1826	2765	2007	70
17	1132	3467	1743	67	1342	2627	1743	67	1537	3467	1743	67	1747	2627	1743	67
18	1059	3182	1456	62	1251	2411	1456	62	1439	3182	1456	62	1632	2411	1456	62
19	1013	3278	1147	58	1212	2483	1147	58	1367	3278	1147	58	1566	2483	1147	58

	SERVICE I LIMIT STATE							
	LC 1				LC 2			
Pier No.	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)
1	914	1616	1057	58	1012	1224	1057	58
2	914	1424	1243	59	1000	1079	1243	59
3	930	1454	1377	59	1019	1101	1377	59
4	933	1436	1497	59	1020	1088	1497	59
5	1004	1431	1638	59	1090	1084	1638	59
6	1033	1487	1759	60	1123	1127	1759	60
7	1047	1510	1819	62	1138	1144	1819	62
8	984	1506	1650	56	1075	1141	1650	56
9	992	1523	1624	55	1085	1154	1624	55
10	1052	1507	250	8	1143	1142	250	8
11	1052	1505	1178	38	1143	1140	1178	38
12	1063	1541	1858	61	1156	1167	1858	61
13	1000	1543	1936	66	1094	1169	1936	66
14	1073	1743	2003	68	1178	1321	2003	68
15	1198	1883	2187	73	1312	1426	2187	73
16	1150	1777	2007	70	1257	1346	2007	70
17	1102	1691	1743	67	1204	1281	1743	67
18	1034	1551	1456	62	1128	1175	1456	62
19	975	1599	1147	58	1072	1211	1147	58

FIGG	Project	Date
	Approach Viaduct to Sagadahoc Bridge	06/02/14
	Project Number	**Designed**
	2215.02	ENH
	Description	**Checked**
	Bottom of Footing Loads for Geotechnical Evaluation (Multi-T, Maine Modified Live Load)	

Extreme Event II		
	Long.*	Trans.**
$\gamma_{DC,MIN}$	0.90	0.90
$\gamma_{DC,MAX}$	1.25	1.25
$\gamma_{DW,MIN}$	0.65	0.65
$\gamma_{DW,MAX}$	1.50	1.50
γ_{LL+IM}	0.50	0.50
γ_{FR}	0.00	1.00
$\gamma_{CR+SH+TU}$	0.00	1.00
γ_{CT}	1.00	1.00

NOTES:
* Vehiclular collision force (CT) applied in longitudinal direction of bridge for all piers.
** Vehicular collision force (CT) applied in transverse direction of bridge for piers in proximity to a cross street (Middle St, Franklin St Washington St, Ramp, and Water St).
Vehicular collision force (CT) applied 0° from the edge of the pavement.

	EXTREME EVENT II LIMIT STATE (MIN)*								EXTREME EVENT II LIMIT STATE (MAX)*							
	LC 1				LC 2				LC 1				LC 2			
Pier No.	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)
1	732	808	1675	150	781	612	1675	150	1064	808	1675	150	1113	612	1675	150
2	739	712	1675	150	782	540	1675	150	1077	712	1675	150	1120	540	1675	150
3	752	727	1625	150	796	551	1625	150	1096	727	1625	150	1140	551	1625	150
4	756	718	1638	150	799	544	1638	150	1101	718	1638	150	1144	544	1638	150
5	819	715	1788	150	863	542	1788	150	1189	715	1788	150	1233	542	1788	150
6	843	744	1788	150	888	563	1788	150	1223	744	1788	150	1268	563	1788	150
7	853	755	1750	150	899	572	1750	150	1239	755	1750	150	1285	572	1750	150
8	797	753	1588	150	843	570	1588	150	1161	753	1588	150	1206	570	1588	150
9	804	761	1625	150	850	577	1625	150	1170	761	1625	150	1217	577	1625	150
10	858	754	1750	150	904	571	1750	150	1245	754	1750	150	1291	571	1750	150
11	858	753	1725	150	904	570	1725	150	1246	753	1725	150	1291	570	1725	150
12	866	770	1738	150	913	584	1738	150	1257	770	1738	150	1304	584	1738	150
13	810	771	1600	150	856	584	1600	150	1179	771	1600	150	1226	584	1600	150
14	865	872	1613	150	918	660	1613	150	1259	872	1613	150	1312	660	1613	150
15	970	941	1750	150	1027	713	1750	150	1409	941	1750	150	1466	713	1750	150
16	932	888	1625	150	986	673	1625	150	1354	888	1625	150	1408	673	1625	150
17	894	846	1588	150	945	641	1588	150	1298	846	1588	150	1350	641	1588	150
18	840	775	1825	150	887	587	1825	150	1221	775	1825	150	1268	587	1825	150
19	788	799	1850	150	837	606	1850	150	1142	799	1850	150	1190	606	1850	150

	EXTREME EVENT II LIMIT STATE (MIN)**										EXTREME EVENT II LIMIT STATE (MAX)**									
	LC 1					LC 2					LC 1					LC 2				
Pier No.	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{ux} (k)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{ux} (k)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{ux} (k)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{ux} (k)	V_{uy} (k)
1	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
3	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
4	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
5	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
6	843	2531	1759	150	60	888	2351	1759	150	60	1223	2531	1759	150	60	1268	2351	1759	150	60
7	853	2505	1819	150	62	899	2322	1819	150	62	1239	2505	1819	150	62	1285	2322	1819	150	62
8	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
9	804	2386	1624	150	55	850	2202	1624	150	55	1170	2386	1624	150	55	1217	2202	1624	150	55
10	858	2504	250	150	8	904	2321	250	150	8	1245	2504	250	150	8	1291	2321	250	150	8
11	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
12	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
13	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
14	865	2484	2003	150	68	918	2273	2003	150	68	1259	2484	2003	150	68	1312	2273	2003	150	68
15	970	2691	2187	150	73	1027	2463	2187	150	73	1409	2691	2187	150	73	1466	2463	2187	150	73
16	932	2513	2007	150	70	986	2298	2007	150	70	1354	2513	2007	150	70	1408	2298	2007	150	70
17	894	2433	1743	150	67	945	2228	1743	150	67	1298	2433	1743	150	67	1350	2228	1743	150	67
18	840	2600	1456	150	62	887	2412	1456	150	62	1221	2600	1456	150	62	1268	2412	1456	150	62
19	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

FIGG	**Project** Approach Viaduct to Sagadahoc Bridge	**Date** 06/02/14
	Project Number 2215.02	**Designed** ENH
	Description Bottom of Footing Loads for Geotechnical Evaluation (Multi-T)	**Checked**

PARAMETERS				
w_{DC} (k/ft)	7.54	N_{brg}	3	
w_{DW} (k/ft)	2.12	1 + IM	1.33	
$P_{sw,cap}$ (k)	77.8	m_1	1.20	
$P_{sw,ftg1}$ (k)	69.3	m_2	1.00	
$P_{sw,ftg2}$ (k)	134.4	d_1 (ft)	11.67	(distance from CL of pier to outside bearing)
V_{CT} (k)	150	d_2 (ft)	5.67	(distance from top of pier column to top of bearing)
μ	0.10			

											UNFACTORED							
															LC 1*		LC 2**	
Pier No.	L_{trib} (ft)	Group	Trans. CT Load?	Col. Height (ft)	Ftg. Depth (ft)	Fill Depth (ft)		V_y*** (k)	$P_{sw,col}$ (k)	$P_{SW,total}$ (k)	P_{DC} (k)	P_{DW} (k)	M_{CT} (k*ft)	M_l*** (k*ft)	P_{LL+IM} (k)	M_t (k*ft)	P_{LL+IM} (k)	M_t (k*ft)
1	60.0	1	N	9.58	3.00	3.17	S	19.3	40	187	640	127	1675	1057	147	1616	245	1224
2	60.5	1	N	12.58	3.00	3.17	S	19.5	53	200	656	128	1675	1243	129	1424	216	1079
3	61.0	1	N	14.75	3.00	2.83	S	19.6	62	209	669	129	1625	1377	132	1454	220	1101
4	60.5	1	N	16.92	3.00	2.92	S	19.5	71	218	674	128	1638	1497	131	1436	218	1088
5	60.5	1	N	18.33	4.00	2.92	S	19.5	77	289	745	128	1788	1638	130	1431	217	1084
6	62.5	2	Y	19.50	4.00	2.92	S	20.1	82	294	765	133	1788	1759	135	1487	225	1127
7	63.5	2	Y	19.92	4.00	2.67	E	20.5	84	296	775	135	1750	1819	137	1510	229	1144
8	63.5	2	N	20.58	3.00	2.58	E	18.8	86	234	712	135	1588	1650	137	1506	228	1141
9	64.0	2	Y	21.08	3.00	2.83	E	18.2	89	236	718	136	1625	1624	138	1523	231	1154
10	63.5	2	Y	21.17	4.00	2.67	E	2.7	89	301	780	135	1750	250	137	1507	228	1142
11	63.5	2	N	21.25	4.00	2.50	E	12.7	89	301	780	135	1725	1178	137	1505	228	1140
12	64.4	2	N	21.00	4.00	2.58	E	20.2	88	300	786	137	1738	1858	140	1541	233	1167
13	64.8	2	N	20.67	3.00	2.67	E	22.0	87	234	722	137	1600	1936	140	1543	234	1169
14	70.4	3	Y	20.75	3.00	2.75	S	22.7	87	234	765	149	1613	2003	158	1743	264	1321
15	75.5	3	Y	20.33	4.00	2.67	S	24.3	85	298	867	160	1750	2187	171	1883	285	1426
16	72.0	3	Y	19.17	4.00	1.83	S	23.2	81	293	836	153	1625	2007	162	1777	269	1346
17	69.0	3	Y	16.50	4.00	1.58	S	22.2	69	282	802	146	1588	1743	154	1691	256	1281
18	64.5	2	Y	13.67	4.00	3.17	S	20.8	57	270	756	137	1825	1456	141	1551	235	1175
19	59.5	1	N	10.25	4.00	3.33	S	19.2	43	255	704	126	1850	1147	145	1599	242	1211

NOTES:

* Load Combination 1: One lane of live load (lane A loaded).

** Load Combination 2: Two lanes of live load (lanes A & B loaded).

*** Longitudinal load at each bearing due to CR + SH + TU or FR depending on bearing type (E = elastomeric or S = sliding).

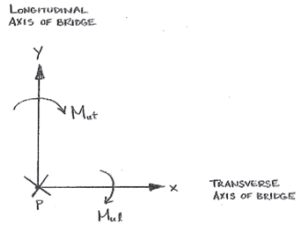
Fill depth per 1957 plans.

P = Axial Load

M_t = Transverse Moment (about longitudinal axis)

M_l = Longitudinal Moment (about transverse axis)

V = Shear



FIGG	Project	Date
	Approach Viaduct to Sagadahoc Bridge	06/02/14
	Project Number	**Designed**
	2215	ENH
	Description	**Checked**
	Bottom of Footing Loads for Geotechnical Evaluation (Multi-T)	

Strength I		Service I	
$\gamma_{DC,MIN}$	0.90	γ_{DC}	1.00
$\gamma_{DC,MAX}$	1.25		
$\gamma_{DW,MIN}$	0.65	γ_{DW}	1.00
$\gamma_{DW,MAX}$	1.50		
γ_{LL+IM}	1.75	γ_{LL+IM}	1.00
γ_{FR}	1.00	γ_{FR}	1.00
$\gamma_{CR+SH+TU}$	1.00	$\gamma_{CR+SH+TU}$	1.00

	STRENGTH I LIMIT STATE (MIN)								STRENGTH I LIMIT STATE (MAX)							
	LC 1				LC 2				LC 1				LC 2			
Pier No.	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)
1	916	2828	1057	58	1087	2143	1057	58	1248	2828	1057	58	1419	2143	1057	58
2	900	2493	1243	59	1052	1889	1243	59	1239	2493	1243	59	1390	1889	1243	59
3	917	2544	1377	59	1072	1927	1377	59	1261	2544	1377	59	1416	1927	1377	59
4	919	2514	1497	59	1071	1904	1497	59	1264	2514	1497	59	1416	1904	1497	59
5	982	2504	1638	59	1134	1897	1638	59	1352	2504	1638	59	1504	1897	1638	59
6	1012	2603	1759	60	1169	1972	1759	60	1392	2603	1759	60	1550	1972	1759	60
7	1025	2642	1819	62	1185	2002	1819	62	1410	2642	1819	62	1571	2002	1819	62
8	968	2635	1650	56	1128	1996	1650	56	1332	2635	1650	56	1492	1996	1650	56
9	977	2665	1624	55	1138	2019	1624	55	1344	2665	1624	55	1505	2019	1624	55
10	1029	2637	250	8	1189	1998	250	8	1417	2637	250	8	1576	1998	250	8
11	1029	2634	1178	38	1189	1996	1178	38	1417	2634	1178	38	1576	1996	1178	38
12	1041	2696	1858	61	1205	2042	1858	61	1432	2696	1858	61	1596	2042	1858	61
13	985	2700	1936	66	1149	2046	1936	66	1355	2700	1936	66	1518	2046	1936	66
14	1063	3051	2003	68	1248	2311	2003	68	1458	3051	2003	68	1642	2311	2003	68
15	1184	3295	2187	73	1383	2496	2187	73	1623	3295	2187	73	1823	2496	2187	73
16	1134	3109	2007	70	1322	2355	2007	70	1556	3109	2007	70	1744	2355	2007	70
17	1086	2959	1743	67	1265	2242	1743	67	1491	2959	1743	67	1670	2242	1743	67
18	1016	2714	1456	62	1180	2056	1456	62	1397	2714	1456	62	1561	2056	1456	62
19	970	2798	1147	58	1139	2120	1147	58	1323	2798	1147	58	1493	2120	1147	58

	SERVICE I LIMIT STATE							
	LC 1				LC 2			
Pier No.	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)
1	914	1616	1057	58	1012	1224	1057	58
2	914	1424	1243	59	1000	1079	1243	59
3	930	1454	1377	59	1019	1101	1377	59
4	933	1436	1497	59	1020	1088	1497	59
5	1004	1431	1638	59	1090	1084	1638	59
6	1033	1487	1759	60	1123	1127	1759	60
7	1047	1510	1819	62	1138	1144	1819	62
8	984	1506	1650	56	1075	1141	1650	56
9	992	1523	1624	55	1085	1154	1624	55
10	1052	1507	250	8	1143	1142	250	8
11	1052	1505	1178	38	1143	1140	1178	38
12	1063	1541	1858	61	1156	1167	1858	61
13	1000	1543	1936	66	1094	1169	1936	66
14	1073	1743	2003	68	1178	1321	2003	68
15	1198	1883	2187	73	1312	1426	2187	73
16	1150	1777	2007	70	1257	1346	2007	70
17	1102	1691	1743	67	1204	1281	1743	67
18	1034	1551	1456	62	1128	1175	1456	62
19	975	1599	1147	58	1072	1211	1147	58

FIGG	**Project** Approach Viaduct to Sagadahoc Bridge	**Date** 06/02/14
	Project Number 2215.02	**Designed** ENH
	Description Bottom of Footing Loads for Geotechnical Evaluation (Multi-T)	**Checked**

Extreme Event II		
	Long.*	Trans.**
$\gamma_{DC,MIN}$	0.90	0.90
$\gamma_{DC,MAX}$	1.25	1.25
$\gamma_{DW,MIN}$	0.65	0.65
$\gamma_{DW,MAX}$	1.50	1.50
γ_{LL+IM}	0.50	0.50
γ_{FR}	0.00	1.00
$\gamma_{CR+SH+TU}$	0.00	1.00
γ_{CT}	1.00	1.00

NOTES:
* Vehiclular collision force (CT) applied in longitudinal direction of bridge for all piers.
** Vehicular collision force (CT) applied in transverse direction of bridge for piers in proximity to a cross street (Middle St, Franklin St, Washington St, Ramp, and Water St).
Vehicular collision force (CT) applied 0° from the edge of the pavement.

	EXTREME EVENT II LIMIT STATE (MIN)*								EXTREME EVENT II LIMIT STATE (MAX)*							
	LC 1				LC 2				LC 1				LC 2			
Pier No.	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)
1	732	808	1675	150	781	612	1675	150	1064	808	1675	150	1113	612	1675	150
2	739	712	1675	150	782	540	1675	150	1077	712	1675	150	1120	540	1675	150
3	752	727	1625	150	796	551	1625	150	1096	727	1625	150	1140	551	1625	150
4	756	718	1638	150	799	544	1638	150	1101	718	1638	150	1144	544	1638	150
5	819	715	1788	150	863	542	1788	150	1189	715	1788	150	1233	542	1788	150
6	843	744	1788	150	888	563	1788	150	1223	744	1788	150	1268	563	1788	150
7	853	755	1750	150	899	572	1750	150	1239	755	1750	150	1285	572	1750	150
8	797	753	1588	150	843	570	1588	150	1161	753	1588	150	1206	570	1588	150
9	804	761	1625	150	850	577	1625	150	1170	761	1625	150	1217	577	1625	150
10	858	754	1750	150	904	571	1750	150	1245	754	1750	150	1291	571	1750	150
11	858	753	1725	150	904	570	1725	150	1246	753	1725	150	1291	570	1725	150
12	866	770	1738	150	913	584	1738	150	1257	770	1738	150	1304	584	1738	150
13	810	771	1600	150	856	584	1600	150	1179	771	1600	150	1226	584	1600	150
14	865	872	1613	150	918	660	1613	150	1259	872	1613	150	1312	660	1613	150
15	970	941	1750	150	1027	713	1750	150	1409	941	1750	150	1466	713	1750	150
16	932	888	1625	150	986	673	1625	150	1354	888	1625	150	1408	673	1625	150
17	894	846	1588	150	945	641	1588	150	1298	846	1588	150	1350	641	1588	150
18	840	775	1825	150	887	587	1825	150	1221	775	1825	150	1268	587	1825	150
19	788	799	1850	150	837	606	1850	150	1142	799	1850	150	1190	606	1850	150

	EXTREME EVENT II LIMIT STATE (MIN)**										EXTREME EVENT II LIMIT STATE (MAX)**									
	LC 1					LC 2					LC 1					LC 2				
Pier No.	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{ux} (k)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{ux} (k)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{ux} (k)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{ux} (k)	V_{uy} (k)
1	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
3	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
4	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
5	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
6	843	2531	1759	150	60	888	2351	1759	150	60	1223	2531	1759	150	60	1268	2351	1759	150	60
7	853	2505	1819	150	62	899	2322	1819	150	62	1239	2505	1819	150	62	1285	2322	1819	150	62
8	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
9	804	2386	1624	150	55	850	2202	1624	150	55	1170	2386	1624	150	55	1217	2202	1624	150	55
10	858	2504	250	150	8	904	2321	250	150	8	1245	2504	250	150	8	1291	2321	250	150	8
11	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
12	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
13	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
14	865	2484	2003	150	68	918	2273	2003	150	68	1259	2484	2003	150	68	1312	2273	2003	150	68
15	970	2691	2187	150	73	1027	2463	2187	150	73	1409	2691	2187	150	73	1466	2463	2187	150	73
16	932	2513	2007	150	70	986	2298	2007	150	70	1354	2513	2007	150	70	1408	2298	2007	150	70
17	894	2433	1743	150	67	945	2228	1743	150	67	1298	2433	1743	150	67	1350	2228	1743	150	67
18	840	2600	1456	150	62	887	2412	1456	150	62	1221	2600	1456	150	62	1268	2412	1456	150	62
19	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

FIGG	**Project** Approach Viaduct to Sagadahoc Bridge	**Date** 06/05/14
	Project Number 2215.02	**Designed** ENH
	Description Bottom of Footing Loads for Geotechnical Evaluation (NEXT, Maine Modified Live Load)	**Checked**

PARAMETERS				
w_{DC} (k/ft)	8.70	1 + IM	1.33	
w_{DW} (k/ft)	2.12	m_1	1.20	
$P_{sw,cap}$ (k)	77.8	m_2	1.00	
$P_{sw,ftg1}$ (k)	69.3	d_1 (ft)	14.00	(distance from CL of pier to brg 1)
$P_{sw,ftg2}$ (k)	134.4	d_2 (ft)	9.00	(distance from CL of pier to brg 2)
V_{CT} (k)	150	d_3 (ft)	2.50	(distance from CL of pier to brg 3)
μ	0.10	d_4 (ft)	5.67	(distance from top of pier column to top of bearing)
N_{brg}	6			

											UNFACTORED											
															STRENGTH I LIMIT STATE				ALL OTHER LIMIT STATES			
															LC 1*		LC 2**		LC 1*		LC 2**	
Pier No.	L_{trib} (ft)	Group	Trans. CT Load?	Col. Height (ft)	Ftg. Depth (ft)	Fill Depth (ft)	V_y***	(k)	$P_{sw,col}$ (k)	$P_{SW,total}$ (k)	P_{DC} (k)	P_{DW} (k)	M_{CT} (k*ft)	M_l*** (k*ft)	P_{LL+IM} (k)	M_t (k*ft)	P_{LL+IM} (k)	M_t (k*ft)	P_{LL+IM} (k)	M_t (k*ft)	P_{LL+IM} (k)	M_t (k*ft)
1	60.0	1	N	9.58	3.00	3.17	S	10.8	40	187	709	127	1675	1183	172	1891	287	1433	147	1616	245	1224
2	60.5	1	N	12.58	3.00	3.17	S	10.9	53	200	726	128	1675	1390	152	1670	253	1265	129	1424	216	1079
3	61.0	1	N	14.75	3.00	2.83	S	11.0	62	209	740	129	1625	1546	155	1700	258	1288	132	1454	220	1101
4	60.5	1	N	16.92	3.00	2.92	S	10.9	71	218	745	128	1638	1673	153	1681	255	1273	131	1436	218	1088
5	60.5	1	N	18.33	4.00	2.92	S	10.9	77	289	816	128	1788	1831	152	1675	254	1269	130	1431	217	1084
6	62.5	2	Y	19.50	4.00	2.92	S	11.3	82	294	838	133	1788	1978	158	1740	264	1318	135	1487	225	1127
7	63.5	2	Y	19.92	4.00	2.67	E	10.6	84	296	848	135	1750	1882	161	1766	268	1338	137	1510	229	1144
8	63.5	2	N	20.58	3.00	2.58	E	9.7	86	234	786	135	1588	1702	160	1762	267	1335	137	1506	228	1141
9	64.0	2	Y	21.08	3.00	2.83	E	9.5	89	236	792	136	1625	1696	162	1781	270	1349	138	1523	231	1154
10	63.5	2	Y	21.17	4.00	2.67	E	1.4	89	301	854	135	1750	259	160	1763	267	1336	137	1507	228	1142
11	63.5	2	N	21.25	4.00	2.50	E	6.6	89	301	854	135	1725	1224	160	1762	267	1335	137	1505	228	1140
12	64.4	2	N	21.00	4.00	2.58	E	10.5	88	300	861	137	1738	1932	164	1802	273	1365	140	1541	233	1167
13	64.8	2	N	20.67	3.00	2.67	E	11.4	87	234	798	137	1600	2006	164	1809	274	1370	140	1543	234	1169
14	70.4	3	Y	20.75	3.00	2.75	S	12.7	87	234	847	149	1613	2242	186	2045	310	1549	158	1743	264	1321
15	75.5	3	Y	20.33	4.00	2.67	S	13.6	85	298	954	160	1750	2448	201	2206	334	1671	171	1883	285	1426
16	72.0	3	Y	19.17	4.00	1.83	S	13.0	81	293	919	153	1625	2249	190	2085	316	1580	162	1777	269	1346
17	69.0	3	Y	16.50	4.00	1.58	S	12.4	69	282	882	146	1588	1947	180	1981	300	1501	154	1691	256	1281
18	64.5	2	Y	13.67	4.00	3.17	S	11.6	57	270	831	137	1825	1624	165	1818	276	1378	141	1551	235	1175
19	59.5	1	N	10.25	4.00	3.33	S	10.7	43	255	773	126	1850	1279	170	1873	284	1419	145	1599	242	1211

NOTES:

* Load Combination 1: One lane of live load (lane A loaded). Strength I limit state uses Maine Modified Live Load. All other limit states use AASHTO HL-93.

** Load Combination 2: Two lanes of live load (lanes A & B loaded). Strength I limit state uses Maine Modified Live Load. All other limit states use AASHTO HL-93.

*** Longitudinal load at each bearing due to CR + SH + TU or FR depending on bearing type (E = elastomeric or S = sliding).

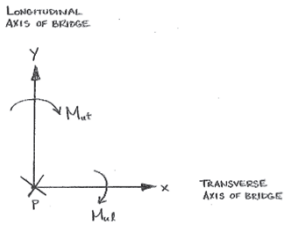
Fill depth per 1957 plans.

P = Axial Load

M_t = Transverse Moment (about longitudinal axis)

M_l = Longitudinal Moment (about transverse axis)

V = Shear



FIGG	**Project** Approach Viaduct to Sagadahoc Bridge	**Date** 06/05/14
	Project Number 2215.02	**Designed** ENH
	Description Bottom of Footing Loads for Geotechnical Evaluation (NEXT, Maine Modified Live Load)	**Checked**

Strength I		Service I	
$\gamma_{DC,MIN}$	0.90	γ_{DC}	1.00
$\gamma_{DC,MAX}$	1.25		
$\gamma_{DW,MIN}$	0.65	γ_{DW}	1.00
$\gamma_{DW,MAX}$	1.50		
γ_{LL+IM}	1.75	γ_{LL+IM}	1.00
γ_{FR}	1.00	γ_{FR}	1.00
$\gamma_{CR+SH+TU}$	1.00	$\gamma_{CR+SH+TU}$	1.00

	STRENGTH I LIMIT STATE (MIN)								STRENGTH I LIMIT STATE (MAX)							
	LC 1				LC 2				LC 1				LC 2			
Pier No.	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)
1	1022	3310	1183	65	1223	2507	1183	65	1378	3310	1183	65	1579	2507	1183	65
2	1003	2922	1390	65	1180	2214	1390	65	1366	2922	1390	65	1543	2214	1390	65
3	1020	2974	1546	66	1200	2253	1546	66	1389	2974	1546	66	1569	2253	1546	66
4	1021	2942	1673	65	1199	2229	1673	65	1390	2942	1673	65	1569	2229	1673	65
5	1084	2931	1831	65	1261	2220	1831	65	1478	2931	1831	65	1656	2220	1831	65
6	1117	3045	1978	68	1301	2306	1978	68	1523	3045	1978	68	1707	2306	1978	68
7	1132	3091	1882	64	1319	2342	1882	64	1543	3091	1882	64	1731	2342	1882	64
8	1075	3084	1702	58	1262	2336	1702	58	1465	3084	1702	58	1652	2336	1702	58
9	1085	3117	1696	57	1274	2361	1696	57	1477	3117	1696	57	1666	2361	1696	57
10	1136	3086	259	8	1323	2338	259	8	1549	3086	259	8	1736	2338	259	8
11	1136	3083	1224	40	1323	2335	1224	40	1550	3083	1224	40	1736	2335	1224	40
12	1150	3154	1932	63	1341	2389	1932	63	1567	3154	1932	63	1758	2389	1932	63
13	1095	3166	2006	68	1287	2398	2006	68	1491	3166	2006	68	1683	2398	2006	68
14	1184	3578	2242	76	1401	2711	2242	76	1608	3578	2242	76	1824	2711	2242	76
15	1314	3860	2448	82	1548	2925	2448	82	1784	3860	2448	82	2018	2925	2448	82
16	1258	3649	2249	78	1479	2765	2249	78	1710	3649	2249	78	1931	2765	2249	78
17	1204	3467	1947	74	1414	2627	1947	74	1637	3467	1947	74	1847	2627	1947	74
18	1126	3182	1624	70	1319	2411	1624	70	1533	3182	1624	70	1726	2411	1624	70
19	1076	3278	1279	64	1274	2483	1279	64	1453	3278	1279	64	1652	2483	1279	64

	SERVICE I LIMIT STATE							
	LC 1				LC 2			
Pier No.	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)
1	983	1616	1183	65	1081	1224	1183	65
2	984	1424	1390	65	1070	1079	1390	65
3	1001	1454	1546	66	1089	1101	1546	66
4	1003	1436	1673	65	1090	1088	1673	65
5	1074	1431	1831	65	1161	1084	1831	65
6	1106	1487	1978	68	1196	1127	1978	68
7	1120	1510	1882	64	1212	1144	1882	64
8	1058	1506	1702	58	1149	1141	1702	58
9	1067	1523	1696	57	1159	1154	1696	57
10	1125	1507	259	8	1217	1142	259	8
11	1125	1505	1224	40	1217	1140	1224	40
12	1137	1541	1932	63	1231	1167	1932	63
13	1075	1543	2006	68	1169	1169	2006	68
14	1154	1743	2242	76	1260	1321	2242	76
15	1286	1883	2448	82	1400	1426	2448	82
16	1233	1777	2249	78	1341	1346	2249	78
17	1182	1691	1947	74	1284	1281	1947	74
18	1108	1551	1624	70	1202	1175	1624	70
19	1044	1599	1279	64	1141	1211	1279	64

FIGG	**Project** Approach Viaduct to Sagadahoc Bridge	**Date** 06/05/14
	Project Number 2215.02	**Designed** ENH
	Description Bottom of Footing Loads for Geotechnical Evaluation (NEXT, Maine Modified Live Load)	**Checked**

Extreme Event II		
	Long.*	Trans.**
$\gamma_{DC,MIN}$	0.90	0.90
$\gamma_{DC,MAX}$	1.25	1.25
$\gamma_{DW,MIN}$	0.65	0.65
$\gamma_{DW,MAX}$	1.50	1.50
γ_{LL+IM}	0.50	0.50
γ_{FR}	0.00	1.00
$\gamma_{CR+SH+TU}$	0.00	1.00
γ_{CT}	1.00	1.00

NOTES:
* Vehicular collision force (CT) applied in longitudinal direction of bridge for all piers.
** Vehicular collision force (CT) applied in transverse direction of bridge for piers in proximity to a cross street (Middle St, Franklin St, Washington St, Ramp, and Water St).
Vehicular collision force (CT) applied 0" from the edge of the pavement.

	EXTREME EVENT II LIMIT STATE (MIN)*								EXTREME EVENT II LIMIT STATE (MAX)*							
	LC 1				LC 2				LC 1				LC 2			
Pier No.	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)
1	795	808	1675	150	844	612	1675	150	1151	808	1675	150	1200	612	1675	150
2	802	712	1675	150	845	540	1675	150	1165	712	1675	150	1208	540	1675	150
3	816	727	1625	150	860	551	1625	150	1185	727	1625	150	1229	551	1625	150
4	819	718	1638	150	862	544	1638	150	1188	718	1638	150	1232	544	1638	150
5	882	715	1788	150	926	542	1788	150	1277	715	1788	150	1320	542	1788	150
6	908	744	1788	150	953	563	1788	150	1314	744	1788	150	1359	563	1788	150
7	920	755	1750	150	965	572	1750	150	1331	755	1750	150	1377	572	1750	150
8	863	753	1588	150	909	570	1588	150	1253	753	1588	150	1299	570	1588	150
9	871	761	1625	150	917	577	1625	150	1263	761	1625	150	1309	577	1625	150
10	924	754	1750	150	970	571	1750	150	1337	754	1750	150	1383	571	1750	150
11	924	753	1725	150	970	570	1725	150	1338	753	1725	150	1383	570	1725	150
12	933	770	1738	150	980	584	1738	150	1351	770	1738	150	1397	584	1738	150
13	877	771	1600	150	924	584	1600	150	1273	771	1600	150	1320	584	1600	150
14	938	872	1613	150	991	660	1613	150	1362	872	1613	150	1414	660	1613	150
15	1049	941	1750	150	1106	713	1750	150	1519	941	1750	150	1576	713	1750	150
16	1007	888	1625	150	1061	673	1625	150	1459	888	1625	150	1512	673	1625	150
17	966	846	1588	150	1017	641	1588	150	1399	846	1588	150	1450	641	1588	150
18	907	775	1825	150	954	587	1825	150	1314	775	1825	150	1361	587	1825	150
19	850	799	1850	150	899	606	1850	150	1228	799	1850	150	1276	606	1850	150

	EXTREME EVENT II LIMIT STATE (MIN)**										EXTREME EVENT II LIMIT STATE (MAX)**									
	LC 1					LC 2					LC 1					LC 2				
Pier No.	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{ux} (k)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{ux} (k)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{ux} (k)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{ux} (k)	V_{uy} (k)
1	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
3	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
4	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
5	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
6	908	2531	1978	150	68	953	2351	1978	150	68	1314	2531	1978	150	68	1359	2351	1978	150	68
7	920	2505	1882	150	64	965	2322	1882	150	64	1331	2505	1882	150	64	1377	2322	1882	150	64
8	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
9	871	2386	1696	150	57	917	2202	1696	150	57	1263	2386	1696	150	57	1309	2202	1696	150	57
10	924	2504	259	150	8	970	2321	259	150	8	1337	2504	259	150	8	1383	2321	259	150	8
11	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
12	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
13	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
14	938	2484	2242	150	76	991	2273	2242	150	76	1362	2484	2242	150	76	1414	2273	2242	150	76
15	1049	2691	2448	150	82	1106	2463	2448	150	82	1519	2691	2448	150	82	1576	2463	2448	150	82
16	1007	2513	2249	150	78	1061	2298	2249	150	78	1459	2513	2249	150	78	1512	2298	2249	150	78
17	966	2433	1947	150	74	1017	2228	1947	150	74	1399	2433	1947	150	74	1450	2228	1947	150	74
18	907	2600	1624	150	70	954	2412	1624	150	70	1314	2600	1624	150	70	1361	2412	1624	150	70
19	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

FIGG	**Project** Approach Viaduct to Sagadahoc Bridge	**Date** 06/05/14
	Project Number 2215.02	**Designed** ENH
	Description Bottom of Footing Loads for Geotechnical Evaluation (NEXT)	**Checked**

PARAMETERS				
w_{DC} (k/ft)	8.70	1 + IM	1.33	
w_{DW} (k/ft)	2.12	m_1	1.20	
$P_{sw,cap}$ (k)	77.8	m_2	1.00	
$P_{sw,ftg1}$ (k)	69.3	d_1 (ft)	14.00	(distance from CL of pier to brg 1)
$P_{sw,ftg2}$ (k)	134.4	d_2 (ft)	9.00	(distance from CL of pier to brg 2)
V_{CT} (k)	150	d_3 (ft)	2.50	(distance from CL of pier to brg 3)
μ	0.10	d_4 (ft)	5.67	(distance from top of pier column to top of bearing)
N_{brg}	6			

											UNFACTORED							
															LC 1*		LC 2**	
Pier No.	L_{trib} (ft)	Group	Trans. CT Load?	Col. Height (ft)	Ftg. Depth (ft)	Fill Depth (ft)	V_y***	(k)	$P_{sw,col}$ (k)	$P_{SW,total}$ (k)	P_{DC} (k)	P_{DW} (k)	M_{CT} (k*ft)	M_l*** (k*ft)	P_{LL+IM} (k)	M_t (k*ft)	P_{LL+IM} (k)	M_t (k*ft)
1	60.0	1	N	9.58	3.00	3.17	S	10.8	40	187	709	127	1675	1183	147	1616	245	1224
2	60.5	1	N	12.58	3.00	3.17	S	10.9	53	200	726	128	1675	1390	129	1424	216	1079
3	61.0	1	N	14.75	3.00	2.83	S	11.0	62	209	740	129	1625	1546	132	1454	220	1101
4	60.5	1	N	16.92	3.00	2.92	S	10.9	71	218	745	128	1638	1673	131	1436	218	1088
5	60.5	1	N	18.33	4.00	2.92	S	10.9	77	289	816	128	1788	1831	130	1431	217	1084
6	62.5	2	Y	19.50	4.00	2.92	S	11.3	82	294	838	133	1788	1978	135	1487	225	1127
7	63.5	2	Y	19.92	4.00	2.67	E	10.6	84	296	848	135	1750	1882	137	1510	229	1144
8	63.5	2	N	20.58	3.00	2.58	E	9.7	86	234	786	135	1588	1702	137	1506	228	1141
9	64.0	2	Y	21.08	3.00	2.83	E	9.5	89	236	792	136	1625	1696	138	1523	231	1154
10	63.5	2	Y	21.17	4.00	2.67	E	1.4	89	301	854	135	1750	259	137	1507	228	1142
11	63.5	2	N	21.25	4.00	2.50	E	6.6	89	301	854	135	1725	1224	137	1505	228	1140
12	64.4	2	N	21.00	4.00	2.58	E	10.5	88	300	861	137	1738	1932	140	1541	233	1167
13	64.8	2	N	20.67	3.00	2.67	E	11.4	87	234	798	137	1600	2006	140	1543	234	1169
14	70.4	3	Y	20.75	3.00	2.75	S	12.7	87	234	847	149	1613	2242	158	1743	264	1321
15	75.5	3	Y	20.33	4.00	2.67	S	13.6	85	298	954	160	1750	2448	171	1883	285	1426
16	72.0	3	Y	19.17	4.00	1.83	S	13.0	81	293	919	153	1625	2249	162	1777	269	1346
17	69.0	3	Y	16.50	4.00	1.58	S	12.4	69	282	882	146	1588	1947	154	1691	256	1281
18	64.5	2	Y	13.67	4.00	3.17	S	11.6	57	270	831	137	1825	1624	141	1551	235	1175
19	59.5	1	N	10.25	4.00	3.33	S	10.7	43	255	773	126	1850	1279	145	1599	242	1211

NOTES:

* Load Combination 1: One lane of live load (lane A loaded).

** Load Combination 2: Two lanes of live load (lanes A & B loaded).

*** Longitudinal load at each bearing due to CR + SH + TU or FR depending on bearing type (E = elastomeric or S = sliding).

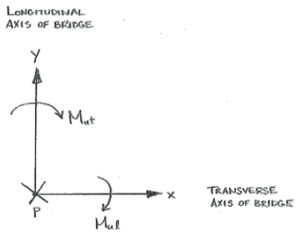
Fill depth per 1957 plans.

P = Axial Load

M_t = Transverse Moment (about longitudinal axis)

M_l = Longitudinal Moment (about transverse axis)

V = Shear



FIGG	**Project** Approach Viaduct to Sagadahoc Bridge	**Date** 06/05/14
	Project Number 2215.02	**Designed** ENH
	Description Bottom of Footing Loads for Geotechnical Evaluation (NEXT)	**Checked**

Strength I		Service I	
$\gamma_{DC,MIN}$	0.90	γ_{DC}	1.00
$\gamma_{DC,MAX}$	1.25		
$\gamma_{DW,MIN}$	0.65	γ_{DW}	1.00
$\gamma_{DW,MAX}$	1.50		
γ_{LL+IM}	1.75	γ_{LL+IM}	1.00
γ_{FR}	1.00	γ_{FR}	1.00
$\gamma_{CR+SH+TU}$	1.00	$\gamma_{CR+SH+TU}$	1.00

	STRENGTH I LIMIT STATE (MIN)								STRENGTH I LIMIT STATE (MAX)							
	LC 1				LC 2				LC 1				LC 2			
Pier No.	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)
1	978	2828	1183	65	1150	2143	1183	65	1335	2828	1183	65	1506	2143	1183	65
2	964	2493	1390	65	1115	1889	1390	65	1327	2493	1390	65	1478	1889	1390	65
3	981	2544	1546	66	1135	1927	1546	66	1350	2544	1546	66	1504	1927	1546	66
4	982	2514	1673	65	1134	1904	1673	65	1352	2514	1673	65	1504	1904	1673	65
5	1045	2504	1831	65	1197	1897	1831	65	1439	2504	1831	65	1591	1897	1831	65
6	1077	2603	1978	68	1235	1972	1978	68	1483	2603	1978	68	1640	1972	1978	68
7	1091	2642	1882	64	1251	2002	1882	64	1503	2642	1882	64	1663	2002	1882	64
8	1034	2635	1702	58	1194	1996	1702	58	1424	2635	1702	58	1584	1996	1702	58
9	1044	2665	1696	57	1205	2019	1696	57	1436	2665	1696	57	1598	2019	1696	57
10	1095	2637	259	8	1255	1998	259	8	1509	2637	259	8	1668	1998	259	8
11	1095	2634	1224	40	1255	1996	1224	40	1509	2634	1224	40	1668	1996	1224	40
12	1108	2696	1932	63	1272	2042	1932	63	1526	2696	1932	63	1689	2042	1932	63
13	1053	2700	2006	68	1216	2046	2006	68	1449	2700	2006	68	1612	2046	2006	68
14	1136	3051	2242	76	1321	2311	2242	76	1560	3051	2242	76	1745	2311	2242	76
15	1263	3295	2448	82	1462	2496	2448	82	1733	3295	2448	82	1932	2496	2448	82
16	1209	3109	2249	78	1397	2355	2249	78	1660	3109	2249	78	1849	2355	2249	78
17	1158	2959	1947	74	1337	2242	1947	74	1591	2959	1947	74	1770	2242	1947	74
18	1083	2714	1624	70	1248	2056	1624	70	1490	2714	1624	70	1655	2056	1624	70
19	1032	2798	1279	64	1202	2120	1279	64	1410	2798	1279	64	1579	2120	1279	64

	SERVICE I LIMIT STATE							
	LC 1				LC 2			
Pier No.	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)
1	983	1616	1183	65	1081	1224	1183	65
2	984	1424	1390	65	1070	1079	1390	65
3	1001	1454	1546	66	1089	1101	1546	66
4	1003	1436	1673	65	1090	1088	1673	65
5	1074	1431	1831	65	1161	1084	1831	65
6	1106	1487	1978	68	1196	1127	1978	68
7	1120	1510	1882	64	1212	1144	1882	64
8	1058	1506	1702	58	1149	1141	1702	58
9	1067	1523	1696	57	1159	1154	1696	57
10	1125	1507	259	8	1217	1142	259	8
11	1125	1505	1224	40	1217	1140	1224	40
12	1137	1541	1932	63	1231	1167	1932	63
13	1075	1543	2006	68	1169	1169	2006	68
14	1154	1743	2242	76	1260	1321	2242	76
15	1286	1883	2448	82	1400	1426	2448	82
16	1233	1777	2249	78	1341	1346	2249	78
17	1182	1691	1947	74	1284	1281	1947	74
18	1108	1551	1624	70	1202	1175	1624	70
19	1044	1599	1279	64	1141	1211	1279	64

FIGG	**Project** Approach Viaduct to Sagadahoc Bridge	**Date** 06/05/14
	Project Number 2215.02	**Designed** ENH
	Description Bottom of Footing Loads for Geotechnical Evaluation (NEXT)	**Checked**

Extreme Event II		
	Long.*	Trans.**
$\gamma_{DC,MIN}$	0.90	0.90
$\gamma_{DC,MAX}$	1.25	1.25
$\gamma_{DW,MIN}$	0.65	0.65
$\gamma_{DW,MAX}$	1.50	1.50
γ_{LL+IM}	0.50	0.50
γ_{FR}	0.00	1.00
$\gamma_{CR+SH+TU}$	0.00	1.00
γ_{CT}	1.00	1.00

NOTES:
* Vehiclular collision force (CT) applied in longitudinal direction of bridge for all piers.
** Vehicular collision force (CT) applied in transverse direction of bridge for piers in proximity to a cross street (Middle St, Franklin St, Washington St, Ramp, and Water St).
Vehicular collision force (CT) applied 0° from the edge of the pavement.

	EXTREME EVENT II LIMIT STATE (MIN)*								EXTREME EVENT II LIMIT STATE (MAX)*							
	LC 1				LC 2				LC 1				LC 2			
Pier No.	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{uy} (k)
1	795	808	1675	150	844	612	1675	150	1151	808	1675	150	1200	612	1675	150
2	802	712	1675	150	845	540	1675	150	1165	712	1675	150	1208	540	1675	150
3	816	727	1625	150	860	551	1625	150	1185	727	1625	150	1229	551	1625	150
4	819	718	1638	150	862	544	1638	150	1188	718	1638	150	1232	544	1638	150
5	882	715	1788	150	926	542	1788	150	1277	715	1788	150	1320	542	1788	150
6	908	744	1788	150	953	563	1788	150	1314	744	1788	150	1359	563	1788	150
7	920	755	1750	150	965	572	1750	150	1331	755	1750	150	1377	572	1750	150
8	863	753	1588	150	909	570	1588	150	1253	753	1588	150	1299	570	1588	150
9	871	761	1625	150	917	577	1625	150	1263	761	1625	150	1309	577	1625	150
10	924	754	1750	150	970	571	1750	150	1337	754	1750	150	1383	571	1750	150
11	924	753	1725	150	970	570	1725	150	1338	753	1725	150	1383	570	1725	150
12	933	770	1738	150	980	584	1738	150	1351	770	1738	150	1397	584	1738	150
13	877	771	1600	150	924	584	1600	150	1273	771	1600	150	1320	584	1600	150
14	938	872	1613	150	991	660	1613	150	1362	872	1613	150	1414	660	1613	150
15	1049	941	1750	150	1106	713	1750	150	1519	941	1750	150	1576	713	1750	150
16	1007	888	1625	150	1061	673	1625	150	1459	888	1625	150	1512	673	1625	150
17	966	846	1588	150	1017	641	1588	150	1399	846	1588	150	1450	641	1588	150
18	907	775	1825	150	954	587	1825	150	1314	775	1825	150	1361	587	1825	150
19	850	799	1850	150	899	606	1850	150	1228	799	1850	150	1276	606	1850	150

	EXTREME EVENT II LIMIT STATE (MIN)**										EXTREME EVENT II LIMIT STATE (MAX)**									
	LC 1					LC 2					LC 1					LC 2				
Pier No.	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{ux} (k)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{ux} (k)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{ux} (k)	V_{uy} (k)	P_u (k)	M_{ut} (k*ft)	M_{ul} (k*ft)	V_{ux} (k)	V_{uy} (k)
1	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
2	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
3	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
4	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
5	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
6	908	2531	1978	150	68	953	2351	1978	150	68	1314	2531	1978	150	68	1359	2351	1978	150	68
7	920	2505	1882	150	64	965	2322	1882	150	64	1331	2505	1882	150	64	1377	2322	1882	150	64
8	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
9	871	2386	1696	150	57	917	2202	1696	150	57	1263	2386	1696	150	57	1309	2202	1696	150	57
10	924	2504	259	150	8	970	2321	259	150	8	1337	2504	259	150	8	1383	2321	259	150	8
11	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
12	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
13	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
14	938	2484	2242	150	76	991	2273	2242	150	76	1362	2484	2242	150	76	1414	2273	2242	150	76
15	1049	2691	2448	150	82	1106	2463	2448	150	82	1519	2691	2448	150	82	1576	2463	2448	150	82
16	1007	2513	2249	150	78	1061	2298	2249	150	78	1459	2513	2249	150	78	1512	2298	2249	150	78
17	966	2433	1947	150	74	1017	2228	1947	150	74	1399	2433	1947	150	74	1450	2228	1947	150	74
18	907	2600	1624	150	70	954	2412	1624	150	70	1314	2600	1624	150	70	1361	2412	1624	150	70
19	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A

HALEY & ALDRICH

CALCULATIONS

File No.	40725-000		
Sheet	1 of 1		
Client	Maine Department of Transportation	**Date**	4/25/2014
Project	Route 1 Viaduct Bridge Rehabilitation	**Computed By**	BCS
Subject	Steel H-Pile Section Loss Due to Corrosion	**Checked By**	JLL

HP Section =	HP10x42
Depth of Section, d (in) =	9.720
Flange Thickness, t_f (in) =	0.418
Flange Width, b_f (in) =	10.078
Web Thickness, t_w (in) =	0.418
Cross Sectional Area, A_s (in^2) =	12.35
Moment of Inertia, $I_{g\text{-}xx}$ (in^4) =	210.8
Moment of Inertia, $I_{g\text{-}yy}$ (in^4) =	71.4

Note:

Dimensions and section properties taken from various historic references and are likely consistent with existing steel H-piles supporting substructures that were installed in the late 1950's.

Assumed Corrosion Allowance:

	1/32 in.	1/16 in.	1/8 in.
t_c (in.) =	0.0313	0.0625	0.125

Corroded Section Properties:

A_{eff} (in^2) =	10.30	8.47	4.84
$I_{eff\text{-}xx}$ (in^4) =	175	144	82
$I_{eff\text{-}yy}$ (in^4) =	60	48	27

Check (perform corroded section calculations a different way):

A_{eff} (in^2) =	10.30	8.47	4.84
$I_{eff\text{-}xx}$ (in^4) =	175	144	82
$I_{eff\text{-}yy}$ (in^4) =	60	48	27

HALEY & ALDRICH

CALCULATIONS

File No.	40725-000		
Sheet	1 of 1		
Client	Maine Department of Transportation	**Date**	4/25/2014
Project	Route 1 Viaduct Bridge Rehabilitation	**Computed By**	BCS
Subject	Structural Resistance of Existing Steel H-Piles	**Checked By**	JLL

OBJECTIVE:

Calculate nominal and factored axial compressive structural resistance of steel H-piles driven to bedrock in accordance with AASHTO LRFD Bridge Design Specifications 6th Edition 2012 with 2013 Interim Revisions.

SUMMARY OF EXISTING BRIDGE STRUCTURE FOUNDATION SUPPORT:

Information provided below is based on our review of historic viaduct bridge plans provided by MaineDOT.

Existing viaduct bridge originally constructed in late 1950's and consists of a 20-span structure supported on 19 piers and two abutments. A summary of the foundation support for each abutment and pier is provided below.

Abutment 1, Piers 1-4, 8-9 and 13-14 are supported on spread footings bearing directly on bedrock or on seal concrete, which bears directly on bedrock.

Piers 5-7, 10-12, 15-19 and Abutment 2 are supported on HP10x42 steel H-piles.

EXISTING FOUNDATION PILE EVALUATION:

Based on our review of historic bridge plans, recent subsurface explorations (test borings and test pits) and the results of non-destructive testing the tips of some piles supporting select piers are located in/on bedrock.

NOMINAL AXIAL COMPRESSIVE STRUCTURAL RESISTANCE:

$P_n = 0.66^{\lambda} \times F_y \times A_s$ (EQN. 6.9.4.1-1)

For piles with unbraced length = 0 ft….. $\lambda = 0$ and $P_n = F_y \times A_s$

Yield Strength of Steel, F_y = 33 ksi (late 1950's structural steel conforming to ASTM A-7 per MaineDOT Std. Spec's)

	0" loss	1/32" loss	1/16" loss	1/8" loss
Cross Sectional Area of HP10x42 steel H-pile, A_s =	12.35 in^2	10.30 in^2	8.47 in^2	4.84 in^2
Nominal Structural Resistance, P_n =	408 kips	340 kips	280 kips	160 kips

FACTORED AXIAL COMPRESSIVE STRUCTURAL RESISTANCE:

$P_f = \phi \times P_n$ (EQN. 6.9.2.1-1)

Limit State	Resistance Factor (ϕ)	LRFD Reference
Service	1.0	Article 10.5.5.1
Strength	0.5	Article 6.5.4.2
Extreme	1.0	Article 10.5.5.3

Corrosion Loss	Factored Structural Resistance (P_f, kips)		
	Service	Strength	Extreme
0"	408	204	408
1/32"	340	170	340
1/16"	280	140	280
1/8"	160	80	160

MULTI-T FB-MULTIPIER RESULTS

HALEY & ALDRICH	CALCULATIONS	File No.	40725-000
		Sheet	1 of 1
Client	MaineDOT	Date	30JUN2014
Project	Route 1 Viaduct Rehabilitation	Computed by	JLL
Subject	FB-MultiPier Results - Combined Internal Pile Stresses	Checked by	

Data File: C:\Bath\STR XML\2014-0701-HAI-Bath-Pier 7-MultiT-UnMod-STR-D2.XML

FB-MultiPier Results - Pier 7 - Pile Stresses for Multi-T UnModified HL-93 STR Load Cases

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	1	65	0.77	23.98
2	1	95	1.00	30.19
3	1	118	1.08	31.78
4	1	139	1.20	34.81
5	1	41	0.63	20.36
6	1	73	0.83	25.73
7	1	99	0.95	28.25
8	1	120	1.03	29.79
9	1	14	0.54	18.33
10	1	49	0.77	24.96
11	1	78	0.86	26.23
12	1	101	0.95	28.07
13	1	-7	0.49	15.82
14	1	19	0.60	20.38
15	1	51	0.70	22.52
16	1	77	0.75	22.39

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	2	85	0.81	24.05
2	2	107	1.00	29.46
3	2	124	1.04	29.92
4	2	139	1.15	32.80
5	2	63	0.68	20.72
6	2	87	0.85	25.35
7	2	106	0.91	26.41
8	2	120	0.98	27.80
9	2	38	0.54	18.98
10	2	64	0.79	24.82
11	2	85	0.83	24.84
12	2	101	0.90	26.18
13	2	7	0.46	15.30
14	2	35	0.57	19.83
15	2	58	0.67	20.88
16	2	77	0.70	20.63

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	3	91	0.92	27.81
2	3	120	1.16	34.46
3	3	141	1.22	35.36
4	3	161	1.34	38.12
5	3	68	0.78	24.04
6	3	99	0.99	29.62
7	3	123	1.08	31.23
8	3	142	1.17	33.17
9	3	41	0.68	22.30
10	3	76	0.94	29.57
11	3	104	1.01	29.97
12	3	125	1.10	31.96
13	3	8	0.54	18.24
14	3	46	0.75	24.45
15	3	78	0.85	26.11
16	3	102	0.90	25.93

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	4	111	0.97	27.94
2	4	132	1.16	33.78
3	4	147	1.19	33.64
4	4	163	1.31	38.02
5	4	88	0.82	24.31
6	4	111	1.00	29.22
7	4	129	1.05	29.72
8	4	144	1.12	31.36
9	4	63	0.74	22.86
10	4	89	0.97	29.47
11	4	110	0.99	28.88
12	4	126	1.06	30.23
13	4	30	0.53	18.52
14	4	60	0.76	23.94
15	4	84	0.82	24.48
16	4	102	0.85	24.27

HALEY & ALDRICH	CALCULATIONS	File No.	40725-000
		Sheet	1 of 1
Client	MaineDOT	Date	30JUN2014
Project	Route 1 Viaduct Rehabilitation	Computed by	JLL
Subject	FB-MultiPier Results - Combined Internal Pile Stresses	Checked by	

Data File: C:\Bath\STR XML\2014-0701-HAI-Bath-Pier 10-MultiT-UnMod-STR-D2.XML

FB-MultiPier Results - Pier 10 - Pile Stresses for Multi-T UnModified HL-93 STR Load Cases

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	1	29	0.52	18.20
2	1	67	0.80	24.96
3	1	91	0.87	25.79
4	1	90	0.86	38.64
5	1	26	0.54	18.61
6	1	75	0.81	24.59
7	1	109	0.98	28.59
8	1	108	0.87	24.64
9	1	20	0.55	18.98
10	1	71	0.87	27.37
11	1	106	1.00	29.40
12	1	106	0.89	25.39
13	1	14	0.53	17.88
14	1	59	0.75	23.74
15	1	85	0.85	25.53
16	1	86	0.73	35.81

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	2	46	0.61	19.21
2	2	78	0.80	23.97
3	2	97	0.83	23.93
4	2	91	0.83	39.61
5	2	49	0.63	19.98
6	2	89	0.84	24.97
7	2	116	0.96	27.40
8	2	110	0.83	24.49
9	2	44	0.63	20.41
10	2	85	0.90	27.49
11	2	113	0.99	28.61
12	2	107	0.85	23.92
13	2	35	0.51	18.16
14	2	69	0.76	23.10
15	2	89	0.82	24.16
16	2	86	0.70	36.33

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	3	49	0.76	24.68
2	3	91	1.00	30.65
3	3	115	1.03	30.19
4	3	107	1.02	50.22
5	3	51	0.79	25.55
6	3	107	1.08	32.60
7	3	142	1.23	35.42
8	3	131	1.05	32.40
9	3	46	0.79	25.89
10	3	103	1.16	35.60
11	3	139	1.25	36.59
12	3	130	1.07	31.74
13	3	36	0.69	23.85
14	3	82	0.98	30.65
15	3	108	1.09	32.73
16	3	103	0.88	47.65

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	4	66	0.81	25.47
2	4	102	0.97	28.66
3	4	120	0.98	27.98
4	4	108	0.97	50.92
5	4	74	0.86	26.78
6	4	123	1.11	32.27
7	4	149	1.20	33.90
8	4	133	0.99	32.88
9	4	70	0.87	27.25
10	4	118	1.18	35.26
11	4	146	1.24	35.58
12	4	131	1.02	32.14
13	4	55	0.75	23.96
14	4	93	0.97	29.56
15	4	113	1.04	30.63
16	4	104	0.83	48.09

HALEY & ALDRICH	CALCULATIONS	File No.	40725-000
		Sheet	1 of 1
Client	MaineDOT	Date	30JUN2014
Project	Route 1 Viaduct Rehabilitation	Computed by	JLL
Subject	FB-MultiPier Results - Combined Internal Pile Stresses	Checked by	

Data File: C:\Bath\STR XML\2014-0701-HAI-Bath-Pier 12-MultiT-UnMod-STR-D2.XML

FB-MultiPier Results - Pier 12 - Pile Stresses for Multi-T UnModified HL-93 STR Load Cases

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	1	55	0.79	30.48
2	1	92	1.05	32.53
3	1	111	1.09	32.51
4	1	100	1.19	58.65
5	1	36	0.69	24.03
6	1	108	1.11	33.41
7	1	148	1.28	37.04
8	1	111	0.95	32.14
9	1	-4	0.63	20.56
10	1	62	0.90	29.09
11	1	116	1.14	33.89
12	1	96	0.91	27.04
13	1	-23	0.67	21.30
14	1	-1	0.70	22.96
15	1	47	0.81	26.83
16	1	63	0.68	35.87

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	2	70	0.75	40.40
2	2	101	1.01	30.36
3	2	116	1.03	33.96
4	2	101	1.16	58.57
5	2	58	0.77	24.39
6	2	127	1.15	33.59
7	2	155	1.27	36.01
8	2	112	0.91	32.59
9	2	24	0.61	20.90
10	2	84	0.96	29.71
11	2	124	1.12	32.84
12	2	96	0.88	26.66
13	2	-14	0.59	18.97
14	2	16	0.65	22.06
15	2	51	0.77	24.78
16	2	61	0.64	34.75

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	3	75	0.94	44.32
2	3	115	1.23	37.39
3	3	129	1.26	37.79
4	3	111	1.41	64.00
5	3	62	0.97	31.68
6	3	148	1.38	40.75
7	3	183	1.47	41.40
8	3	130	1.12	37.78
9	3	20	0.81	27.25
10	3	106	1.25	38.69
11	3	155	1.40	40.93
12	3	119	1.09	34.96
13	3	-19	0.81	26.13
14	3	27	0.89	30.21
15	3	72	1.07	34.49
16	3	82	0.83	49.08

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	4	88	0.93	53.59
2	4	125	1.20	36.74
3	4	133	1.22	38.58
4	4	113	1.40	64.46
5	4	84	1.06	33.20
6	4	166	1.42	40.95
7	4	191	1.48	41.33
8	4	131	1.10	38.21
9	4	50	0.87	28.76
10	4	127	1.30	39.30
11	4	163	1.41	40.62
12	4	120	1.06	35.40
13	4	-8	0.74	24.10
14	4	42	0.88	29.64
15	4	76	1.03	32.76
16	4	80	0.80	48.45

HALEY & ALDRICH	CALCULATIONS	File No.	40725-000
		Sheet	1 of 1
Client	MaineDOT	Date	30JUN2014
Project	Route 1 Viaduct Rehabilitation	Computed by	JLL
Subject	FB-MultiPier Results - Combined Internal Pile Stresses	Checked by	

Data File: C:\Bath\STR XML\2014-0701-HAI-Bath-Pier 15-MultiT-UnMod-STR-D2.XML

FB-MultiPier Results - Pier 15 - Pile Stresses for Multi-T UnModified HL-93 STR Load Cases

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	1	71	0.96	30.73
2	1	114	1.26	38.80
3	1	141	1.34	39.71
4	1	150	1.39	41.02
5	1	45	0.80	26.65
6	1	91	1.10	34.28
7	1	124	1.23	36.91
8	1	143	1.30	38.01
9	1	7	0.70	23.48
10	1	59	1.01	33.26
11	1	99	1.14	35.18
12	1	123	1.22	36.71
13	1	-14	0.67	21.81
14	1	16	0.80	26.99
15	1	61	0.91	29.48
16	1	87	0.95	28.99

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	2	94	1.00	30.31
2	2	128	1.26	37.67
3	2	148	1.30	37.70
4	2	152	1.35	39.33
5	2	74	0.88	27.21
6	2	108	1.11	33.63
7	2	131	1.20	35.21
8	2	145	1.25	36.21
9	2	40	0.74	24.52
10	2	79	1.04	33.01
11	2	107	1.11	33.66
12	2	124	1.17	34.59
13	2	-1	0.61	20.10
14	2	37	0.76	26.16
15	2	70	0.88	27.63
16	2	87	0.87	26.74

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	3	99	1.17	36.27
2	3	142	1.42	42.75
3	3	167	1.47	42.70
4	3	172	1.49	43.18
5	3	78	1.05	33.46
6	3	121	1.29	39.24
7	3	150	1.37	40.20
8	3	169	1.42	40.56
9	3	39	0.90	30.51
10	3	92	1.25	39.69
11	3	128	1.33	40.07
12	3	150	1.37	40.32
13	3	-3	0.79	25.82
14	3	49	0.99	33.23
15	3	93	1.14	35.72
16	3	116	1.15	34.54

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	4	122	1.19	35.61
2	4	156	1.42	41.65
3	4	174	1.43	40.73
4	4	175	1.49	42.62
5	4	105	1.12	34.12
6	4	136	1.31	38.95
7	4	158	1.36	39.08
8	4	171	1.40	39.76
9	4	70	0.99	31.56
10	4	109	1.27	39.54
11	4	135	1.31	38.96
12	4	151	1.35	39.27
13	4	21	0.75	25.42
14	4	67	1.00	32.26
15	4	101	1.11	33.95
16	4	115	1.09	32.75

HALEY & ALDRICH	CALCULATIONS	File No.	40725-000
		Sheet	1 of 1
Client	MaineDOT	Date	30JUN2014
Project	Route 1 Viaduct Rehabilitation	Computed by	JLL
Subject	FB-MultiPier Results - Combined Internal Pile Stresses	Checked by	

Data File: C:\Bath\STR XML\2014-0701-HAI-Bath-Pier 19-MultiT-UnMod-STR-D2.XML

FB-MultiPier Results - Pier 19 - Pile Stresses for Multi-T UnModified HL-93 STR Load Cases

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	1	44	0.72	23.46
2	1	77	0.95	29.60
3	1	96	1.02	31.03
4	1	101	1.02	32.10
5	1	28	0.60	20.83
6	1	82	0.94	28.97
7	1	120	1.11	32.62
8	1	102	0.98	29.10
9	1	3	0.56	18.55
10	1	59	0.87	28.06
11	1	102	1.01	30.37
12	1	95	0.95	28.54
13	1	-17	0.52	16.48
14	1	27	0.65	22.50
15	1	62	0.80	25.28
16	1	79	0.79	24.72

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	2	62	0.76	23.71
2	2	87	0.95	28.92
3	2	100	0.98	29.28
4	2	103	0.98	32.80
5	2	50	0.67	21.76
6	2	100	0.97	28.83
7	2	129	1.08	31.24
8	2	103	0.93	27.55
9	2	30	0.56	19.80
10	2	77	0.89	27.62
11	2	110	1.00	29.41
12	2	95	0.91	26.91
13	2	-1	0.49	16.38
14	2	41	0.67	22.12
15	2	66	0.76	23.65
16	2	78	0.74	23.41

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	3	63	0.90	29.06
2	3	96	1.14	35.60
3	3	113	1.19	36.06
4	3	114	1.17	40.55
5	3	49	0.81	26.60
6	3	116	1.18	35.37
7	3	155	1.30	37.62
8	3	118	1.15	34.17
9	3	25	0.72	24.59
10	3	93	1.12	34.90
11	3	138	1.26	36.88
12	3	112	1.13	33.90
13	3	-7	0.65	21.09
14	3	51	0.86	28.35
15	3	83	0.97	30.31
16	3	96	0.95	29.69

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	4	81	0.94	29.09
2	4	105	1.14	34.73
3	4	118	1.16	34.49
4	4	117	1.14	41.83
5	4	71	0.87	27.85
6	4	134	1.22	36.01
7	4	164	1.30	37.67
8	4	120	1.11	33.14
9	4	50	0.78	25.90
10	4	110	1.16	35.15
11	4	145	1.26	36.78
12	4	113	1.09	32.67
13	4	17	0.63	21.97
14	4	61	0.87	27.87
15	4	87	0.94	29.38
16	4	96	0.91	29.43

NEXT FB-MULTIPIER RESULTS

HALEY & ALDRICH	CALCULATIONS	File No.	40725-000
		Sheet	1 of 1
Client	MaineDOT	Date	14AUG2014
Project	Route 1 Viaduct Rehabilitation	Computed by	JLL
Subject	FB-MultiPier Results - Combined Internal Pile Stresses	Checked by	

Data File: G:\40725\000\FB-MPier\2014-0611\STR\2014-0814\2014-0814-HAI-Bath-Pier 7-NEXT-UnMod-STR-D2.XML

FB-MultiPier Results - Pier 7 - Pile Stresses for NEXT UnModified HL-93 STR Load Cases

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	1	71	0.81	24.92
2	1	101	1.04	31.27
3	1	123	1.12	32.73
4	1	143	1.24	35.76
5	1	46	0.66	21.17
6	1	78	0.87	26.66
7	1	104	0.98	29.08
8	1	124	1.06	30.61
9	1	19	0.56	19.12
10	1	53	0.80	25.92
11	1	82	0.88	26.96
12	1	105	0.98	28.92
13	1	-6	0.50	16.23
14	1	23	0.62	21.12
15	1	54	0.73	23.20
16	1	80	0.77	23.05

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	2	91	0.85	24.99
2	2	113	1.04	30.56
3	2	129	1.08	30.91
4	2	144	1.19	33.88
5	2	68	0.71	21.56
6	2	92	0.88	26.29
7	2	110	0.95	27.26
8	2	125	1.01	28.63
9	2	42	0.61	19.79
10	2	68	0.83	25.80
11	2	89	0.86	25.73
12	2	105	0.93	27.03
13	2	10	0.47	15.81
14	2	38	0.59	20.59
15	2	61	0.70	21.58
16	2	80	0.73	21.30

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	3	99	0.98	29.15
2	3	127	1.21	35.90
3	3	148	1.27	36.59
4	3	168	1.38	39.33
5	3	74	0.82	25.25
6	3	106	1.04	30.92
7	3	129	1.12	32.38
8	3	148	1.21	34.35
9	3	46	0.72	23.48
10	3	82	0.99	30.99
11	3	109	1.05	31.24
12	3	130	1.14	33.18
13	3	12	0.57	19.13
14	3	51	0.79	25.58
15	3	83	0.89	27.15
16	3	108	0.93	26.98

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	4	119	1.02	29.38
2	4	139	1.22	35.27
3	4	154	1.24	34.95
4	4	170	1.35	39.88
5	4	95	0.87	25.50
6	4	118	1.05	30.56
7	4	135	1.09	30.92
8	4	150	1.17	32.65
9	4	68	0.78	23.94
10	4	95	1.02	30.93
11	4	115	1.03	30.19
12	4	131	1.10	31.50
13	4	34	0.56	19.46
14	4	65	0.80	25.12
15	4	89	0.86	25.54
16	4	107	0.89	25.34

HALEY & ALDRICH	CALCULATIONS	File No.	40725-000
		Sheet	1 of 1
Client	MaineDOT	Date	14AUG2014
Project	Route 1 Viaduct Rehabilitation	Computed by	JLL
Subject	FB-MultiPier Results - Combined Internal Pile Stresses	Checked by	

Data File: G:\40725\000\FB-MPier\2014-0611\STR\2014-0814\2014-0814-HAI-Bath-Pier 10-NEXT-UnMod-STR-D2.XML

FB-MultiPier Results - Pier 10 - Pile Stresses for NEXT UnModified HL-93 STR Load Cases

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	1	33	0.56	19.41
2	1	72	0.83	25.89
3	1	95	0.90	26.65
4	1	93	0.89	40.74
5	1	30	0.57	19.88
6	1	80	0.86	26.07
7	1	115	1.03	29.92
8	1	112	0.90	25.56
9	1	24	0.59	20.23
10	1	76	0.92	28.81
11	1	112	1.05	30.83
12	1	110	0.92	26.34
13	1	17	0.56	18.93
14	1	63	0.79	24.82
15	1	89	0.88	26.48
16	1	88	0.76	37.87

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	2	50	0.64	20.32
2	2	82	0.83	24.94
3	2	101	0.86	24.79
4	2	94	0.85	41.68
5	2	53	0.68	21.29
6	2	95	0.89	26.20
7	2	122	1.01	28.67
8	2	114	0.86	25.86
9	2	48	0.68	21.69
10	2	90	0.95	28.85
11	2	119	1.04	29.97
12	2	111	0.88	25.14
13	2	38	0.54	19.20
14	2	73	0.79	24.26
15	2	93	0.86	25.41
16	2	89	0.72	38.32

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	3	54	0.82	26.53
2	3	97	1.05	31.96
3	3	120	1.08	31.53
4	3	110	1.06	52.77
5	3	58	0.85	27.61
6	3	116	1.16	34.69
7	3	150	1.26	36.24
8	3	137	1.10	34.35
9	3	52	0.86	28.09
10	3	111	1.22	37.32
11	3	147	1.29	37.48
12	3	135	1.12	33.86
13	3	41	0.77	25.57
14	3	88	1.04	32.42
15	3	114	1.15	34.56
16	3	107	0.93	50.24

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	4	71	0.86	26.85
2	4	107	1.00	29.51
3	4	126	1.02	29.76
4	4	112	1.00	53.49
5	4	80	0.93	28.66
6	4	131	1.17	34.24
7	4	157	1.25	35.35
8	4	139	1.04	34.56
9	4	76	0.93	29.21
10	4	126	1.24	36.96
11	4	154	1.29	36.95
12	4	137	1.07	34.06
13	4	60	0.80	25.49
14	4	98	1.02	31.03
15	4	118	1.11	32.60
16	4	108	0.86	50.68

HALEY & ALDRICH	CALCULATIONS	File No.	40725-000
		Sheet	1 of 1
Client	MaineDOT	Date	14AUG2014
Project	Route 1 Viaduct Rehabilitation	Computed by	JLL
Subject	FB-MultiPier Results - Combined Internal Pile Stresses	Checked by	

Data File: G:\40725\000\FB-MPier\2014-0611\STR\2014-0814\2014-0814-HAI-Bath-Pier 12-NEXT-UnMod-STR-D2.XML

FB-MultiPier Results - Pier 12 - Pile Stresses for NEXT UnModified HL-93 STR Load Cases

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	1	60	0.82	33.72
2	1	97	1.09	33.65
3	1	116	1.12	33.91
4	1	103	1.24	59.20
5	1	42	0.77	25.48
6	1	116	1.17	35.07
7	1	155	1.32	38.07
8	1	115	0.98	33.61
9	1	-1	0.65	21.50
10	1	69	0.96	30.79
11	1	122	1.19	35.32
12	1	100	0.95	28.09
13	1	-22	0.69	22.17
14	1	1	0.73	23.96
15	1	50	0.85	27.97
16	1	65	0.70	37.28

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	2	74	0.79	43.76
2	2	107	1.05	31.48
3	2	120	1.07	35.20
4	2	104	1.22	60.67
5	2	64	0.83	26.23
6	2	134	1.22	35.69
7	2	162	1.33	37.68
8	2	117	0.95	34.22
9	2	28	0.65	22.48
10	2	91	1.03	31.85
11	2	130	1.19	34.85
12	2	100	0.92	28.08
13	2	-14	0.63	20.24
14	2	18	0.69	23.35
15	2	53	0.81	26.24
16	2	63	0.67	36.49

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	3	81	1.01	48.70
2	3	122	1.30	39.49
3	3	134	1.33	39.79
4	3	115	1.48	64.85
5	3	70	1.08	34.95
6	3	159	1.45	42.35
7	3	193	1.51	42.28
8	3	134	1.20	38.68
9	3	25	0.88	30.00
10	3	116	1.33	41.09
11	3	165	1.46	42.40
12	3	124	1.16	36.60
13	3	-20	0.89	28.60
14	3	30	0.96	32.68
15	3	77	1.15	37.21
16	3	86	0.89	51.85

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	4	94	0.99	57.20
2	4	130	1.28	38.18
3	4	137	1.29	39.35
4	4	116	1.49	65.39
5	4	92	1.16	36.64
6	4	178	1.49	42.53
7	4	202	1.53	42.44
8	4	135	1.18	39.10
9	4	55	0.96	31.74
10	4	138	1.39	41.71
11	4	173	1.48	42.42
12	4	125	1.14	37.03
13	4	-9	0.81	26.46
14	4	45	0.95	32.15
15	4	80	1.12	35.70
16	4	84	0.86	51.21

HALEY & ALDRICH	CALCULATIONS	File No.	40725-000
		Sheet	1 of 1
Client	MaineDOT	Date	14AUG2014
Project	Route 1 Viaduct Rehabilitation	Computed by	JLL
Subject	FB-MultiPier Results - Combined Internal Pile Stresses	Checked by	

Data File: G:\40725\000\FB-MPier\2014-0611\STR\2014-0814\2014-0814-HAI-Bath-Pier 15-NEXT-UnMod-STR-D2.XML

FB-MultiPier Results - Pier 15 - Pile Stresses for NEXT UnModified HL-93 STR Load Cases

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	1	82	1.06	33.30
2	1	125	1.33	40.46
3	1	150	1.41	41.48
4	1	158	1.44	42.33
5	1	53	0.91	29.85
6	1	100	1.19	37.22
7	1	131	1.31	39.14
8	1	150	1.36	39.57
9	1	9	0.78	26.07
10	1	64	1.10	36.22
11	1	103	1.21	37.43
12	1	128	1.28	38.41
13	1	-15	0.73	23.53
14	1	14	0.85	28.40
15	1	61	0.98	31.90
16	1	89	0.99	30.71

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	2	105	1.09	33.15
2	2	138	1.33	39.66
3	2	157	1.36	39.47
4	2	159	1.41	41.05
5	2	82	0.97	30.15
6	2	116	1.20	36.40
7	2	138	1.27	37.42
8	2	151	1.32	38.01
9	2	43	0.81	26.95
10	2	83	1.12	35.69
11	2	111	1.17	35.64
12	2	128	1.23	36.48
13	2	-3	0.67	21.97
14	2	35	0.80	27.70
15	2	70	0.93	29.58
16	2	88	0.92	28.45

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	3	112	1.27	39.23
2	3	154	1.51	44.92
3	3	177	1.51	43.37
4	3	181	1.55	44.43
5	3	88	1.16	36.92
6	3	131	1.37	41.52
7	3	159	1.43	41.85
8	3	177	1.46	41.68
9	3	45	1.00	33.77
10	3	99	1.33	42.29
11	3	135	1.39	41.82
12	3	156	1.43	41.93
13	3	-5	0.86	28.25
14	3	50	1.06	35.98
15	3	96	1.22	38.31
16	3	119	1.21	36.78

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	4	134	1.30	38.57
2	4	167	1.49	43.21
3	4	184	1.46	40.98
4	4	185	1.55	44.15
5	4	115	1.23	37.31
6	4	146	1.39	41.24
7	4	166	1.42	41.06
8	4	180	1.46	41.34
9	4	76	1.09	34.83
10	4	116	1.35	42.00
11	4	142	1.38	40.98
12	4	158	1.41	41.16
13	4	19	0.83	27.92
14	4	68	1.08	35.34
15	4	103	1.18	36.45
16	4	118	1.16	35.09

HALEY & ALDRICH	CALCULATIONS	File No.	40725-000
		Sheet	1 of 1
Client	MaineDOT	Date	14AUG2014
Project	Route 1 Viaduct Rehabilitation	Computed by	JLL
Subject	FB-MultiPier Results - Combined Internal Pile Stresses	Checked by	

Data File: G:\40725\000\FB-MPier\2014-0611\STR\2014-0814\2014-0814-HAI-Bath-Pier 19-NEXT-UnMod-STR-D2.XML

FB-MultiPier Results - Pier 19 - Pile Stresses for NEXT UnModified HL-93 STR Load Cases

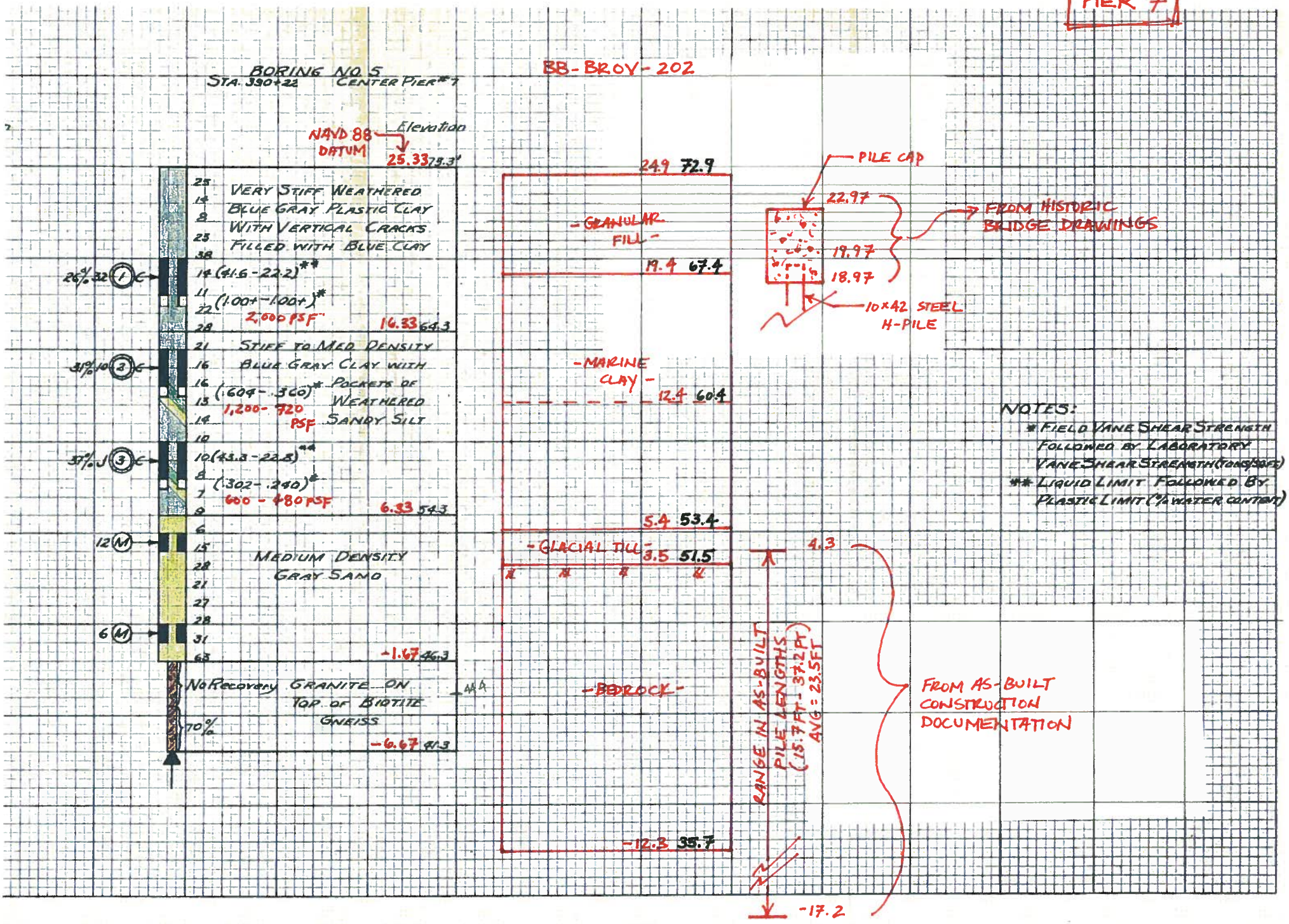
Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	1	51	0.79	25.58
2	1	83	1.02	31.83
3	1	101	1.08	32.87
4	1	105	1.07	34.12
5	1	33	0.64	22.37
6	1	90	1.00	30.76
7	1	128	1.16	34.03
8	1	105	1.02	30.50
9	1	6	0.59	19.63
10	1	64	0.90	29.01
11	1	108	1.07	32.03
12	1	97	0.99	29.88
13	1	-18	0.55	17.56
14	1	28	0.70	23.87
15	1	63	0.84	26.55
16	1	80	0.82	25.92

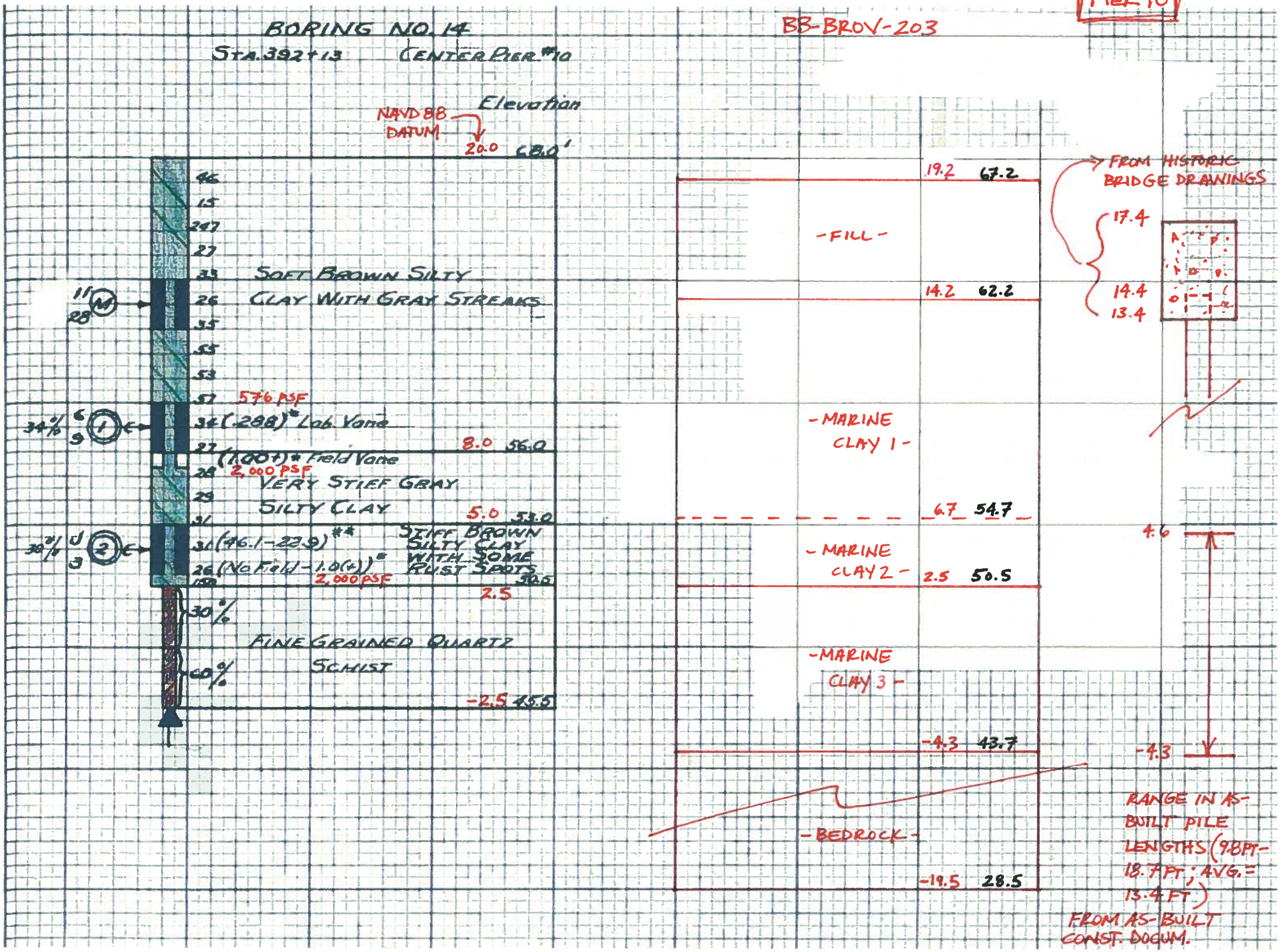
Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	2	69	0.82	25.45
2	2	92	1.01	30.78
3	2	105	1.04	31.01
4	2	106	1.02	34.96
5	2	55	0.72	23.48
6	2	107	1.03	30.61
7	2	137	1.15	33.28
8	2	107	0.98	29.08
9	2	33	0.60	21.33
10	2	82	0.94	29.18
11	2	116	1.06	31.21
12	2	98	0.95	28.35
13	2	-2	0.52	17.42
14	2	41	0.71	23.27
15	2	68	0.80	24.84
16	2	80	0.77	24.58

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	3	71	0.99	31.68
2	3	103	1.22	37.99
3	3	118	1.26	38.14
4	3	118	1.22	43.55
5	3	56	0.89	29.20
6	3	127	1.25	37.42
7	3	165	1.34	39.06
8	3	122	1.20	36.03
9	3	30	0.78	26.77
10	3	100	1.19	37.03
11	3	146	1.31	38.57
12	3	116	1.19	35.78
13	3	-8	0.70	22.75
14	3	53	0.92	30.44
15	3	86	1.04	32.48
16	3	99	1.00	31.71

Pile Element	Load Case	N [kip]	Ratio	Max. σ [ksi]
1	4	88	1.02	31.80
2	4	112	1.22	37.21
3	4	123	1.23	36.69
4	4	121	1.20	45.00
5	4	77	0.95	30.50
6	4	145	1.28	38.23
7	4	175	1.35	39.31
8	4	124	1.18	35.43
9	4	54	0.85	28.40
10	4	118	1.23	37.25
11	4	154	1.32	38.74
12	4	117	1.16	34.99
13	4	15	0.68	23.81
14	4	63	0.93	30.00
15	4	89	1.00	31.56
16	4	98	0.96	30.73

SOIL PROFILE AND PILE ELEVATION INFORMATION

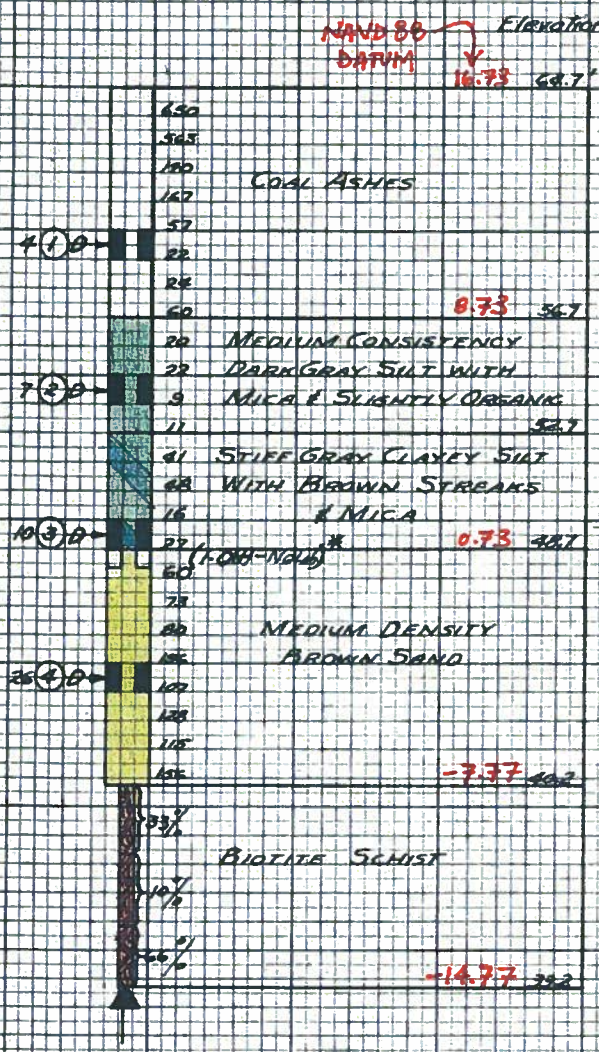



PIER 10
BB-BROV-203
BORING NO. 14
STA. 392+13 CENTER PIER #10
NAVD 88 DATUM
Elevation
20.0 68.0'
46
15
247
27
33 SOFT BROWN SILTY
26 CLAY WITH GRAY STREAKS
35
55
53
57 576 PSF
34 (.288)* Lab. Vane
27 (1.00+)* Field Vane
8.0 56.0
28 2,000 PSF
29 VERY STIFF GRAY
SILTY CLAY
31 5.0 53.0
31 (46.1-22.9)** STIFF BROWN SILTY CLAY WITH SOME RUST SPOTS
26 (No Field-1.0(+))* 2,000 PSF
30.5
2.5
30%
FINE GRAINED QUARTZ
SCHIST
60%
-2.5 45.5
11
28
34%
30%
-FILL-
19.2 67.2
17.4
14.2 62.2
14.4
13.4
-MARINE CLAY 1-
6.7 54.7
4.6
-MARINE CLAY 2-
2.5 50.5
-MARINE CLAY 3-
-4.3 43.7
-4.3
-BEDROCK-
-19.5 28.5
FROM HISTORIC BRIDGE DRAWINGS
RANGE IN AS-BUILT PILE LENGTHS (9.8 FT-18.7 FT; AVG.= 13.4 FT)
FROM AS-BUILT CONST. DOCUM.

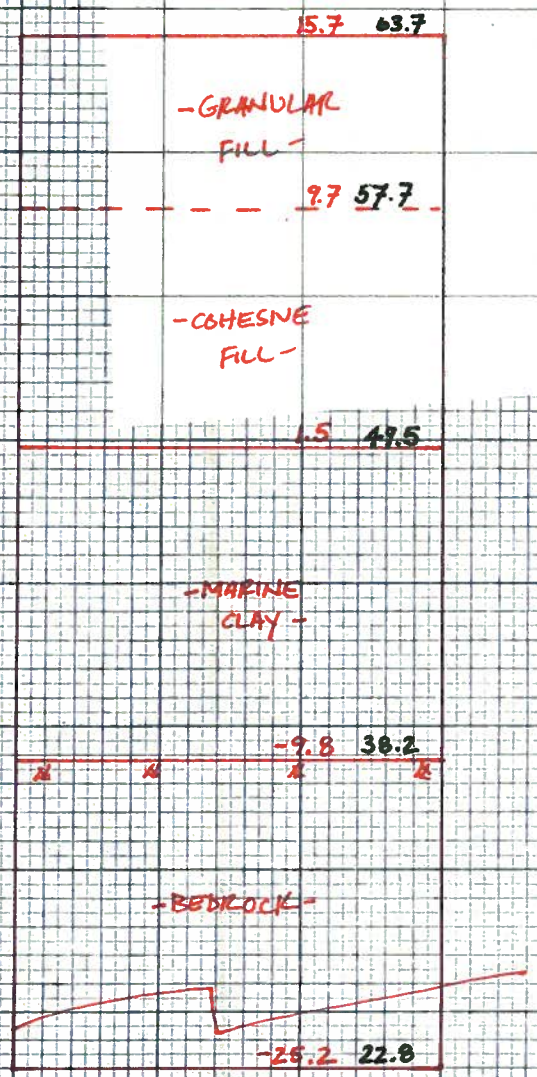
PIER 12

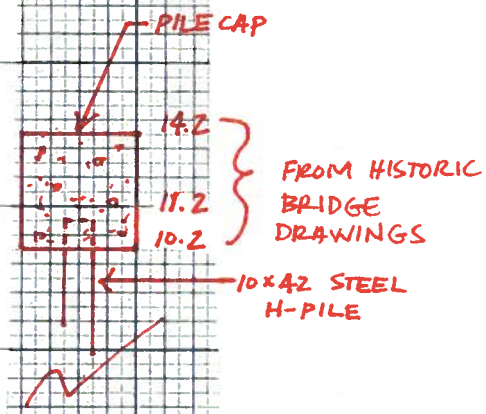
BORING NO. 7

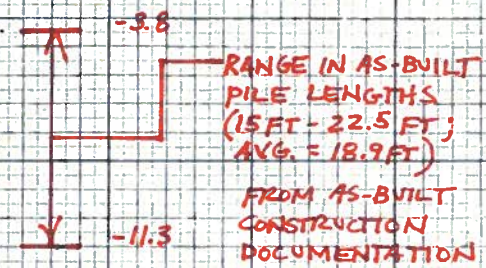
STA. 393740 CENTER PIER #12

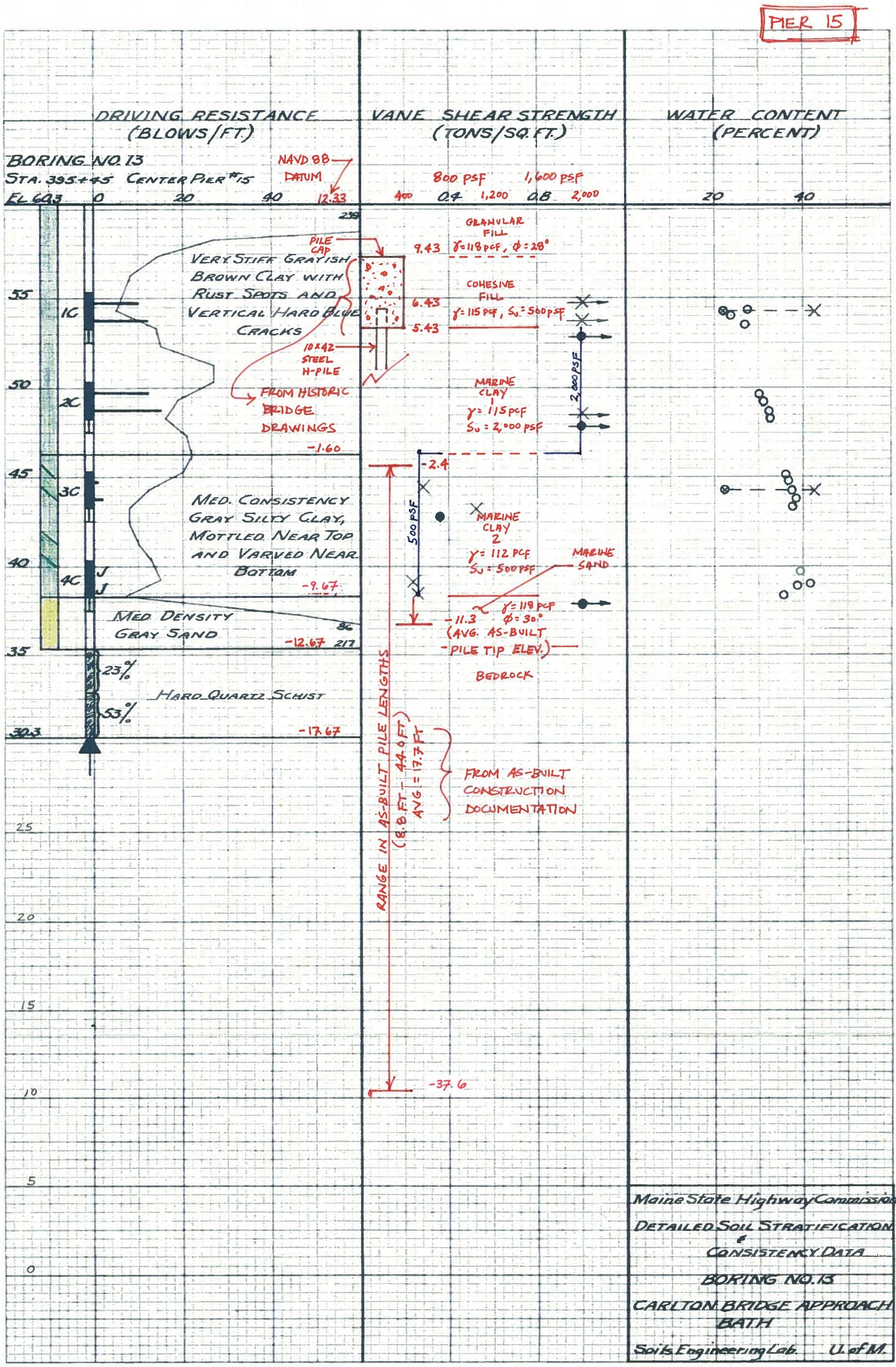


BB-BROV-204








PIER 15
DRIVING RESISTANCE (BLOWS/FT.)
VANE SHEAR STRENGTH (TONS/SQ. FT.)
WATER CONTENT (PERCENT)
BORING NO. 13
STA. 395+45 CENTER PIER #15
NAVD 88 DATUM
EL 6.43
12.33
800 PSF
1,600 PSF
GRANULAR FILL
9.43
γ=118 PCF, φ=28°
PILE CAP
VERY STIFF GRAYISH BROWN CLAY WITH RUST SPOTS AND VERTICAL HARD BLUE CRACKS
COHESIVE FILL
6.43
γ=115 PCF, Su=500 PSF
5.43
10x42 STEEL H-PILE
FROM HISTORIC BRIDGE DRAWINGS
MARINE CLAY 1
γ=115 PCF
Su=2,000 PSF
2,000 PSF
-1.60
-2.4
MED. CONSISTENCY GRAY SILTY CLAY, MOTTLED NEAR TOP AND VARVED NEAR BOTTOM
500 PSF
MARINE CLAY 2
γ=112 PCF
Su=500 PSF
MARINE SAND
γ=118 PCF
φ=30°
-9.67
MED. DENSITY GRAY SAND
-11.3
(AVG. AS-BUILT PILE TIP ELEV.)
-12.67
BEDROCK
23%
HARD QUARTZ SCHIST
53%
-17.67
RANGE IN AS-BUILT PILE LENGTHS
(8.8 FT – 44.0 FT)
AVG = 17.7 FT
FROM AS-BUILT CONSTRUCTION DOCUMENTATION
-37.6
Maine State Highway Commission
DETAILED SOIL STRATIFICATION & CONSISTENCY DATA
BORING NO. 13
CARLTON BRIDGE APPROACH
BATH
Soils Engineering Lab. U. of M.
SHEET #6

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Route 1 Viaduct Bridge Rehabilitation
Location: Bath, Maine

DRAFT

Boring No.: BB-BROV-205
PIN: 19273.00

PIER 19

Driller:	MaineDOT	**Elevation (ft.)**	8.6	**Auger ID/OD:**	--
Operator:	C. Giles	**Datum:**	NAVD 88	**Sampler:**	Split Spoon-1.375 in. ID
Logged By:	S. Shay	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	HW-140#/30 in. SS-140#/30
Date Start/Finish:	4-07-2014/4-08-2014	**Drilling Method:**	SSA; HW	**Core Barrel:**	NQ-2.0 in.
Boring Location:	E1131830, N393542.7 (See Plan)	**Casing ID/OD:**	SSA-2.25 in. ID; HW-4.0 in. ID	**Water Level*:**	2.5
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.)
0	1D	24/8	0.0 - 2.0	1/2/1/2	3	3		
5	2D	24/24	5.0 - 7.0	1/1/1/7	2	2	10	
							18	
							34	
							26	
10	3D	24/24	9.0 - 11.0	3/5/5/7	10	10	52	
							65	
							92	
							96	
							88	
15							72	
							51	
							57	
							59	
							63	
20	4D	24/24	19.0 - 21.0	WOH(24.0)			54	
							35	
							39	
							32	
							25	
25							25	
							31	
							36	
							29	
							27	
30	5D	24/17	29.0 - 31.0	WOH(24.0)			48	
							60	
							67	
							66	
							55	
35							56	
							71	
							67	
							62	
							56	
40							63	
							65	
							68	
							73	
							128	-34.80
45	6D	11/9	44.5 - 45.4	22/100(5.0)			96(6.0)	-36.40
	R1	60/57	46.5 - 51.5	RQD = 25*%				-37.90
50								
	R2	60/60	51.5 - 56.5	RQD = 57%				
55								
	R3	60/60	56.5 - 61.5	RQD = 95%				
60								
								-52.90
65								

Graphic Log / Visual Description and Remarks

Brown, very loose (may have been disturbed by excavator), coarse to fine SAND
-FILl-(SP)

6.60 — 2.0

Note: Viewed color/stratum change with hand dig to 3.0 ft.

Black to dark olive-gray, very soft, ORGANIC SOIL, some medium to fine sand, 5% wood fibers, no odor
-ORGANIC DEPOSITS-(OL/OH)

2.10 — 6.5

Medium stiff, mottled, silty CLAY (CL)

Olive-gray with weak mottling, stiff, silty CLAY (CL)
Pocket Penetrometer = 3.5 ton/ft2

Dark gray, very soft, CLAY, little silt (CL)

Dark gray, very soft, CLAY, little silt (CL)

-29.5
ROD PROBE REFUSAL ELEVATION

-34.80 — 43.4

Note: Abrupt chane in effort advance casing at 43.4 ft.
Pink-gray, very dense, coarse to fine SAND, some gravel, grading to severely to completely weathered rock with gravel
-GLACIAL TILL-(SP)

-36.40 — 45.0

-37.90

Top of Bedrock at El. -36.4
Moderately to highly weathered ROCK
Note: Weathered zone to 46.5 ft. Continued to advance borehole with roller bit to 46.5 ft. Advanced casing to seat in bedrock at 46.5 ft prior to coring.

46.5

Very hard to hard, moderately weathered gray/gray-black, medium to fine grained pegmatitic SCHIST. Irregular foliatoin joints dipping at low angles, very close, rough, planar, discolored or micaceouse surfaces, tight to open.
Rock Mass Quality=_____
Recovery=95%
R1 Core Times (min:sec):
46.5-47.5' (7:00); 47.5-48.5' (10:00); 48.5 49.5' (8:00); 49.5-50.5' (8:00); 50.5'51.5' (8:00)
Similar to above with intermittant planar foliation. Slightly weathered white/gray pegmatitic SCHIST, low angle joints, close.
Rock Mass Quality=_____
Recovery=100%
R2 Core Times (min:sec):
51.5-52.5' (7:00); 52.5-53.5' (6:00); 53.5-54.5' (6:00); 54.5-55.5' (7:00); 55.5-56.5' (7:00)
Very hard, fresh, gray with light green tint, medium grained pegmatitic SCHIST. Weak, planar foliation. Joints dipping at low angles, moderately spaced, rough, planar at fresh micaceous zones, tight. Occasional rehealed high angled secondary joint present.
Minimal water loss R1 - R3 during coring.
Rock Mass Quality=_____
Recovery=100%
R3 Core times (min:sec):
56.5-57.5' (6:00); 57.5-58.5' (6:00); 58.5-59.5' (6:00); 59.5-60.5' (6:00); 60.5'61.5' (7:00)

61.5

Bottom of Exploration at 61.5 feet below ground surface.

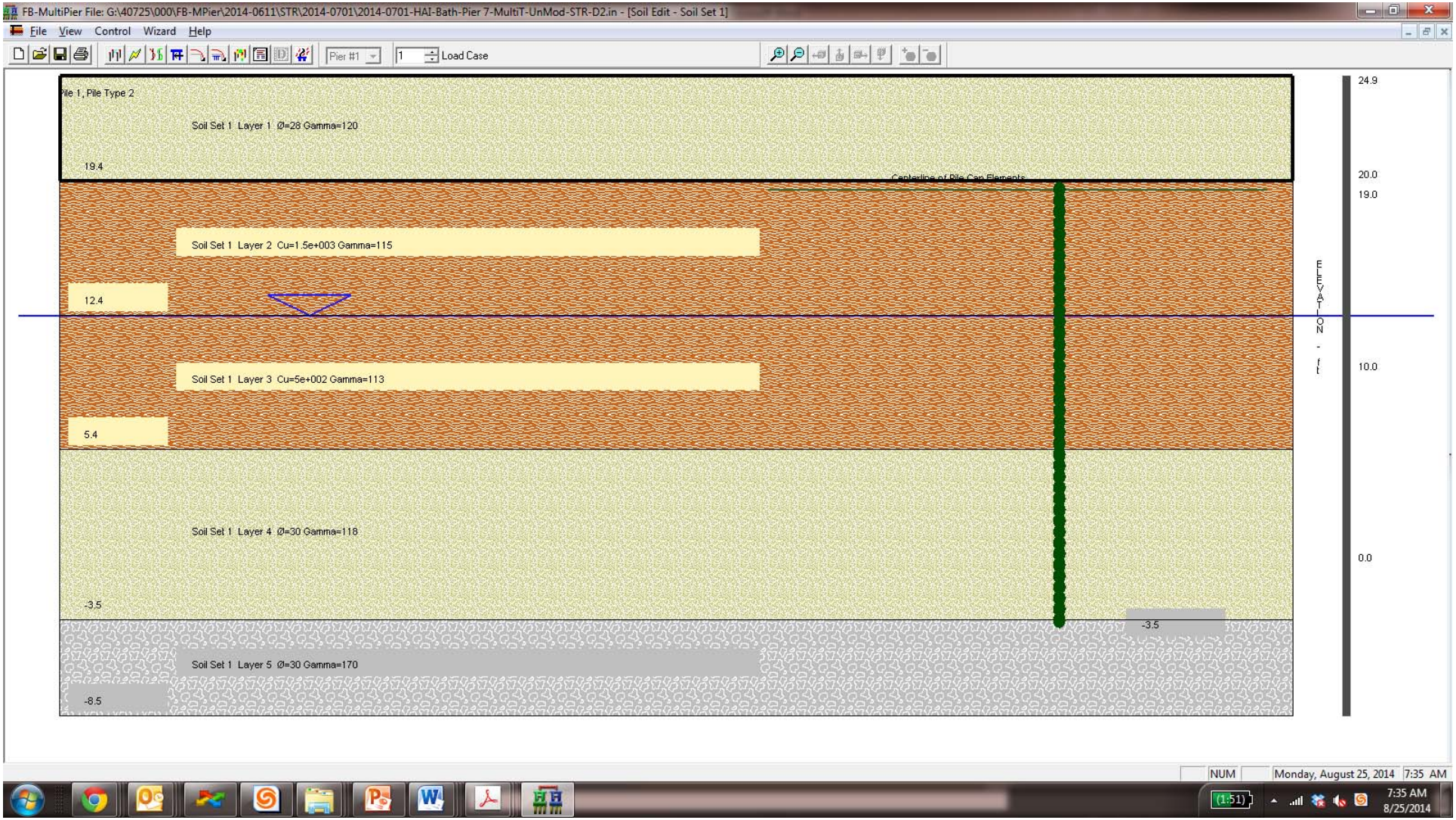
Note: * RQD modifier for moderately weathered rock.

Laboratory Testing Results/ AASHTO and Unified Class.

6.8
3.8
2.8

-27.5
RANGE IN AS-BUILT PILE LENGTHS (31.3 FT - 46.2 FT; AVG. = 37.6 FT)
- 42.4

PIER 7 FB-MultiPier Model



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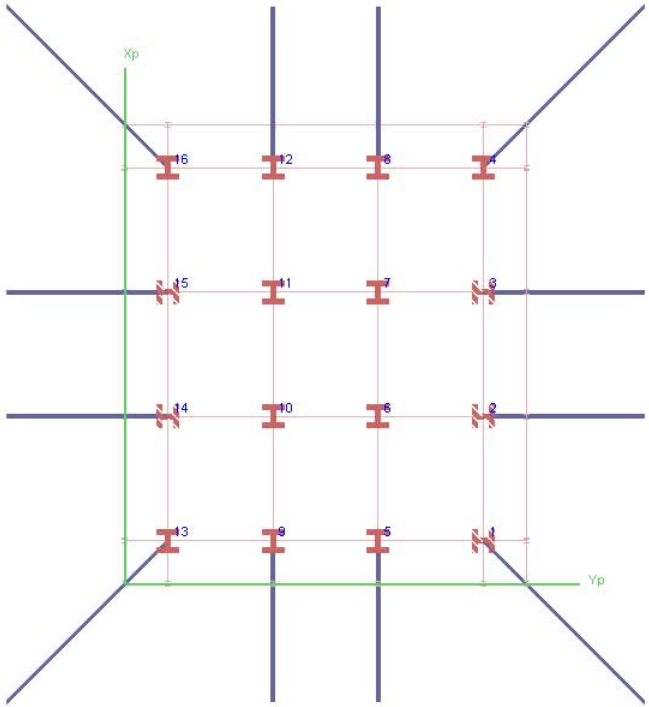
File View Control Wizard Help

Pier #1 | 1 | Load Case



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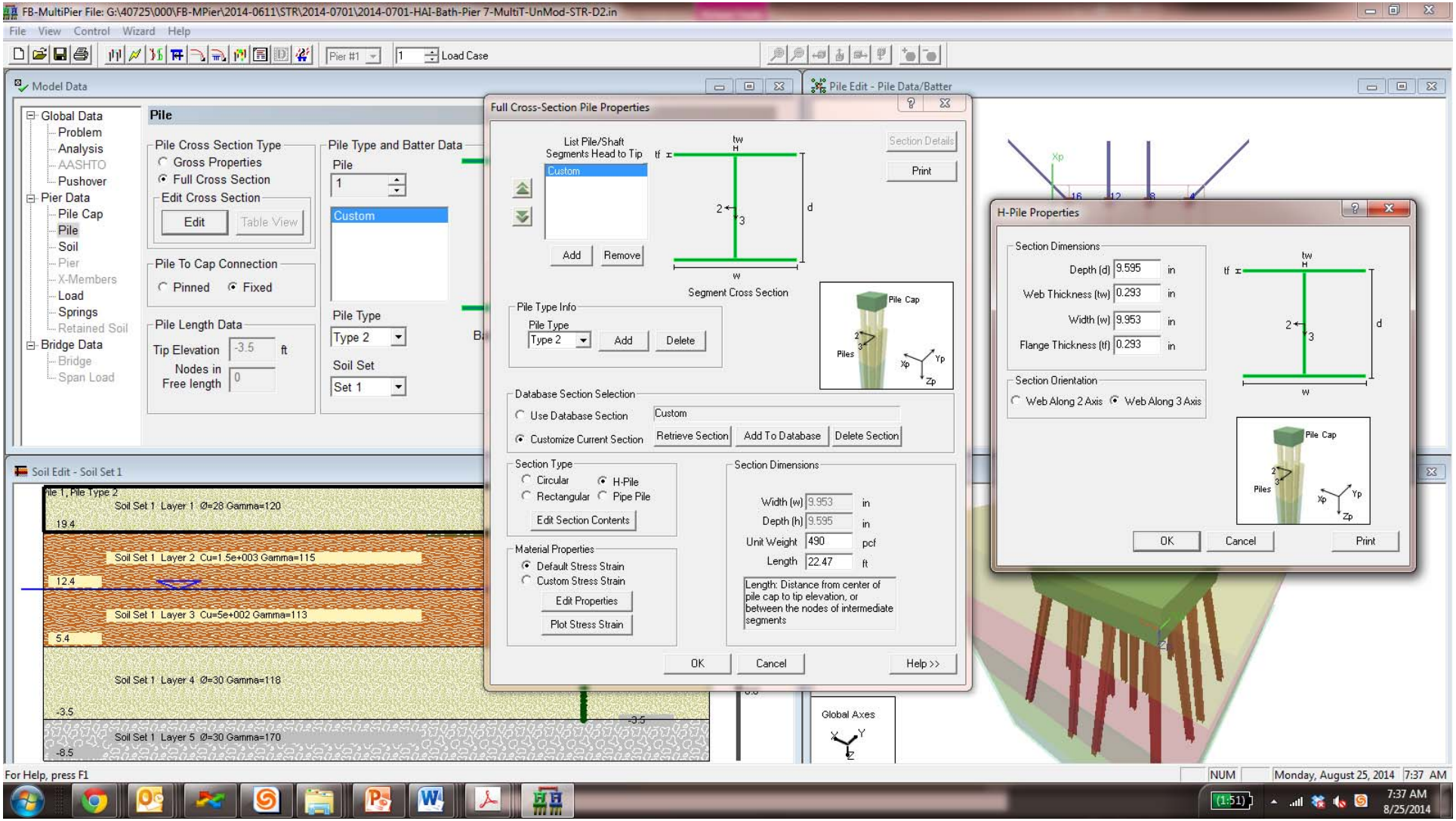
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File View Control Wizard Help
Pier #1
Load Case
Model Data
Global Data
Problem
Analysis
AASHTO
Pushover
Pier Data
Pile Cap
Pile
Soil
Pier
X-Members
Load
Springs
Retained Soil
Bridge Data
Bridge
Span Load
Pile
Pile Cross Section Type
Gross Properties
Full Cross Section
Edit Cross Section
Edit
Table View
Pile To Cap Connection
Pinned
Fixed
Pile Length Data
Tip Elevation
-3.5
ft
Nodes in Free length
0
Pile Type and Batter Data
Pile
1
Custom
Pile Type
Type 2
Soil Set
Set 1
Full Cross-Section Pile Properties
List Pile/Shaft Segments Head to Tip
Custom
Add
Remove
Section Details
Print
Segment Cross Section
Pile Type Info
Pile Type
Type 2
Add
Delete
Pile Cap
Piles
Database Section Selection
Use Database Section
Custom
Customize Current Section
Retrieve Section
Add To Database
Delete Section
Section Type
Circular
H-Pile
Rectangular
Pipe Pile
Edit Section Contents
Section Dimensions
Width (w) 9.953 in
Depth (h) 9.595 in
Unit Weight 490 pcf
Length 22.47 ft
Length: Distance from center of pile cap to tip elevation, or between the nodes of intermediate segments
Material Properties
Default Stress Strain
Custom Stress Strain
Edit Properties
Plot Stress Strain
OK
Cancel
Help >>
Pile Edit - Pile Data/Batter
H-Pile Properties
Section Dimensions
Depth (d) 9.595 in
Web Thickness (tw) 0.293 in
Width (w) 9.953 in
Flange Thickness (tf) 0.293 in
Section Orientation
Web Along 2 Axis
Web Along 3 Axis
OK
Cancel
Print
Soil Edit - Soil Set 1
Pile 1, Pile Type 2
Soil Set 1 Layer 1 Ø=28 Gamma=120
19.4
Soil Set 1 Layer 2 Cu=1.5e+003 Gamma=115
12.4
Soil Set 1 Layer 3 Cu=5e+002 Gamma=113
5.4
Soil Set 1 Layer 4 Ø=30 Gamma=118
-3.5
Soil Set 1 Layer 5 Ø=30 Gamma=170
-8.5
Global Axes
For Help, press F1
NUM
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8/25/2014

FB-MultiPier File: G:\40725\000\FB-MPier\2014-0611\STR\2014-0701\2014-0701-HAI-Bath-Pier 7-MultiT-UnMod-STR-D2.in

File View Control Wizard Help

Pier #1 | 1 | Load Case

Model Data

- Global Data
  - Problem
  - Analysis
  - AASHTO
  - Pushover
- Pier Data
  - Pile Cap
  - Pile
  - Soil
  - Pier
  - X-Members
  - Load
  - Springs
  - Retained Soil
- Bridge Data
  - Bridge
  - Span Load

Pile

Pile Cross Section Type
- Gross Properties
- Full Cross Section

Edit Cross Section: Edit | Table View

Pile To Cap Connection: Pinned | Fixed

Pile Length Data
- Tip Elevation: -3.5 ft
- Nodes in Free length: 0

Pile Type and Batter Data
- Pile: 1
- Custom
- Pile Type: Type 2
- Soil Set: Set 1

Full Cross-Section Pile Properties

List Pile/Shaft Segments Head to Tip: Custom

Add | Remove

Section Details | Print

Segment Cross Section (tf, tw, H, d, w)

Pile Type Info
- Pile Type: Type 2 | Add | Delete

Database Section Selection
- Use Database Section: Custom
- Customize Current Section | Retrieve Section | Add To Database | Delete Section

Section Type
- Circular
- H-Pile
- Rectangular
- Pipe Pile

Edit Section Contents

Material Properties
- Default Stress Strain
- Custom Stress Strain

Edit Properties | Plot Stress Strain

Section Dimensions
- Width (w): 9.953 in
- Depth (h): 9.595 in
- Unit Weight: 490 pcf
- Length: 22.47 ft

Length: Distance from center of pile cap to tip elevation, or between the nodes of intermediate segments

OK | Cancel | Help >>

Pile Edit - Pile Data/Batter

Default Stress/Strain Curves

Print

Custom

Segment1

Mild Steel	33	Yield Stress, ksi
Prestress	29000	Modulus, ksi
Concrete	6	f'c Compressive, ksi
H Pile	4415	Concrete Modulus, ksi
Shell/Casing	0	Ultimate Prestress, ksi
	0	Prestress Modulus, ksi

Note: Grayed values are not applicable for the defined material and can be activated by editing the section contents.

Set concrete f'c to zero for no concrete.

OK | Cancel

Soil Edit - Soil Set 1

Pile 1, Pile Type 2

Soil Set 1 Layer 1 Ø=28 Gamma=120 (19.4)

Soil Set 1 Layer 2 Cu=1.5e+003 Gamma=115 (12.4)

Soil Set 1 Layer 3 Cu=5e+002 Gamma=113 (5.4)

Soil Set 1 Layer 4 Ø=30 Gamma=118 (-3.5)

Soil Set 1 Layer 5 Ø=30 Gamma=170 (-8.5)

Global Axes

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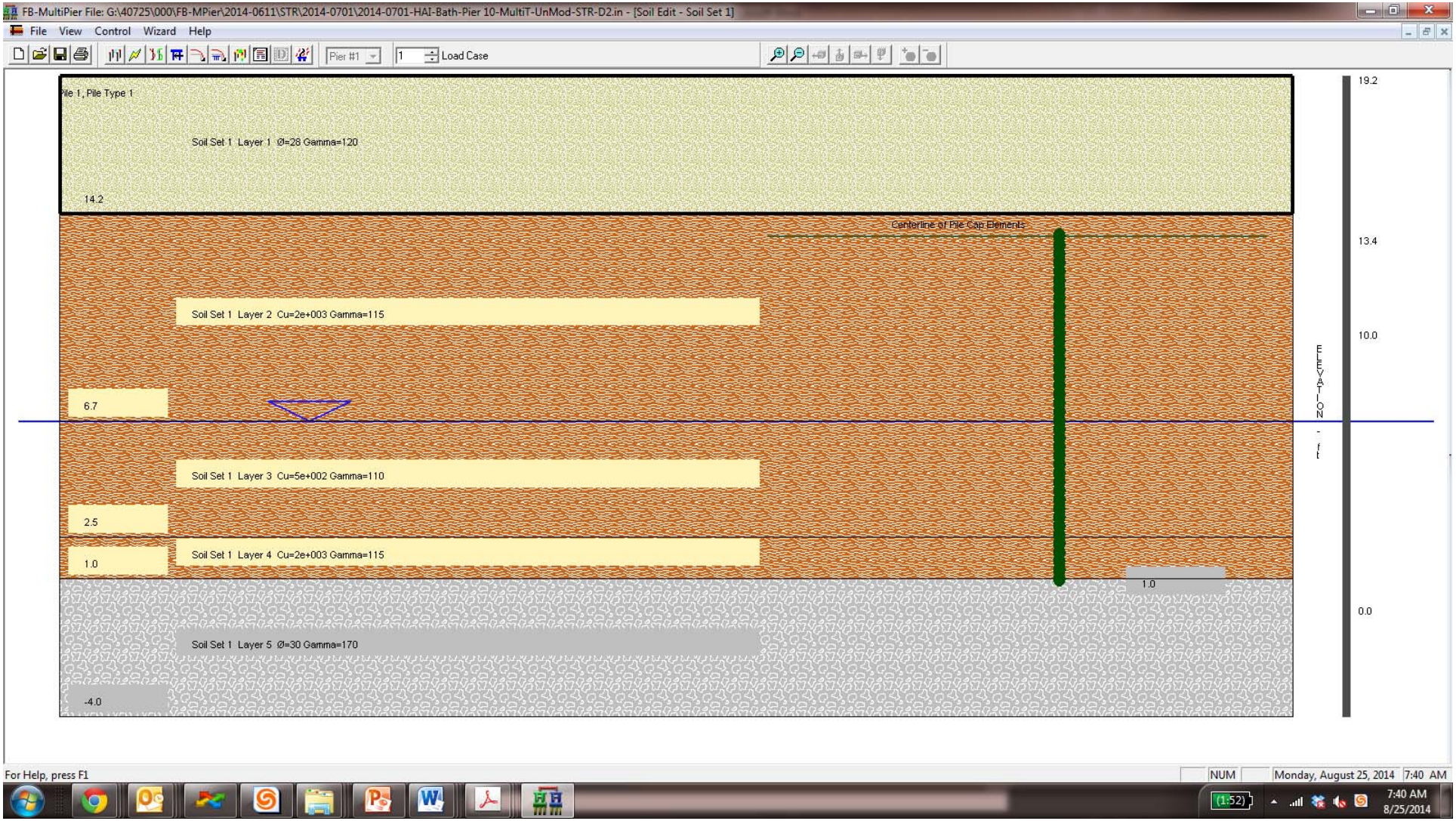
PIER 10 FB-MultiPier Model



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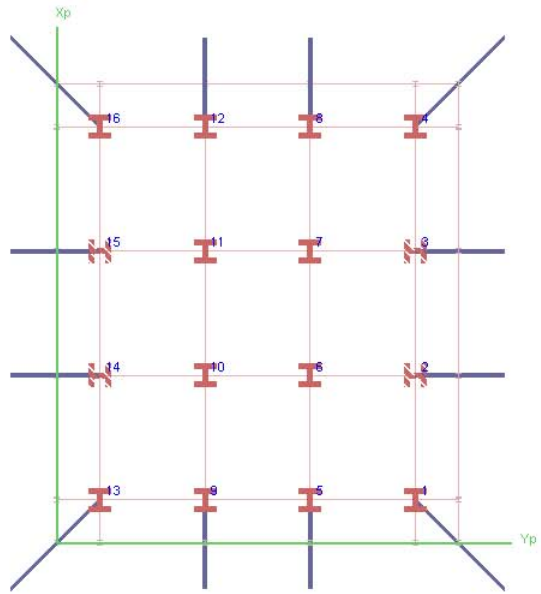
File View Control Wizard Help

Pier #1 | 1 | Load Case



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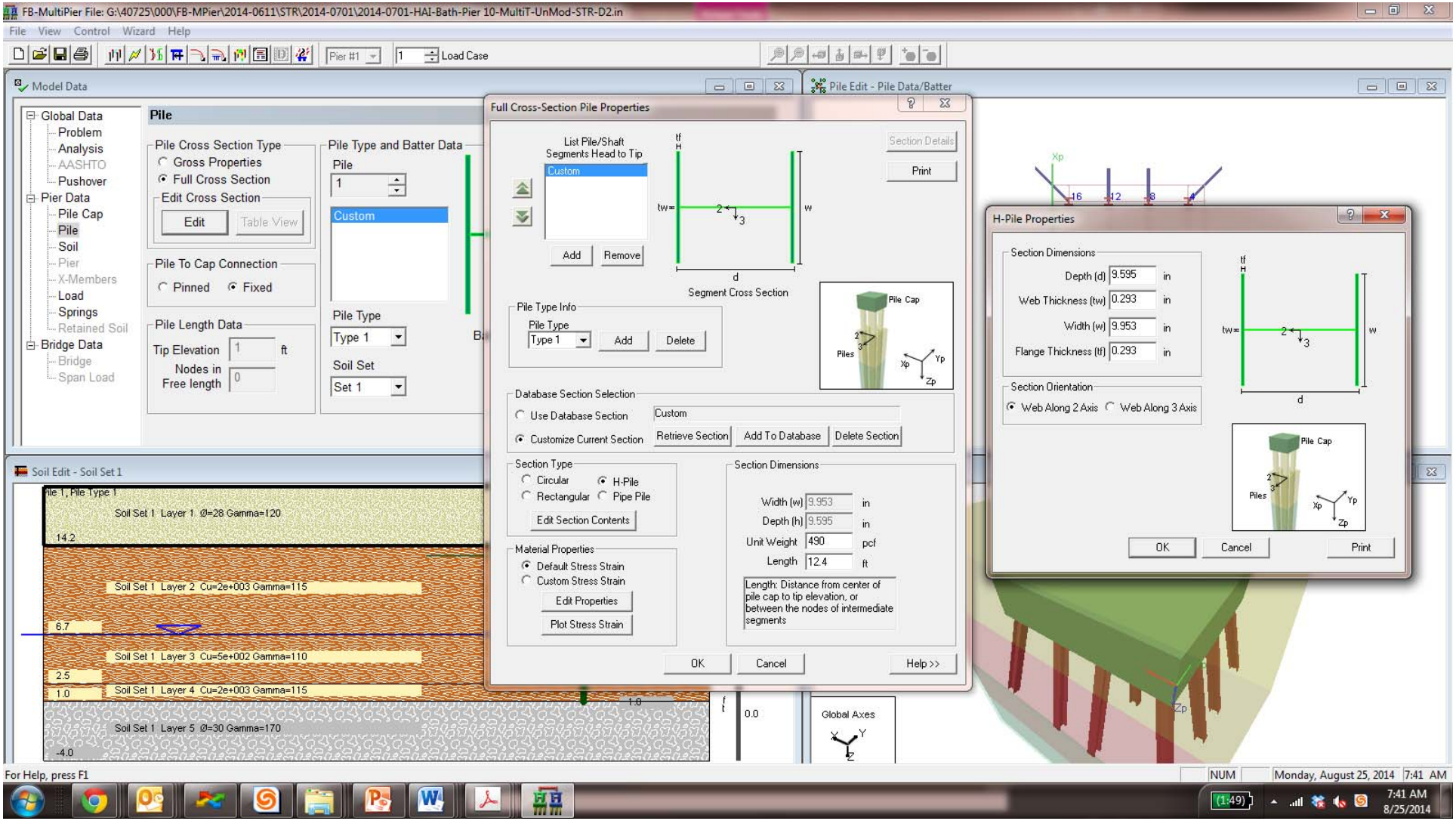
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File View Control Wizard Help
Pier #1
Load Case
Model Data
Global Data
Problem
Analysis
AASHTO
Pushover
Pier Data
Pile Cap
Pile
Soil
Pier
X-Members
Load
Springs
Retained Soil
Bridge Data
Bridge
Span Load
Pile
Pile Cross Section Type
Gross Properties
Full Cross Section
Edit Cross Section
Edit
Table View
Pile To Cap Connection
Pinned
Fixed
Pile Length Data
Tip Elevation
Nodes in Free length
Pile Type and Batter Data
Pile
Custom
Pile Type
Type 1
Soil Set
Set 1
Pile Edit - Pile Data/Batter
Full Cross-Section Pile Properties
List Pile/Shaft Segments Head to Tip
Custom
Add
Remove
Section Details
Print
Segment Cross Section
Pile Type Info
Pile Type
Type 1
Add
Delete
Pile Cap
Piles
Database Section Selection
Use Database Section
Custom
Customize Current Section
Retrieve Section
Add To Database
Delete Section
Section Type
Circular
H-Pile
Rectangular
Pipe Pile
Edit Section Contents
Section Dimensions
Width (w) 9.953 in
Depth (h) 9.595 in
Unit Weight 490 pcf
Length 12.4 ft
Length: Distance from center of pile cap to tip elevation, or between the nodes of intermediate segments
Material Properties
Default Stress Strain
Custom Stress Strain
Edit Properties
Plot Stress Strain
OK
Cancel
Help >>
H-Pile Properties
Section Dimensions
Depth (d) 9.595 in
Web Thickness (tw) 0.293 in
Width (w) 9.953 in
Flange Thickness (tf) 0.293 in
Section Orientation
Web Along 2 Axis
Web Along 3 Axis
OK
Cancel
Print
Soil Edit - Soil Set 1
Pile 1, Pile Type 1
Soil Set 1 Layer 1 Ø=28 Gamma=120
Soil Set 1 Layer 2 Cu=2e+003 Gamma=115
Soil Set 1 Layer 3 Cu=5e+002 Gamma=110
Soil Set 1 Layer 4 Cu=2e+003 Gamma=115
Soil Set 1 Layer 5 Ø=30 Gamma=170
Global Axes
For Help, press F1
NUM
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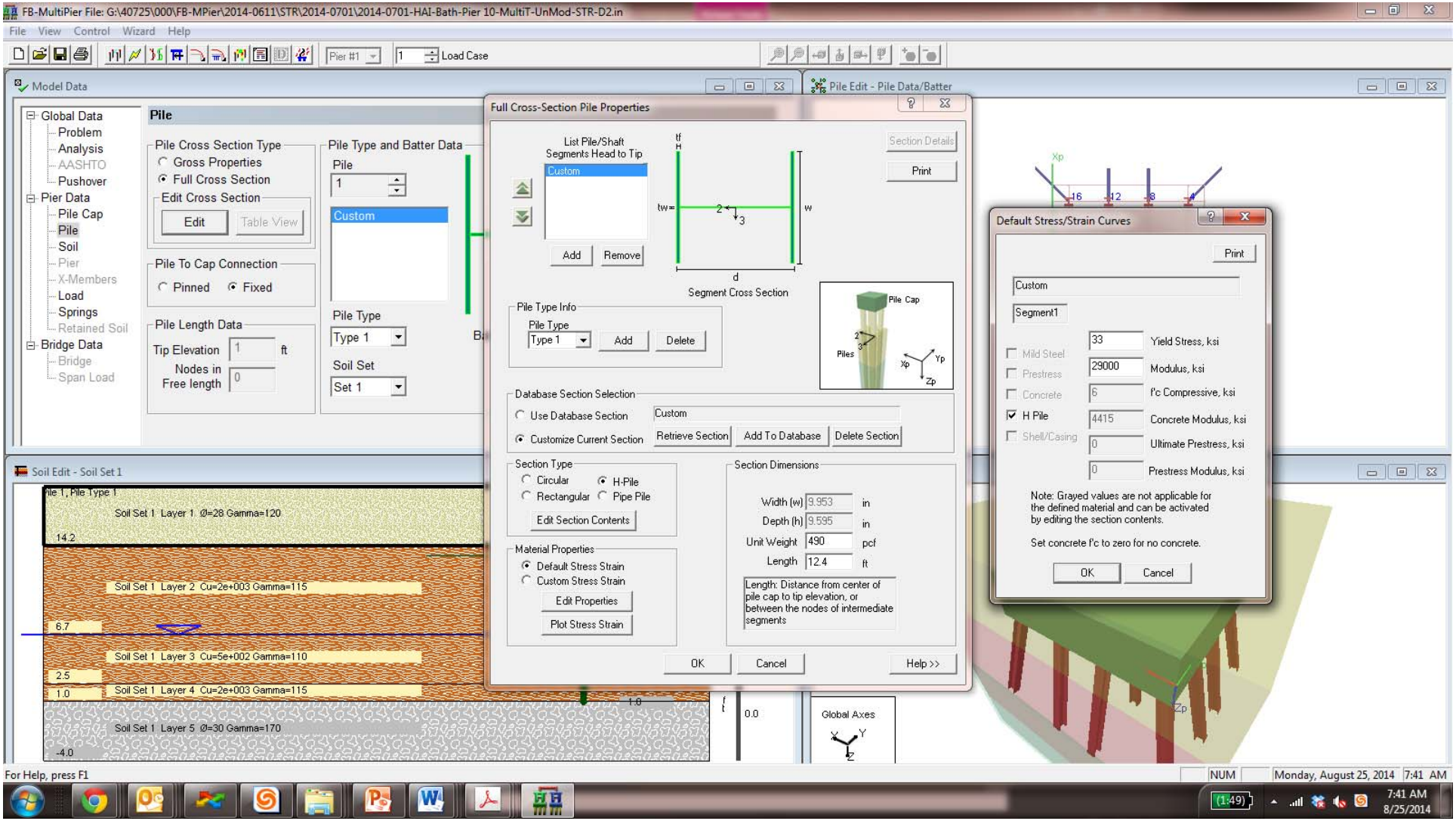
8/25/2014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File View Control Wizard Help
Pier #1
Load Case
Model Data
Global Data
Problem
Analysis
AASHTO
Pushover
Pier Data
Pile Cap
Pile
Soil
Pier
X-Members
Load
Springs
Retained Soil
Bridge Data
Bridge
Span Load
Pile
Pile Cross Section Type
Gross Properties
Full Cross Section
Edit Cross Section
Edit
Table View
Pile To Cap Connection
Pinned
Fixed
Pile Length Data
Tip Elevation
Nodes in Free length
Pile Type and Batter Data
Pile
Custom
Pile Type
Type 1
Soil Set
Set 1
Pile Edit - Pile Data/Batter
Full Cross-Section Pile Properties
List Pile/Shaft Segments Head to Tip
Custom
Add
Remove
Section Details
Print
Segment Cross Section
Pile Type Info
Pile Type
Type 1
Add
Delete
Pile Cap
Piles
Database Section Selection
Use Database Section
Customize Current Section
Custom
Retrieve Section
Add To Database
Delete Section
Section Type
Circular
H-Pile
Rectangular
Pipe Pile
Edit Section Contents
Section Dimensions
Width (w) 9.953 in
Depth (h) 9.595 in
Unit Weight 490 pcf
Length 12.4 ft
Length: Distance from center of pile cap to tip elevation, or between the nodes of intermediate segments
Material Properties
Default Stress Strain
Custom Stress Strain
Edit Properties
Plot Stress Strain
OK
Cancel
Help >>
Default Stress/Strain Curves
Print
Custom
Segment1
Mild Steel
Prestress
Concrete
H Pile
Shell/Casing
33 Yield Stress, ksi
29000 Modulus, ksi
6 f'c Compressive, ksi
4415 Concrete Modulus, ksi
0 Ultimate Prestress, ksi
0 Prestress Modulus, ksi
Note: Grayed values are not applicable for the defined material and can be activated by editing the section contents.
Set concrete f'c to zero for no concrete.
OK
Cancel
Soil Edit - Soil Set 1
Pile 1, Pile Type 1
Soil Set 1 Layer 1 Ø=28 Gamma=120
Soil Set 1 Layer 2 Cu=2e+003 Gamma=115
Soil Set 1 Layer 3 Cu=5e+002 Gamma=110
Soil Set 1 Layer 4 Cu=2e+003 Gamma=115
Soil Set 1 Layer 5 Ø=30 Gamma=170
Global Axes
For Help, press F1
NUM
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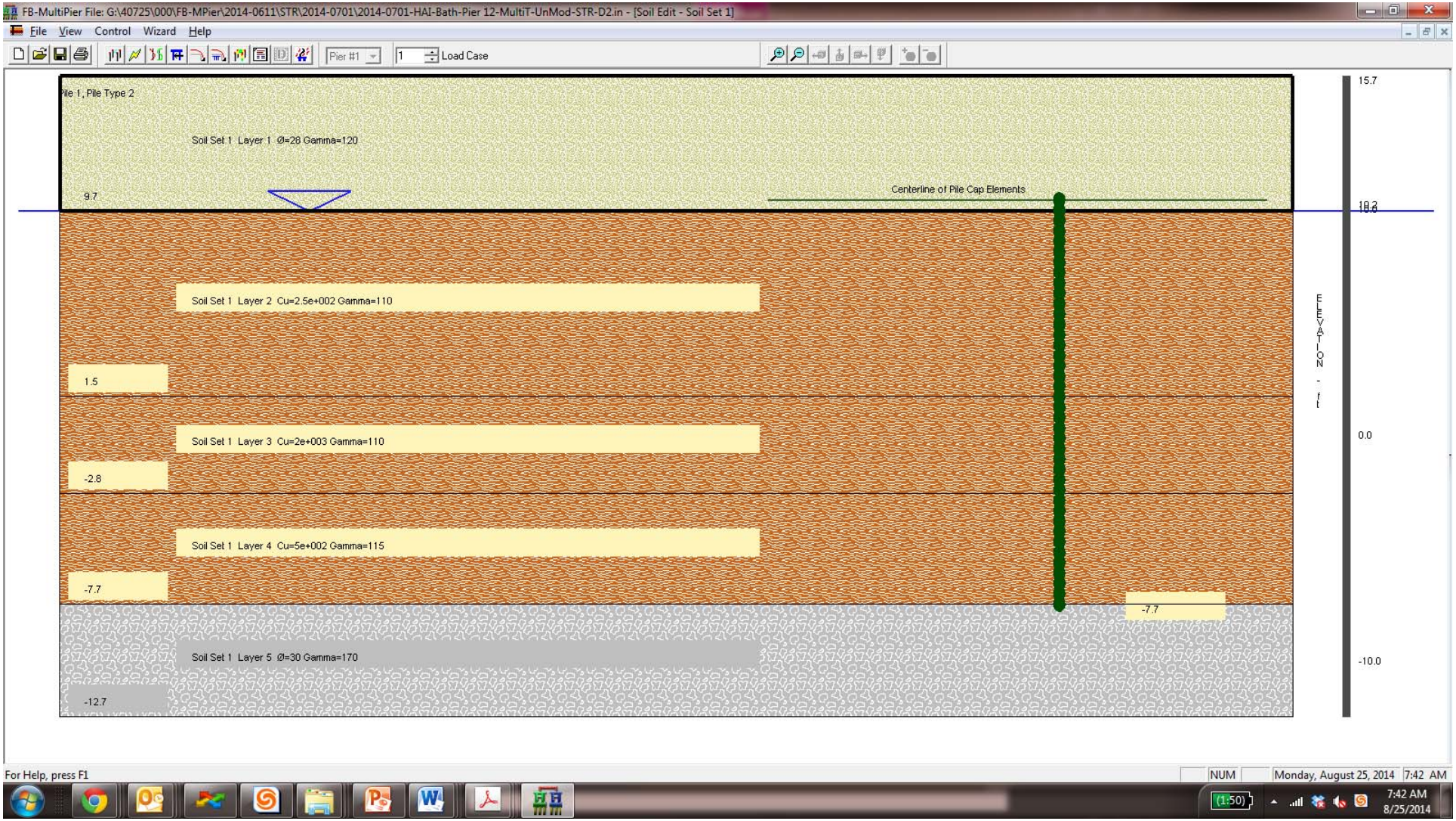
8/25/2014

PIER 12 FB-MultiPier Model



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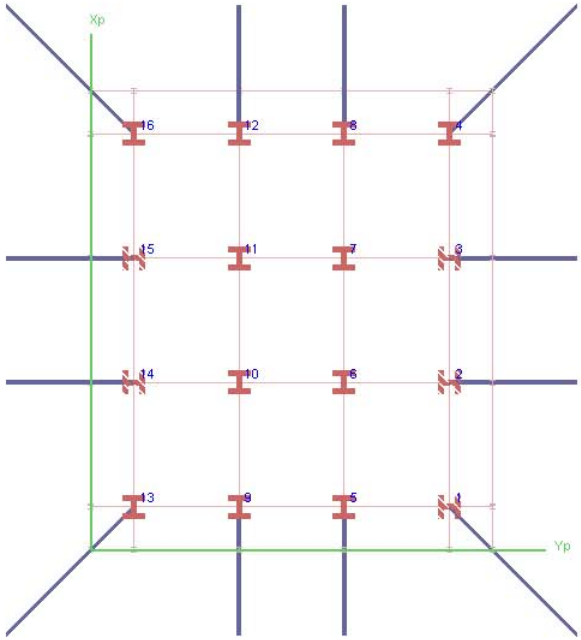
File View Control Wizard Help

Pier #1 | 1 | Load Case



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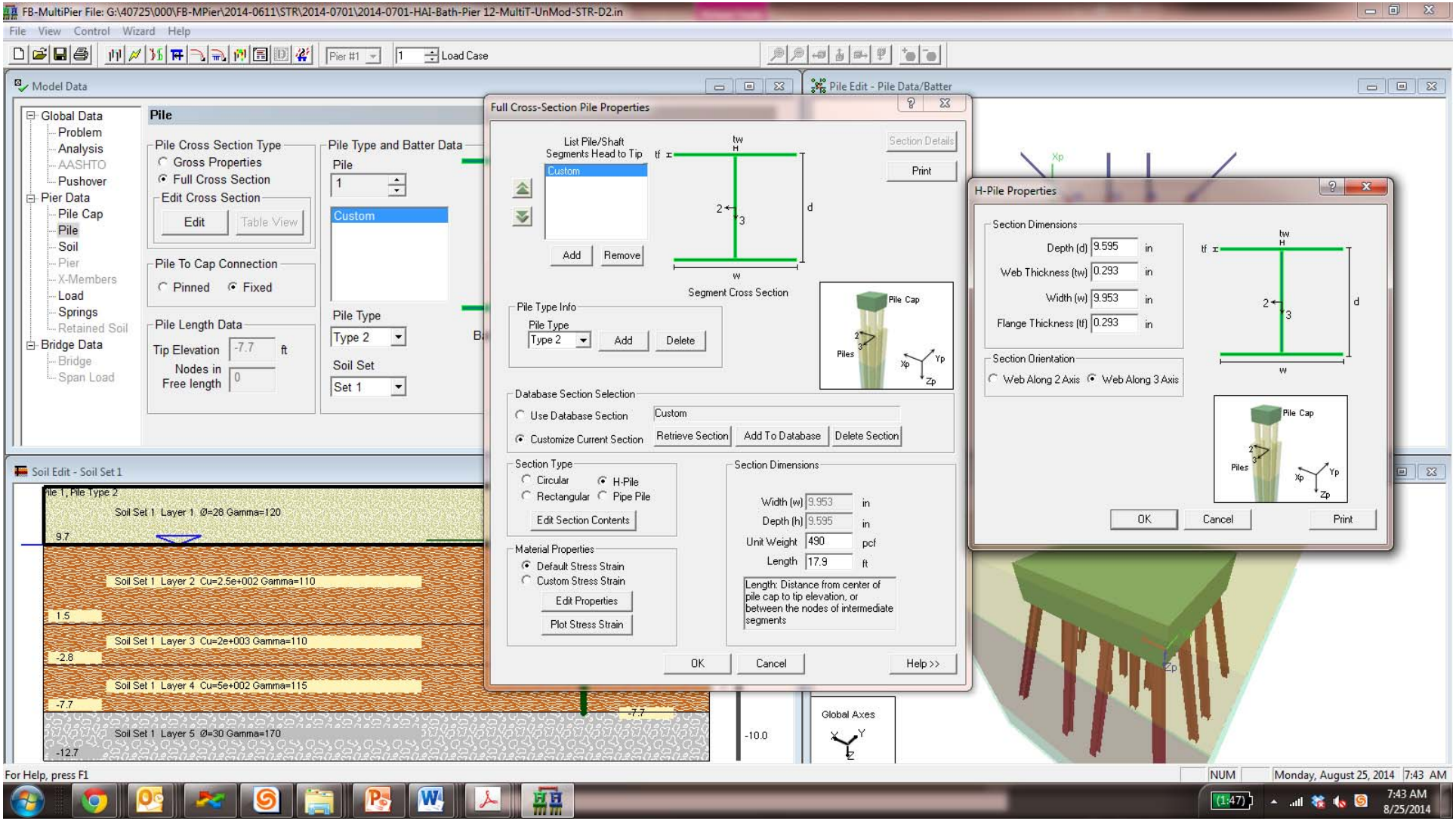
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File View Control Wizard Help
Pier #1
Load Case
Model Data
Global Data
Problem
Analysis
AASHTO
Pushover
Pier Data
Pile Cap
Pile
Soil
Pier
X-Members
Load
Springs
Retained Soil
Bridge Data
Bridge
Span Load
Pile
Pile Cross Section Type
Gross Properties
Full Cross Section
Edit Cross Section
Edit
Table View
Pile To Cap Connection
Pinned
Fixed
Pile Length Data
Tip Elevation
-7.7
ft
Nodes in Free length
0
Pile Type and Batter Data
Pile
1
Custom
Pile Type
Type 2
Soil Set
Set 1
Pile Edit - Pile Data/Batter
Full Cross-Section Pile Properties
List Pile/Shaft Segments Head to Tip
Custom
Add
Remove
Segment Cross Section
Section Details
Print
Pile Type Info
Pile Type
Type 2
Add
Delete
Pile Cap
Piles
Database Section Selection
Use Database Section
Custom
Customize Current Section
Retrieve Section
Add To Database
Delete Section
Section Type
Circular
H-Pile
Rectangular
Pipe Pile
Edit Section Contents
Section Dimensions
Width (w) 9.953 in
Depth (h) 9.595 in
Unit Weight 490 pcf
Length 17.9 ft
Length: Distance from center of pile cap to tip elevation, or between the nodes of intermediate segments
Material Properties
Default Stress Strain
Custom Stress Strain
Edit Properties
Plot Stress Strain
OK
Cancel
Help >>
H-Pile Properties
Section Dimensions
Depth (d) 9.595 in
Web Thickness (tw) 0.293 in
Width (w) 9.953 in
Flange Thickness (tf) 0.293 in
Section Orientation
Web Along 2 Axis
Web Along 3 Axis
Soil Edit - Soil Set 1
Pile 1, Pile Type 2
Soil Set 1 Layer 1 Ø=28 Gamma=120
9.7
Soil Set 1 Layer 2 Cu=2.5e+002 Gamma=110
1.5
Soil Set 1 Layer 3 Cu=2e+003 Gamma=110
-2.8
Soil Set 1 Layer 4 Cu=5e+002 Gamma=115
-7.7
Soil Set 1 Layer 5 Ø=30 Gamma=170
-12.7
-10.0
Global Axes
For Help, press F1
NUM
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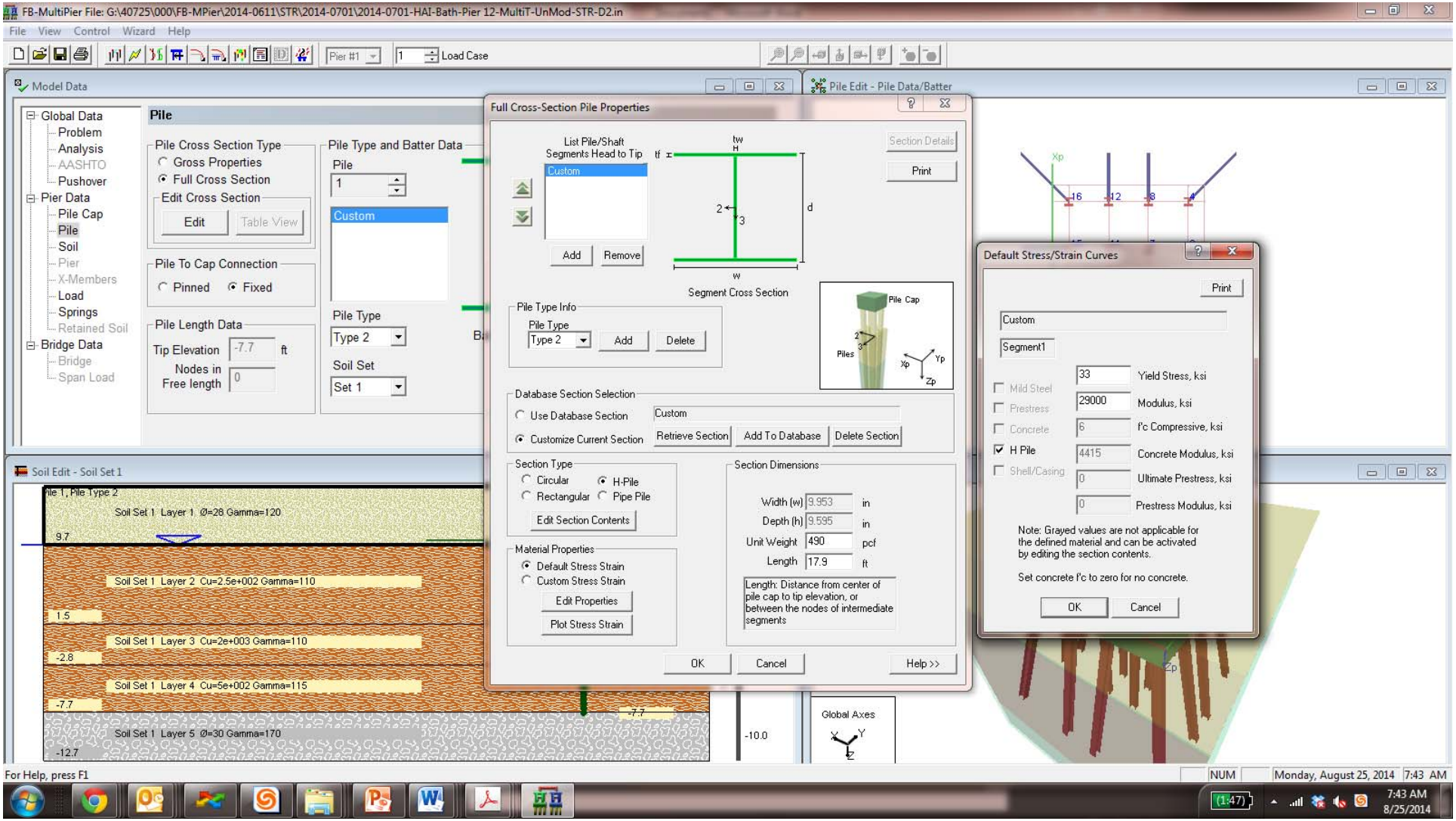
7:43 AM
8/25/2014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File View Control Wizard Help
Pier #1
Load Case
Model Data
Global Data
Problem
Analysis
AASHTO
Pushover
Pier Data
Pile Cap
Pile
Soil
Pier
X-Members
Load
Springs
Retained Soil
Bridge Data
Bridge
Span Load
Pile
Pile Cross Section Type
Gross Properties
Full Cross Section
Edit Cross Section
Edit
Table View
Pile To Cap Connection
Pinned
Fixed
Pile Length Data
Tip Elevation
-7.7
ft
Nodes in Free length
0
Pile Type and Batter Data
Pile
1
Custom
Pile Type
Type 2
Soil Set
Set 1
Full Cross-Section Pile Properties
List Pile/Shaft Segments Head to Tip
Custom
Add
Remove
Section Details
Print
Segment Cross Section
Pile Type Info
Pile Type
Type 2
Add
Delete
Pile Cap
Piles
Database Section Selection
Use Database Section
Custom
Customize Current Section
Retrieve Section
Add To Database
Delete Section
Section Type
Circular
H-Pile
Rectangular
Pipe Pile
Edit Section Contents
Section Dimensions
Width (w) 9.953 in
Depth (h) 9.595 in
Unit Weight 490 pcf
Length 17.9 ft
Length: Distance from center of pile cap to tip elevation, or between the nodes of intermediate segments
Material Properties
Default Stress Strain
Custom Stress Strain
Edit Properties
Plot Stress Strain
OK
Cancel
Help >>
Default Stress/Strain Curves
Print
Custom
Segment1
Mild Steel
Prestress
Concrete
H Pile
Shell/Casing
33 Yield Stress, ksi
29000 Modulus, ksi
6 f'c Compressive, ksi
4415 Concrete Modulus, ksi
0 Ultimate Prestress, ksi
0 Prestress Modulus, ksi
Note: Grayed values are not applicable for the defined material and can be activated by editing the section contents.
Set concrete f'c to zero for no concrete.
OK
Cancel
Soil Edit - Soil Set 1
Pile 1, Pile Type 2
Soil Set 1 Layer 1 Ø=28 Gamma=120
9.7
Soil Set 1 Layer 2 Cu=2.5e+002 Gamma=110
1.5
Soil Set 1 Layer 3 Cu=2e+003 Gamma=110
-2.8
Soil Set 1 Layer 4 Cu=5e+002 Gamma=115
-7.7
Soil Set 1 Layer 5 Ø=30 Gamma=170
-12.7
-10.0
Global Axes
Pile Edit - Pile Data/Batter
For Help, press F1
NUM
Monday, August 25, 2014 7:43 AM
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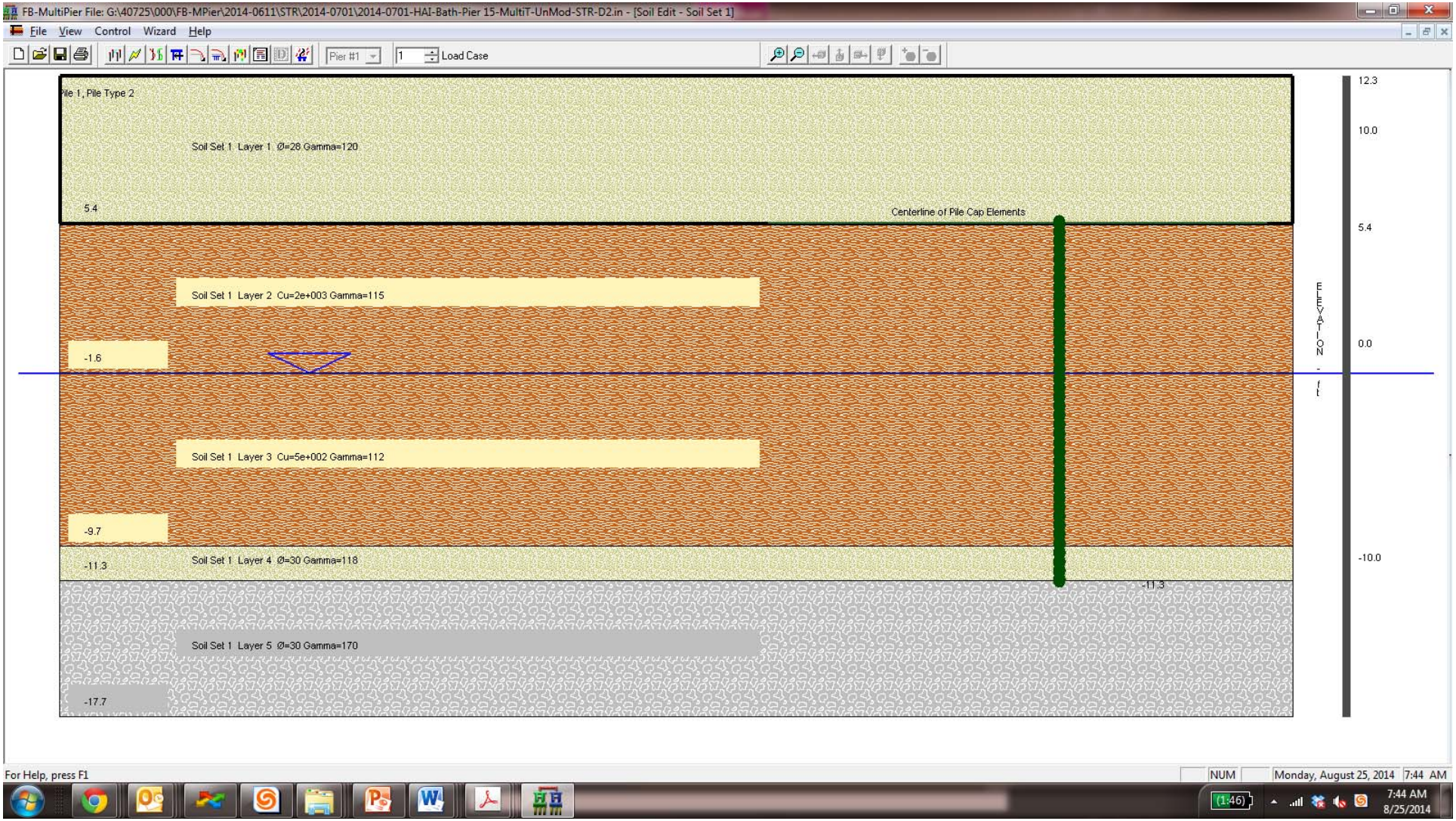
8/25/2014

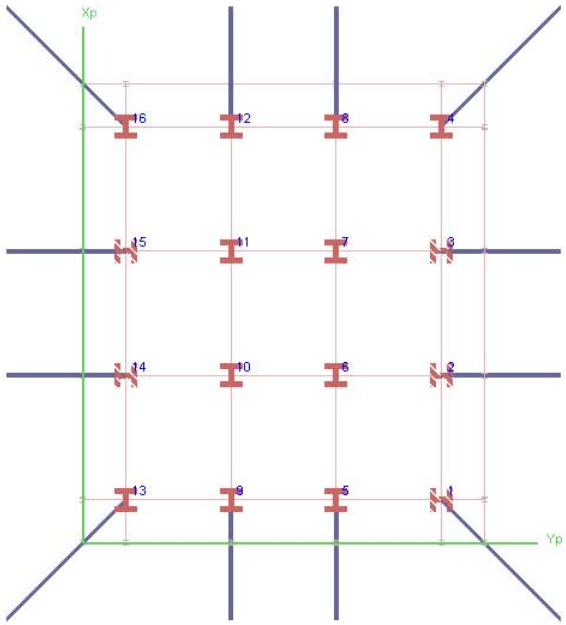
PIER 15 FB-MultiPier Model



FB-MultiPier File: G:\40725\000\FB-MPier\2014-0611\STR\2014-0701\2014-0701-HAI-Bath-Pier 15-MultiT-UnMod-STR-D2.in - [Pile Edit - Pile Data/Batter]

File View Control Wizard Help

Pier #1 | 1 | Load Case



For Help, press F1 | NUM | Monday, August 25, 2014 7:44 AM

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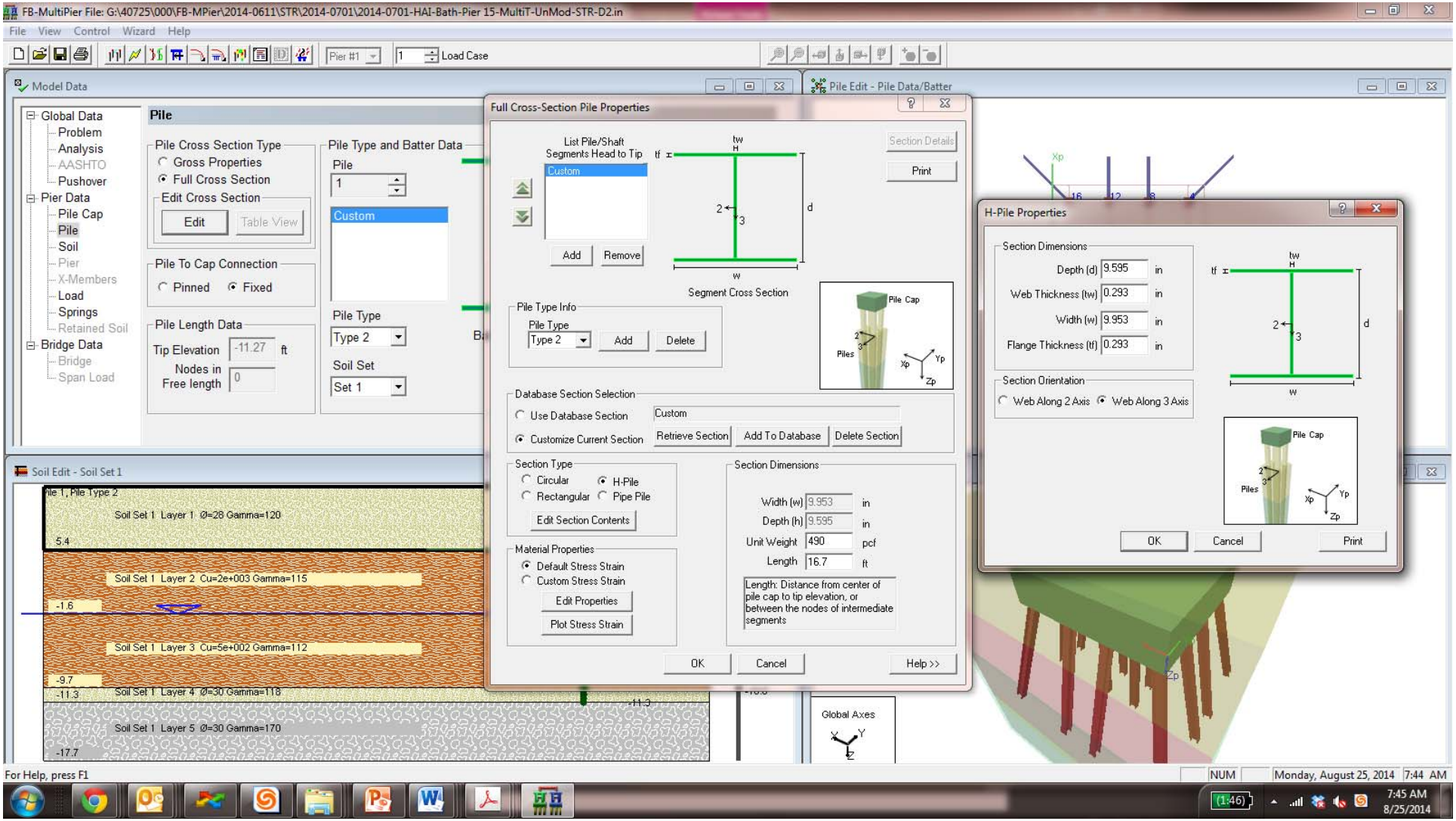
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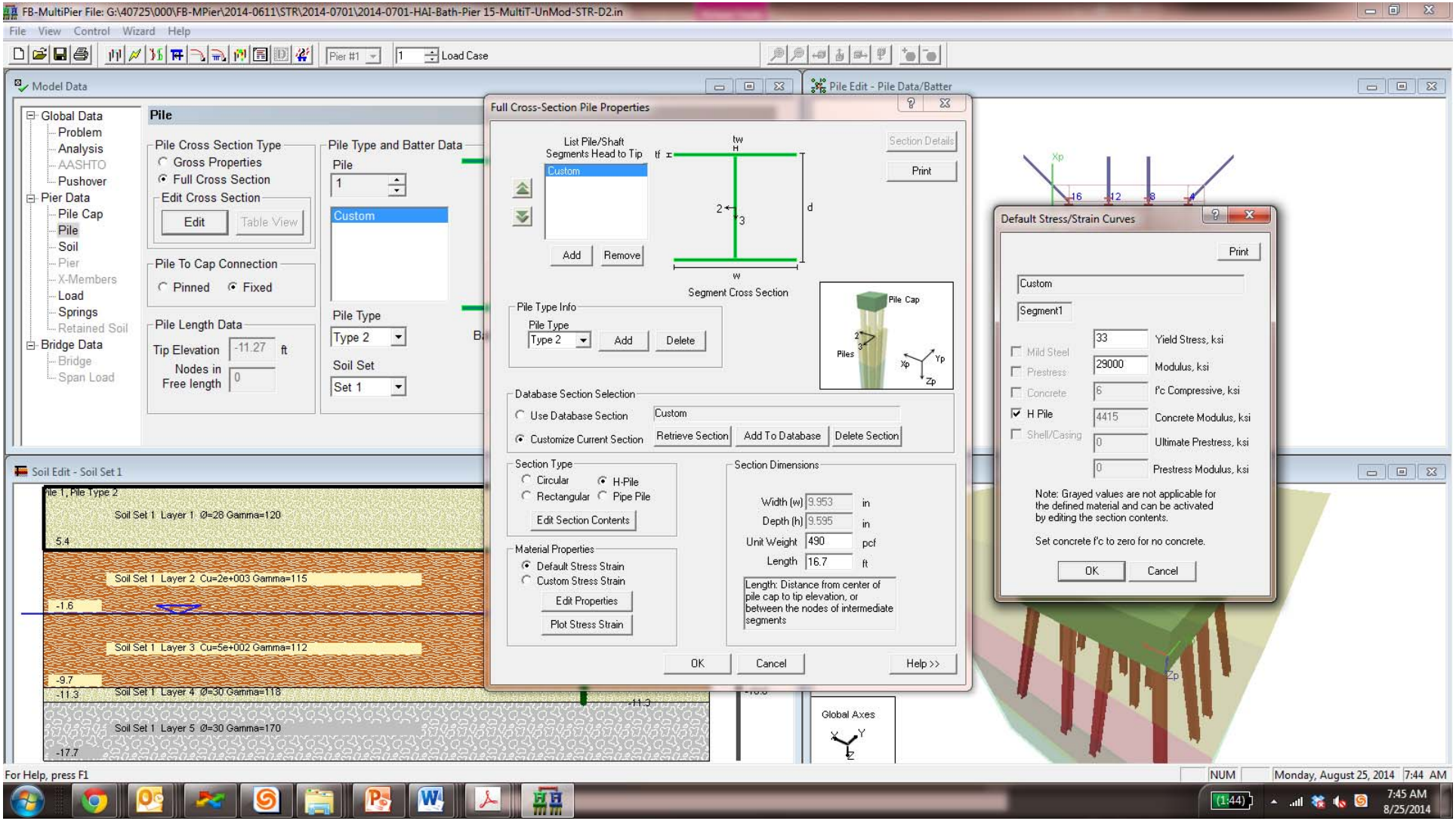
File View Control Wizard Help
Pier #1
Load Case
Model Data
Global Data
Problem
Analysis
AASHTO
Pushover
Pier Data
Pile Cap
Pile
Soil
Pier
X-Members
Load
Springs
Retained Soil
Bridge Data
Bridge
Span Load
Pile
Pile Cross Section Type
Gross Properties
Full Cross Section
Edit Cross Section
Edit
Table View
Pile To Cap Connection
Pinned
Fixed
Pile Length Data
Tip Elevation
-11.27
ft
Nodes in Free length
0
Pile Type and Batter Data
Pile
1
Custom
Pile Type
Type 2
Soil Set
Set 1
Full Cross-Section Pile Properties
List Pile/Shaft Segments Head to Tip
Custom
Add
Remove
Segment Cross Section
Section Details
Print
Pile Type Info
Pile Type
Type 2
Add
Delete
Pile Cap
Piles
Database Section Selection
Use Database Section
Custom
Customize Current Section
Retrieve Section
Add To Database
Delete Section
Section Type
Circular
H-Pile
Rectangular
Pipe Pile
Edit Section Contents
Section Dimensions
Width (w) 9.953 in
Depth (h) 9.595 in
Unit Weight 490 pcf
Length 16.7 ft
Length: Distance from center of pile cap to tip elevation, or between the nodes of intermediate segments
Material Properties
Default Stress Strain
Custom Stress Strain
Edit Properties
Plot Stress Strain
OK
Cancel
Help >>
H-Pile Properties
Section Dimensions
Depth (d) 9.595 in
Web Thickness (tw) 0.293 in
Width (w) 9.953 in
Flange Thickness (tf) 0.293 in
Section Orientation
Web Along 2 Axis
Web Along 3 Axis
OK
Cancel
Print
Pile Edit - Pile Data/Batter
Soil Edit - Soil Set 1
Pile 1, Pile Type 2
Soil Set 1 Layer 1 Ø=28 Gamma=120
Soil Set 1 Layer 2 Cu=2e+003 Gamma=115
Soil Set 1 Layer 3 Cu=5e+002 Gamma=112
Soil Set 1 Layer 4 Ø=30 Gamma=118
Soil Set 1 Layer 5 Ø=30 Gamma=170
5.4
-1.6
-9.7
-11.3
-17.7
Global Axes
For Help, press F1
NUM
Monday, August 25, 2014 7:44 AM
7:45 AM
8/25/2014


FB-MultiPier File: G:\40725\000\FB-MPier\2014-0611\STR\2014-0701\2014-0701-HAI-Bath-Pier 15-MultiT-UnMod-STR-D2.in
File View Control Wizard Help
Pier #1
Load Case
Model Data
Global Data
Problem
Analysis
AASHTO
Pushover
Pier Data
Pile Cap
Pile
Soil
Pier
X-Members
Load
Springs
Retained Soil
Bridge Data
Bridge
Span Load
Pile
Pile Cross Section Type
Gross Properties
Full Cross Section
Edit Cross Section
Edit
Table View
Pile To Cap Connection
Pinned
Fixed
Pile Length Data
Tip Elevation
-11.27
ft
Nodes in Free length
0
Pile Type and Batter Data
Pile
1
Custom
Pile Type
Type 2
Soil Set
Set 1
Full Cross-Section Pile Properties
List Pile/Shaft Segments Head to Tip
Custom
Add
Remove
Section Details
Print
Segment Cross Section
Pile Type Info
Pile Type
Type 2
Add
Delete
Pile Cap
Piles
Database Section Selection
Use Database Section
Custom
Customize Current Section
Retrieve Section
Add To Database
Delete Section
Section Type
Circular
H-Pile
Rectangular
Pipe Pile
Edit Section Contents
Section Dimensions
Width (w)
9.953
in
Depth (h)
9.595
in
Unit Weight
490
pcf
Length
16.7
ft
Length: Distance from center of pile cap to tip elevation, or between the nodes of intermediate segments
Material Properties
Default Stress Strain
Custom Stress Strain
Edit Properties
Plot Stress Strain
OK
Cancel
Help >>
Pile Edit - Pile Data/Batter
Default Stress/Strain Curves
Print
Custom
Segment1
Mild Steel
Prestress
Concrete
H Pile
Shell/Casing
33
Yield Stress, ksi
29000
Modulus, ksi
6
f'c Compressive, ksi
4415
Concrete Modulus, ksi
0
Ultimate Prestress, ksi
0
Prestress Modulus, ksi
Note: Grayed values are not applicable for the defined material and can be activated by editing the section contents.
Set concrete f'c to zero for no concrete.
OK
Cancel
Soil Edit - Soil Set 1
Pile 1, Pile Type 2
Soil Set 1 Layer 1 Ø=28 Gamma=120
5.4
Soil Set 1 Layer 2 Cu=2e+003 Gamma=115
-1.6
Soil Set 1 Layer 3 Cu=5e+002 Gamma=112
-9.7
-11.3
Soil Set 1 Layer 4 Ø=30 Gamma=118
Soil Set 1 Layer 5 Ø=30 Gamma=170
-17.7
Global Axes
For Help, press F1
NUM
Monday, August 25, 2014
7:44 AM
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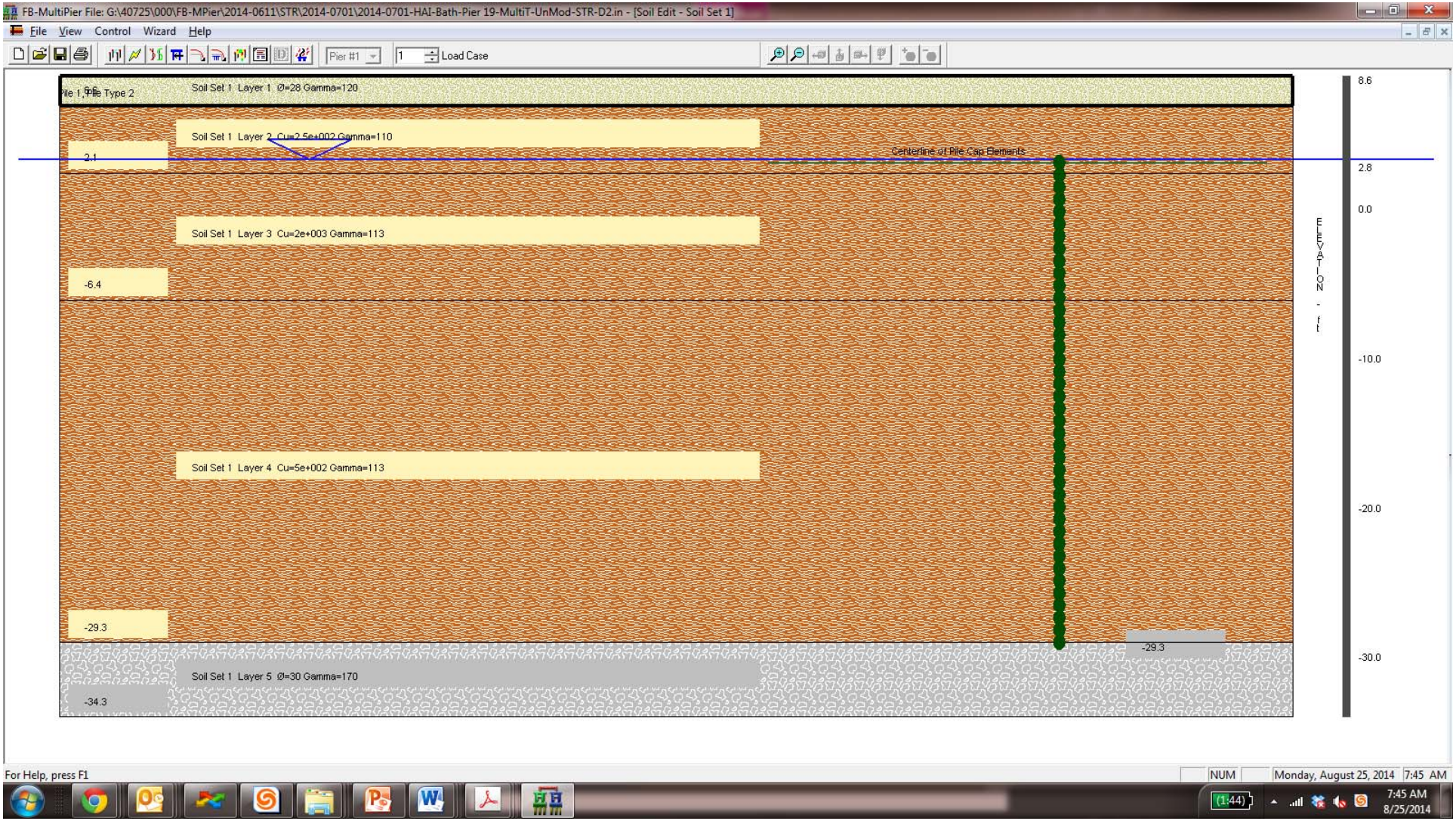
8/25/2014

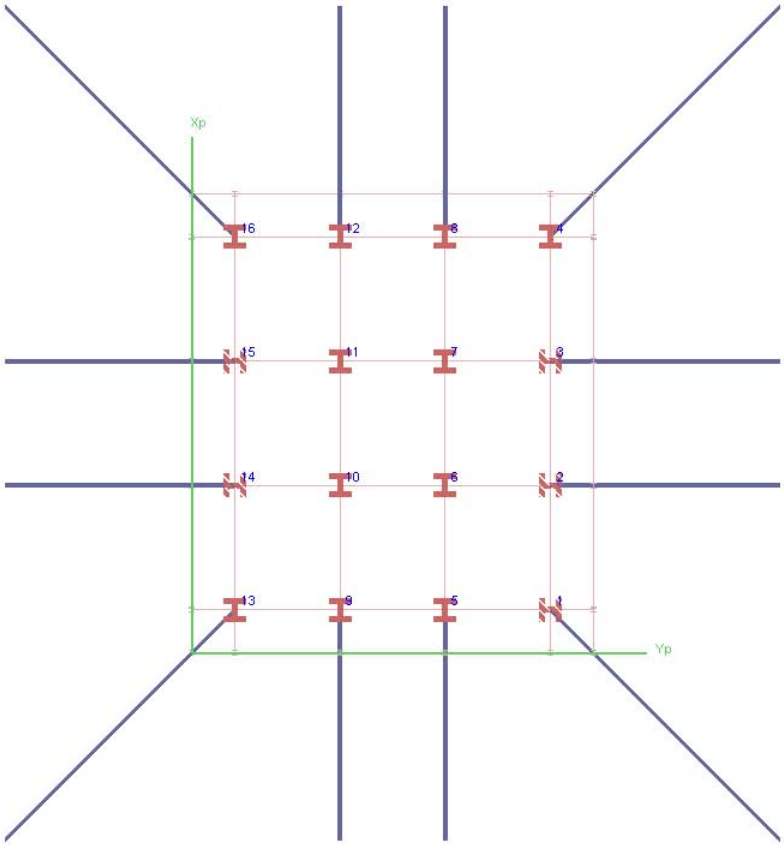
PIER 19 FB-MultiPier Model



FB-MultiPier File: G:\40725\000\FB-MPier\2014-0611\STR\2014-0701\2014-0701-HAI-Bath-Pier 19-MultiT-UnMod-STR-D2.in - [Pile Edit - Pile Data/Batter]

File View Control Wizard Help

Pier #1 | 1 | Load Case



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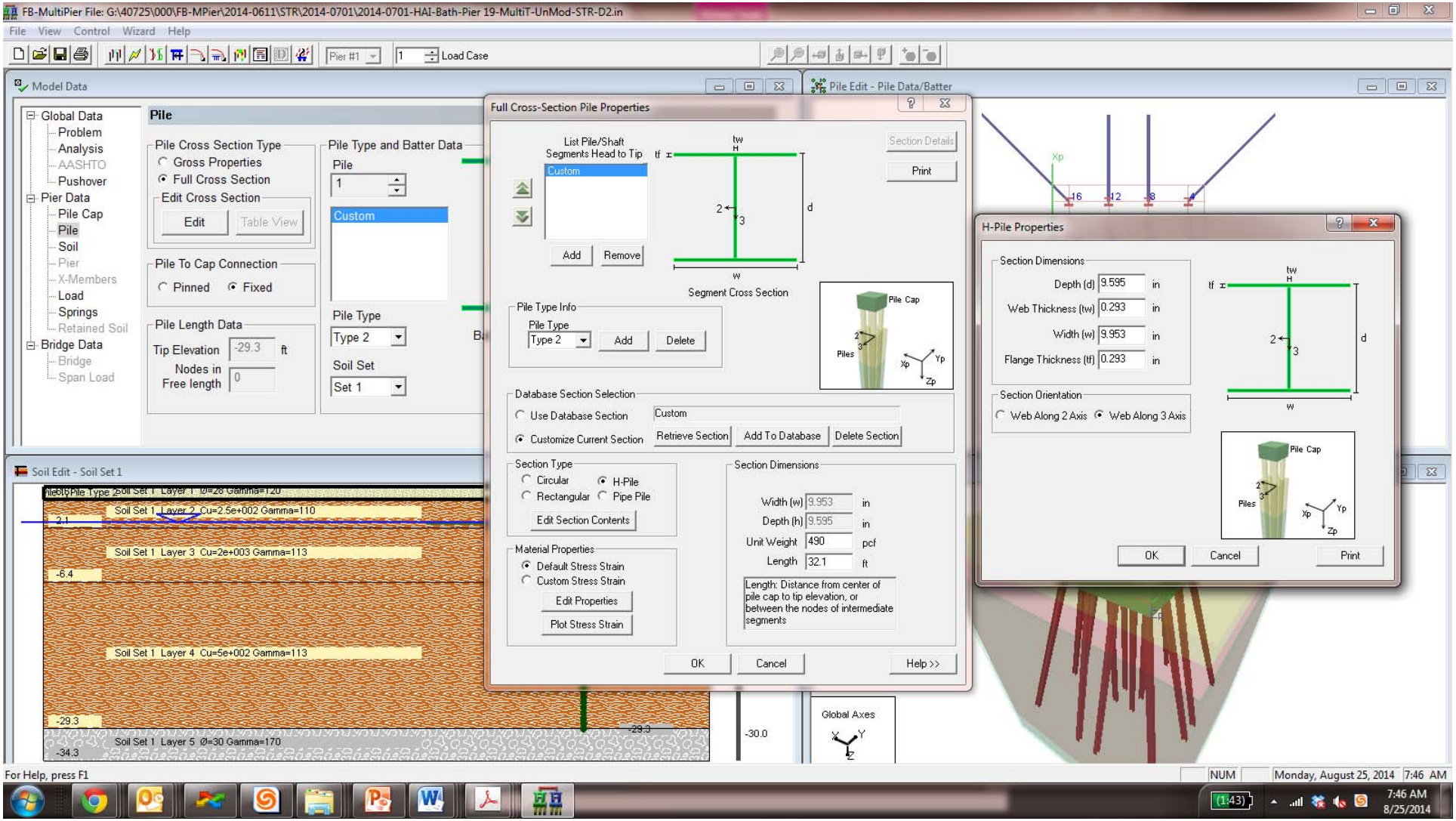
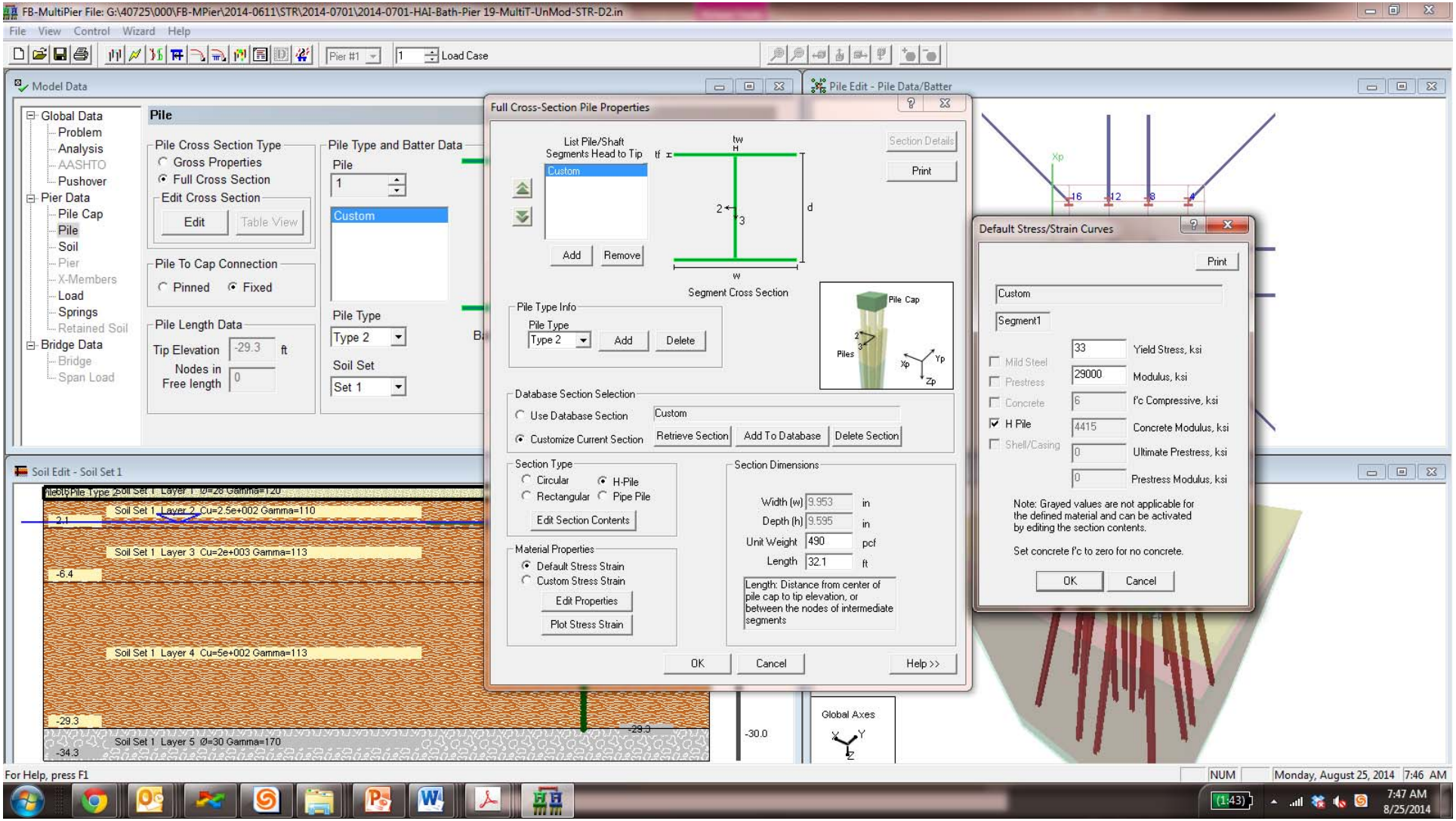

FB-MultiPier File: G:\40725\000\FB-MPier\2014-0611\STR\2014-0701\2014-0701-HAI-Bath-Pier 19-MultiT-UnMod-STR-D2.in
File View Control Wizard Help
Pier #1
Load Case
Model Data
Global Data
Problem
Analysis
AASHTO
Pushover
Pier Data
Pile Cap
Pile
Soil
Pier
X-Members
Load
Springs
Retained Soil
Bridge Data
Bridge
Span Load
Pile
Pile Cross Section Type
Gross Properties
Full Cross Section
Edit Cross Section
Edit
Table View
Pile To Cap Connection
Pinned
Fixed
Pile Length Data
Tip Elevation
-29.3
ft
Nodes in Free length
0
Pile Type and Batter Data
Pile
1
Custom
Pile Type
Type 2
Soil Set
Set 1
Full Cross-Section Pile Properties
List Pile/Shaft Segments Head to Tip
Custom
Add
Remove
Segment Cross Section
Section Details
Print
Pile Type Info
Pile Type
Type 2
Add
Delete
Pile Cap
Piles
Database Section Selection
Use Database Section
Custom
Customize Current Section
Retrieve Section
Add To Database
Delete Section
Section Type
Circular
H-Pile
Rectangular
Pipe Pile
Edit Section Contents
Section Dimensions
Width (w) 9.953 in
Depth (h) 9.595 in
Unit Weight 490 pcf
Length 32.1 ft
Length: Distance from center of pile cap to tip elevation, or between the nodes of intermediate segments
Material Properties
Default Stress Strain
Custom Stress Strain
Edit Properties
Plot Stress Strain
OK
Cancel
Help >>
Pile Edit - Pile Data/Batter
H-Pile Properties
Section Dimensions
Depth (d) 9.595 in
Web Thickness (tw) 0.293 in
Width (w) 9.953 in
Flange Thickness (tf) 0.293 in
Section Orientation
Web Along 2 Axis
Web Along 3 Axis
OK
Cancel
Print
Soil Edit - Soil Set 1
Soil Set 1 Layer 2 Cu=2.5e+002 Gamma=110
Soil Set 1 Layer 3 Cu=2e+003 Gamma=113
Soil Set 1 Layer 4 Cu=5e+002 Gamma=113
Soil Set 1 Layer 5 Ø=30 Gamma=170
-2.1
-6.4
-29.3
-34.3
-30.0
Global Axes
For Help, press F1
NUM
Monday, August 25, 2014 7:46 AM
7:46 AM
8/25/2014


FB-MultiPier File: G:\40725\000\FB-MPier\2014-0611\STR\2014-0701\2014-0701-HAI-Bath-Pier 19-MultiT-UnMod-STR-D2.in
File View Control Wizard Help
Pier #1
Load Case
Model Data
Global Data
Problem
Analysis
AASHTO
Pushover
Pier Data
Pile Cap
Pile
Soil
Pier
X-Members
Load
Springs
Retained Soil
Bridge Data
Bridge
Span Load
Pile
Pile Cross Section Type
Gross Properties
Full Cross Section
Edit Cross Section
Edit
Table View
Pile To Cap Connection
Pinned
Fixed
Pile Length Data
Tip Elevation
-29.3
ft
Nodes in Free length
0
Pile Type and Batter Data
Pile
1
Custom
Pile Type
Type 2
Soil Set
Set 1
Full Cross-Section Pile Properties
List Pile/Shaft Segments Head to Tip
Custom
Add
Remove
Section Details
Print
Segment Cross Section
Pile Type Info
Pile Type
Type 2
Add
Delete
Pile Cap
Piles
Database Section Selection
Use Database Section
Custom
Customize Current Section
Retrieve Section
Add To Database
Delete Section
Section Type
Circular
H-Pile
Rectangular
Pipe Pile
Edit Section Contents
Material Properties
Default Stress Strain
Custom Stress Strain
Edit Properties
Plot Stress Strain
Section Dimensions
Width (w) 9.953 in
Depth (h) 9.595 in
Unit Weight 490 pcf
Length 32.1 ft
Length: Distance from center of pile cap to tip elevation, or between the nodes of intermediate segments
OK
Cancel
Help >>
Pile Edit - Pile Data/Batter
Default Stress/Strain Curves
Print
Custom
Segment1
Mild Steel
Prestress
Concrete
H Pile
Shell/Casing
33 Yield Stress, ksi
29000 Modulus, ksi
6 f'c Compressive, ksi
4415 Concrete Modulus, ksi
0 Ultimate Prestress, ksi
0 Prestress Modulus, ksi
Note: Grayed values are not applicable for the defined material and can be activated by editing the section contents.
Set concrete f'c to zero for no concrete.
OK
Cancel
Soil Edit - Soil Set 1
Soil Set 1 Layer 2 Cu=2.5e+002 Gamma=110
Soil Set 1 Layer 3 Cu=2e+003 Gamma=113
Soil Set 1 Layer 4 Cu=5e+002 Gamma=113
Soil Set 1 Layer 5 Ø=30 Gamma=170
-2.1
-6.4
-29.3
-34.3
-30.0
Global Axes
For Help, press F1
NUM
Monday, August 25, 2014 7:46 AM
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8/25/2014

ELASTIC PILE COMPRESSION CALCULATION
ROUTE 1 VIADUCT BRIDGE REHABILITATION
MaineDOT WIN 19273.00
BATH, MAINE

Full Cross-Sectional Area, A_{full} =	12.35	square inches (0 in. corrosion loss)
Corroded Cross-Sectional Area, A_{cor} =	8.47	square inches (1/16-in. corrosion loss)
Modulus of Elasticity of Steel, E =	29,000	kips per square inch
Pile AE_{full} =	358,150	kips
Pile AE_{cor} =	245,630	kips

Substructure Location	Superstructure Type[1]	Load Combination[1]	SER I Limit State Vertical Load[1] (kips)	Bottom of Pile Cap Elevation[2]	As-Built Pile Tip Elevations	Pile Length[3] (ft)	Calculated δ_v (in.)		Avg. Calculated δ_v (in.)	
							Full Cross-Section	Corroded Cross-Section	Full Cross-Section	Corroded Cross-Section
Pier 5	Multi-T	LC1	84	23.1	14.8	8.3	0.02	0.03	0.03	0.04
					10.2	12.9	0.04	0.05		
		LC2	91		14.8	8.3	0.03	0.04	0.03	0.05
					10.2	12.9	0.04	0.06		
	NEXT	LC1	90		14.8	8.3	0.02	0.04	0.03	0.05
					10.2	12.9	0.04	0.06		
		LC2	97		14.8	8.3	0.03	0.04	0.03	0.05
					10.2	12.9	0.04	0.06		
Pier 6	Multi-T	LC1	65	20.7	12.4	8.3	0.02	0.03	0.04	0.06
					-6.3	27.0	0.06	0.09		
		LC2	70		12.4	8.3	0.02	0.03	0.04	0.06
					-6.3	27.0	0.06	0.09		
	NEXT	LC1	69		12.4	8.3	0.02	0.03	0.04	0.06
					-6.3	27.0	0.06	0.09		
		LC2	75		12.4	8.3	0.02	0.03	0.04	0.06
					-6.3	27.0	0.07	0.10		
Pier 7	Multi-T	LC1	65	18.8	4.1	14.7	0.03	0.05	0.06	0.08
					-17.4	36.2	0.08	0.12		
		LC2	71		4.1	14.7	0.04	0.05	0.06	0.09
					-17.4	36.2	0.09	0.13		
	NEXT	LC1	70		4.1	14.7	0.03	0.05	0.06	0.09
					-17.4	36.2	0.08	0.12		
		LC2	76		4.1	14.7	0.04	0.05	0.06	0.09
					-17.4	36.2	0.09	0.13		
Pier 10	Multi-T	LC1	66	13.2	4.4	8.8	0.02	0.03	0.03	0.04
					-4.5	17.7	0.04	0.06		
		LC2	71		4.4	8.8	0.02	0.03	0.03	0.05
					-4.5	17.7	0.04	0.06		
	NEXT	LC1	70		4.4	8.8	0.02	0.03	0.03	0.05
					-4.5	17.7	0.04	0.06		
		LC2	76		4.4	8.8	0.02	0.03	0.03	0.05
					-4.5	17.7	0.05	0.07		
Pier 11	Multi-T	LC1	132	11.7	7.4	4.3	0.02	0.03	0.03	0.04
					2.7	9.0	0.04	0.06		
		LC2	143		7.4	4.3	0.02	0.03	0.03	0.05
					2.7	9.0	0.04	0.06		
	NEXT	LC1	141		7.4	4.3	0.02	0.03	0.03	0.05
					2.7	9.0	0.04	0.06		
		LC2	152		7.4	4.3	0.02	0.03	0.03	0.05
					2.7	9.0	0.05	0.07		
Pier 12	Multi-T	LC1	59	10	-4.0	14.0	0.03	0.04	0.04	0.05
					-11.5	21.5	0.04	0.06		
		LC2	64		-4.0	14.0	0.03	0.04	0.04	0.06
					-11.5	21.5	0.05	0.07		
	NEXT	LC1	63		-4.0	14.0	0.03	0.04	0.04	0.05
					-11.5	21.5	0.05	0.07		
		LC2	68		-4.0	14.0	0.03	0.05	0.04	0.06
					-11.5	21.5	0.05	0.07		
Pier 15	Multi-T	LC1	75	5.2	-2.6	7.8	0.02	0.03	0.06	0.09
					-37.8	43.0	0.11	0.16		
		LC2	82		-2.6	7.8	0.02	0.03	0.07	0.10
					-37.8	43.0	0.12	0.17		
	NEXT	LC1	80		-2.6	7.8	0.02	0.03	0.07	0.10
					-37.8	43.0	0.12	0.17		
		LC2	88		-2.6	7.8	0.02	0.03	0.07	0.11
					-37.8	43.0	0.13	0.18		
Pier 16	Multi-T	LC1	72	3.3	-23.8	27.1	0.07	0.10	0.08	0.12
					-39.7	43.0	0.10	0.15		
		LC2	79		-23.8	27.1	0.07	0.10	0.09	0.13
					-39.7	43.0	0.11	0.17		
	NEXT	LC1	77		-23.8	27.1	0.07	0.10	0.09	0.13
					-39.7	43.0	0.11	0.16		
		LC2	84		-23.8	27.1	0.08	0.11	0.10	0.14
					-39.7	43.0	0.12	0.18		
Pier 17	Multi-T	LC1	69	2.6	-30.7	33.3	0.08	0.11	0.14	0.20
					-83.4	86.0	0.20	0.29		
		LC2	75		-30.7	33.3	0.08	0.12	0.15	0.22
					-83.4	86.0	0.22	0.32		
	NEXT	LC1	74		-30.7	33.3	0.08	0.12	0.15	0.22
					-83.4	86.0	0.21	0.31		
		LC2	80		-30.7	33.3	0.09	0.13	0.16	0.23
					-83.4	86.0	0.23	0.34		
Pier 18	Multi-T	LC1	65	1.7	-32.0	33.7	0.07	0.11	0.10	0.15
					-61.0	62.7	0.14	0.20		
		LC2	71		-32.0	33.7	0.08	0.12	0.11	0.17
					-61.0	62.7	0.15	0.22		
	NEXT	LC1	69		-32.0	33.7	0.08	0.11	0.11	0.16
					-61.0	62.7	0.15	0.21		
		LC2	75		-32.0	33.7	0.08	0.12	0.12	0.18
					-61.0	62.7	0.16	0.23		
Pier 19	Multi-T	LC1	61	2.5	-27.8	30.3	0.06	0.09	0.08	0.11
					-42.7	45.2	0.09	0.13		
		LC2	67		-27.8	30.3	0.07	0.10	0.08	0.12
					-42.7	45.2	0.10	0.15		
	NEXT	LC1	65		-27.8	30.3	0.07	0.10	0.08	0.12
					-42.7	45.2	0.10	0.14		
		LC2	71		-27.8	30.3	0.07	0.11	0.09	0.13
					-42.7	45.2	0.11	0.16		

Notes:
[1] - Preliminary loads provided by FIGG on 6 June 2014 and divided by as-built number of piles within the group; LC1 and LC2 refer to one and two lanes of live load, respectively.
[2] - Taken from historic drawings provided by MaineDOT and converted to the NAVD 88 vertical datum by Haley & Aldrich.
[3] - Taken from as-built construction documentation provided by MaineDOT.

Prepared By:	JLL	7/16/2014
Checked By:	BCS	7/16/2014
Reviewed By:	WAC	9/4/2014

HALEY & ALDRICH

CALCULATIONS

File No.	38751-030		
Sheet	1 of 3		
Client	Maine Department of Transportation	**Date**	4/8/2013
Project	Route 1 Viaduct Bridge Replacement - Bath, Maine	**Computed By**	BCS
Subject	Abutment/Pier Footing Bearing Resistance Evaluation	**Checked By**	EFW

OBJECTIVE:

Evaluate bearing resistance for existing bridge abutments/piers bearing on bedrock in accordance with AASHTO LRFD and the MaineDOT Bridge Design Guide (BDG).

REFERENCES:

1 AASHTO LRFD Bridge Design Specifications 6th Edition 2012.
2 MaineDOT Bridge Design Guide 2003, with Interim Revisions through August 2008.
3 Report entitled, "Subsurface Investigations For Carlton Bridge and West Approach in Bath, Maine," prepared by the State Highway Commission Soils Division, dated April 1957.
4 Historic bridge plans entitled, "Carlton Bridge Widening and West Approach between Bath & Woolwich, Sagadahoc County, Maine," Sheet Nos. 36 through 39 of 148, prepared by the State Highway Commission Bridge Division, dated January 1957.

SUMMARY OF EXISTING BRIDGE STRUCTURE:

Based on review of historic information provided by MaineDOT (references 3 and 4), the existing viaduct bridge is supported on two abutments and 19 piers. Select abutments/piers are supported on spread footings.

SUMMARY OF BEDROCK DATA:

BEDROCK CORE DATA:

Test Boring No.	Sample No.	Top Depth (ft, BGS)	Bottom Depth (ft, BGS)	Rock Core Length (ft)	Recovery		RQD	
					in.	%	in.	%
BB-BROV-101	R1	12.3	17.2	4.9	59	100%	35	59%
BB-BROV-102	R1	10.7	15.9	5.2	62	100%	53	85%
BB-BROV-103	R1	49.0	54.0	5.0	60	100%	31	52%

LABORATORY TEST DATA:

Test Boring No.	Sample No.	Top Depth (ft, BGS)	Bottom Depth (ft, BGS)	Peak Compressive Stress (psi)	Bulk Density (pcf)
BB-BROV-101	R1	14.6	15.0	2,321	174
BB-BROV-102	R1	14.0	14.4	1,522	173

BEDROCK DESCRIPTION:

Moderately hard, slightly weathered, medium to fine-grained chlorite/biotite SCHIST with frequent quartz and pegmatite intrusions. Primary joints dipping at low to moderate angles, one steep secondary joint, occasional moderately weathered joints, very close to closely spaced, tight to open.

Moderately hard, slightly weathered, medium to fine-grained GNEISS. Primary joints dipping at low to moderate angles, few steep secondary joints, very close to moderately closely spaced, tight to open with occasional silt coatings on joint surfaces.

For additional details refer to test boring logs.

BEARING RESISTANCE EVALUATION:

SERVICE LIMIT STATE:

Determine service limit state bearing resistance based on presumptive values provided in Table C10.6.2.6.1-1 (attached).

For foliated metamorphic rock (schist) values range from 60 to 80 kips per square foot (ksf).

For service limit state design use 60 ksf (low end of range).

STRENGTH LIMIT STATE:

Calculate upper and lower bound nominal bearing resistance for spread footings bearing on bedrock in accordance with Section 10.8.3.5.4c.

Intact/Tightly Jointed Rock: upper bound value

qp = 2.5 x qu (Eqn. 10.8.3.5.4c-1)

Where: qu = unconfined compressive strengh of rock (ksf)

From previous page, 1.522 ksi < qu < 2.321 ksi…… average = 277 ksf

qu = 277 ksf

qp = 692 ksf

Jointed Rock: lower bound solution

$qp = ((s)^{1/2} + (m(s)^{1/2} + s)^{1/2}) \times qu$ (Eqn. 10.8.3.5.4c-2)

Where: s,m = fractured rock mass parameters specified in Table 10.4.6.4-4.

parameters s and m are a function of rock quality as determined use the Rock Mass Rating (RMR) system.
Determine rock quality in accordance with Tables 10.4.6.4-1 and 10.4.6.4-2 (see attached spreadsheet summary).

Test Boring No.	Sample No.	Recovery (%)	RQD (%)	RMR
BB-BROV-101	R1	100%	59	31
BB-BROV-102	R1	100%	85	50
BB-BROV-103	R1	100%	52	31

For SCHIST and GNEISS bedrock assume Rock Types B and E, respectively. Say Rock Type B is predominant….

BEARING RESISTANCE EVALUATION: (CONT.)

STRENGTH LIMIT STATE: (CONT.)

Jointed Rock: lower bound solution

Calculate rock mass constants (m and s) directly from Eqns. 18 and 19 (Hoek and Brown, 1988)

m / mi = e^((RMR-100/14))…. m = mi x e^((RMR-100)/14)

s = e^((RMR-100)/6))

For Rock Type B and intact rock from Table 1 (Hoek and Brown, 1988)….. mi = 10

Calculate m for RMR = 41 (average of 31 and 50)

m = 0.15

s = 0.000054

qp = $((s)^{1/2} + (m(s)^{1/2} + s)^{1/2})$ x qu (Eqn. 10.8.3.5.4c-2)

qp = 11.4 ksf

Based on review of rock core samples, rock more closely resembles "jointed rock" condition. However, the calculated nominal bearing resistance appears low based on experience. Compare to the presumptive summarized previously. Calculate average of presumptive value and nominal value for "jointed rock"….

Calculate average nominal bearing resistance = 41.4 ksf

Resistance factor for bearing resistance of footings on rock, ϕ = 0.45

Calculate factored bearing resistance = 18.6 ksf

Recommend abutment footings bearing on bedrock be designed based on a factored bearing resistance (Strength Limit State) equal to 20 ksf.

EXTREME EVENT LIMIT STATE:

Recommend abutment footings bearing on bedrock be designed based on a factored bearing resistance (Extreme Event Limit State) equal to 44 ksf.

Rock Mass Rating Field Data Sheet

AASHTO Geomechanics Classification of Rock Masses from Rock Core Recovery, for use with LRFD Design

Reference Elev. _____ | X G/S | ☐ Mudline | Core Barrel Type NQ | Diam. 2

Project Datum NAVD 88

HALEY & ALDRICH

Project Name Route 1 Viaduct Bridge Replacement | **Drilling Contractor** Maine Test Borings

Project Location Bath, Maine | **Drill Rig Type** Mobile Drill B-53 Truck-Mounted Drill Rig | **File Number** 38751-030

Client Maine Department of Transportation | **Haley & Aldrich Rep.** Marleigh Snow | **Date** 4/19/2013

Boring Number		Depth (ft)		Elev. (ft)		Strength of Rock Mass			Recovery		RQD			Joint Spacing			Joint Condition				GW Rating	RMR Rating Sum
	Core Run ID	From	To	From	To	Rock Type	Est. Strength	RMR Rating	In.	%	In.	%	RMR Rating	1° Set	2° Set	RMR Rating	1° Set	2° Set	RMR Rating	Joint Adjust.		
BB-BROV-101	R1	12.3	17.2	15.1	10.2	Schist	334	2	59	100%	34	58%	13	2 in	12 in	10			6	-7	7	**31**
BB-BROV-102	R1	10.7	15.9	6.5	1.3	Gneiss	219	2	62	100%	53	85%	17	2 in	39 in	25			6	-7	7	**50**
BB-BROV-103	R1	49.0	54.0	-41.2	-46.2	Schist	277	2	60	100%	31	52%	13	2 in	12 in	10			6	-7	7	**31**

Strength of Intact Rock Mass

UCS Range of Values		RMR Rating
>4320 ksf	>30,000 psi	15
2160 - 4320 ksf	15,000 - 30,000 psi	12
1080 - 2160 ksf	7,500 - 15,000 psi	7
520 - 1080 ksf	3,611 - 7,500 psi	4
215 - 520 ksf	1,495 - 3,611 psi	2
70 - 215 ksf	486 - 1,495 psi	1
20 - 70 ksf	138 - 486 psi	0

Approximate Rock Strength Field Guidelines

Extremely weak rock	36 - 145 psi	Indents by thumbnail (stiff fault gouge)
Very weak rock	145 - 725 psi	Crumbles under firm blow with pick point
Weak rock	725 - 3,625 psi	Shallow indentations with firm blow of pick
Medium strong rock	3,625 - 7,250 psi	Specimen fractured with single firm blow
Strong rock	7,250 - 14,500 psi	Specimen requires more than one blow to fracture
Very strong rock	14,500 - 36,250 psi	Specimen requires many blows to fracture
Extremely. strong rock	> 36,250 psi	Specimen can only be chipped with geological pick

Drill Core Quality from RQD

RQD	RMR Rating
100% - 90%	20
90% - 75%	17
75% - 50%	13
50% - 25%	8
<25%	3

Joint Spacing	RMR Rating
> 10 ft	30
3 ft - 10 ft	25
1 ft - 3 ft	20
2 in. - 1 ft	10
< 2 in.	5

Condition of Joints	RMR Rating
1. ▪Very rough surfaces ▪Not continuous ▪No separation (tight) ▪Hard wall rock in joint	25
2. ▪Slightly rough surfaces ▪Opening/separation <0.05 in. ▪Hard joint wall rock	20
3. ▪Slighty rough surfaces ▪Opening/separation <0.05 in. ▪Soft joint wall rock	12
4. ▪Slickensided surfaces **or** ▪Gouge <0.2 in. thick **or** ▪Joints open 0.05 - 0.2 in. ▪Continuous joints	6
5. ▪Soft gouge >0.2 in. thick or ▪Joints open >0.2 in. ▪Continuous joints	0

General Groundwater Conditions	RMR Rating
Completely dry	10
Moist only or interstitial water	7
Water under moderate pressure	4
Severe water flow/infiltration/problems	0

Adjustment for Joint Orientation (foundations only)

	RMR Rating
Very Favorable	0
Favorable	-2
Fair	-7
Unfavorable	-15
Very Unfavorable	-25

Instructions:
1. Record data from individual rock core runs. Using RMR criteria, enter RMR Ratings for each evaluation category (Strength, RQD, Joints, groundwater, etc.).
2. Sum all RMR Ratings from shaded boxes into right hand column. Value should be between 0 - 100.

BORING NO: **BB-BROV-101,102,103**

Table C10.6.2.6.1-1—Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Type of Bearing Material	Consistency in Place	Bearing Resistance (ksf)	
		Ordinary Range	Recommended Value of Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Very hard, sound rock	120–200	160
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Hard sound rock	60–80	70
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Hard sound rock	30–50	40
Weathered or broken bedrock of any kind, except highly argillaceous rock (shale)	Medium hard rock	16–24	20
Compaction shale or other highly argillaceous rock in sound condition	Medium hard rock	16–24	20
Well-graded mixture of fine- and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very dense	16–24	20
Gravel, gravel-sand mixture, boulder-gravel mixtures (GW, GP, SW, SP)	Very dense Medium dense to dense Loose	12–20 8–14 4–12	14 10 6
Coarse to medium sand, and with little gravel (SW, SP)	Very dense Medium dense to dense Loose	8–12 4–8 2–6	8 6 3
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Fine sand, silty or clayey medium to fine sand (SP, SM, SC)	Very dense Medium dense to dense Loose	6–10 4–8 2–4	6 5 3
Homogeneous inorganic clay, sandy or silty clay (CL, CH)	Very dense Medium dense to dense Loose	6–12 2–6 1–2	8 4 1
Inorganic silt, sandy or clayey silt, varved silt-clay-fine sand (ML, MH)	Very stiff to hard Medium stiff to stiff Soft	4–8 2–6 1–2	6 3 1

10.6.2.6.2—Semiempirical Procedures for Bearing Resistance

Bearing resistance on rock shall be determined using empirical correlation to the Geomechanic Rock Mass Rating System, RMR, as specified in Article 10.4.6.4. Local experience should be considered in the use of these semi-empirical procedures.

If the recommended value of presumptive bearing resistance exceeds either the unconfined compressive strength of the rock or the nominal resistance of the concrete, the presumptive bearing resistance shall be taken as the lesser of the unconfined compressive strength of the rock or the nominal resistance of the concrete. The nominal resistance of concrete shall be taken as $0.3f'_c$.

SERVICE LIMIT STATE SPREAD FOOTING SETTLEMENT EVALUATION

ROUTE 1 VIADUCT BRIDGE REHABILITATION

MAINEDOT WIN 19273.00

BATH, MAINE

REF.: AASHTO LRFD SECTION 10.6.2.4.4 - SETTLEMENT OF FOOTINGS ON ROCK

Substructure Location	Superstructure Alternative	Service I Limit State Loads[1]				Effective Footing Dimensions[3]				Elastic Shape and Rigidity Factors[4]		Rigidity/Dim. Influence Coeff. (I_p)[5]	Settlement[6] (in.)
						Effective Footing Width		Effective Footing Length					
		Load Combination[2]	P_u (kip)	M_{ut} (kip-ft)	M_{ul} (kip-ft)	Eccentricity (e_x, ft)	Width (B', ft)	Eccentricity (e_y, ft)	Length (L', ft)	L'/B'	β_z		
Pier 1	Multi-T	LC1	914	1,616	1,057	1.77	9.46	1.16	7.69	0.81	1.08	0.83	0.01
		LC2	1,012	1,224	1,057	1.21	10.58	1.04	7.91	0.75	1.08	0.80	0.01
	NEXT	LC1	983	1,616	1,183	1.64	9.71	1.20	7.59	0.78	1.08	0.82	0.01
		LC2	1,081	1,224	1,183	1.13	10.74	1.09	7.81	0.73	1.08	0.79	0.01
Pier 2	Multi-T	LC1	914	1,424	1,243	1.56	9.88	1.36	7.28	0.74	1.08	0.79	0.01
		LC2	1,000	1,079	1,243	1.08	10.84	1.24	7.51	0.69	1.08	0.77	0.01
	NEXT	LC1	984	1,424	1,390	1.45	10.11	1.41	7.17	0.71	1.08	0.78	0.01
		LC2	1,070	1,079	1,390	1.01	10.98	1.30	7.40	0.67	1.08	0.76	0.01
Pier 3	Multi-T	LC1	930	1,454	1,377	1.56	9.87	1.48	7.04	0.71	1.08	0.78	0.01
		LC2	1,019	1,101	1,377	1.08	10.84	1.35	7.30	0.67	1.08	0.76	0.01
	NEXT	LC1	1,001	1,454	1,546	1.45	10.09	1.54	6.91	0.68	1.08	0.77	0.01
		LC2	1,089	1,101	1,546	1.01	10.98	1.42	7.16	0.65	1.08	0.75	0.01
Pier 4	Multi-T	LC1	933	1,436	1,497	1.54	9.92	1.60	6.79	0.68	1.08	0.77	0.01
		LC2	1,020	1,088	1,497	1.07	10.87	1.47	7.06	0.65	1.08	0.75	0.01
	NEXT	LC1	1,003	1,436	1,673	1.43	10.14	1.67	6.66	0.66	1.08	0.75	0.01
		LC2	1,090	1,088	1,673	1.00	11.00	1.53	6.93	0.63	1.08	0.73	0.01
Pier 8	Multi-T	LC1	984	1,506	1,650	1.53	9.94	1.68	6.65	0.67	1.08	0.76	0.01
		LC2	1,075	1,141	1,650	1.06	10.88	1.53	6.93	0.64	1.08	0.74	0.01
	NEXT	LC1	1,058	1,506	1,702	1.42	10.15	1.61	6.78	0.67	1.08	0.76	0.01
		LC2	1,149	1,141	1,702	0.99	11.01	1.48	7.04	0.64	1.08	0.74	0.01
Pier 9	Multi-T	LC1	992	1,523	1,624	1.54	9.93	1.64	6.73	0.68	1.08	0.76	0.01
		LC2	1,085	1,154	1,624	1.06	10.87	1.50	7.01	0.64	1.08	0.74	0.01
	NEXT	LC1	1,067	1,523	1,696	1.43	10.15	1.59	6.82	0.67	1.08	0.76	0.01
		LC2	1,159	1,154	1,696	1.00	11.01	1.46	7.07	0.64	1.08	0.74	0.01
Pier 13	Multi-T	LC1	1,000	1,543	1,936	1.54	9.91	1.94	6.13	0.62	1.08	0.73	0.01
		LC2	1,094	1,169	1,936	1.07	10.86	1.77	6.46	0.59	1.08	0.71	0.01
	NEXT	LC1	1,075	1,543	2,006	1.44	10.13	1.87	6.27	0.62	1.08	0.73	0.01
		LC2	1,169	1,169	2,006	1.00	11.00	1.72	6.57	0.60	1.08	0.72	0.01
Pier 14	Multi-T	LC1	1,073	1,743	2,003	1.62	9.75	1.87	6.27	0.64	1.08	0.74	0.01
		LC2	1,178	1,321	2,003	1.12	10.76	1.70	6.60	0.61	1.08	0.73	0.01
	NEXT	LC1	1,154	1,743	2,242	1.51	9.98	1.94	6.11	0.61	1.08	0.72	0.01
		LC2	1,260	1,321	2,242	1.05	10.90	1.78	6.44	0.59	1.08	0.71	0.01

Notes:

[1] - SER I Limit State loads provided by Figg on 6 June 2014 and are based on one continuous Multi-T and NEXT beam superstructures, bridge bearing friction coefficient equal to 0.10 and unmodified HL-93 live load.

[2] - LC1 and LC2 refer to one and two lanes of live load, respectively.

[3] - Refer to AASHTO LRFD Section 10.6.1.3 - Effective Footing Dimensions.

[4] - Refer to AASHTO LRFD Table 10.6.2.4.2-1 assuming a rigid foundation.

[5] - Refer to AASHTO LRFD Eqn. 10.6.2.4.4-4.

[6] - Refer to Variables/Input Parameters (this sheet) and AASHTO LRFD Eqn. 10.6.2.4.4-3.

Prepared By:	BCS	6/27/2014
Checked By:	JLL	8/15/2014
Reviewed By:	WAC	9/4/2014

VARIABLES/INPUT PARAMETERS:

Existing Spread Footing Dimensions: see historic bridge drawings sheet 36 of 148

Footing Width, B_x = 13 ft (transverse to bridge alignment)

Footing Length, L_y = 10 ft (parallel to bridge alignment)

Miscellaneous Input Parameters:

Poisson's Ratio, ν = 0.05 taken from laboratory test results included in Appendix C of 17 September 2013 PGDR

Calculation of Rock Mass Modulus (E_m): refer to AASHTO LRFD Section 10.4.6.5 - Rock Mass Deformation

Rock Mass Ration, RMR = 50 (conservatively assumed)

Rock Mass Modulus, E_{m1} = 1,450 ksi (refer to AASHTO LRFD Eqn. 10.4.6.5-1)

E_m/E_i = 0.40 refer to AASHTO LRFD Table 10.4.6.5-1 based on RQD_{avg} = 70% and jts. varying between open and closed

Elastic Modulus, E_i = 685,000 ksi (taken from laboratory test results included in Appendix C of 17 September 2013 PGDR)

Rock Mass Modulus, E_{m2} = 274,000 ksi (refer to AASHTO LRFD Eqn. 10.4.6.5-2)

Conservatively use lower value of rock mass modulus for settlement calculation....

STRENGTH LIMIT STATE SPREAD FOOTING BEARING RESISTANCE EVALUATION (MULTI-T SUPERSTRUCTURE ALTERNATIVE)

ROUTE 1 VIADUCT BRIDGE REHABILITATION

MAINEDOT WIN 19273.00

BATH, MAINE

Substructure Location	Strength I Limit State Loads[1]					Effective Footing Dimensions[3]				Applied Bearing Pressure (ksf)	Exceed Limiting Value (Yes/No)
						Effective Footing Width		Effective Footing Length			
	Load Combination[2]	Max. or Min.	P_u (kip)	M_{ut} (kip-ft)	M_{ul} (kip-ft)	Eccentricity (e_x, ft)	Width (B', ft)	Eccentricity (e_y, ft)	Length (L', ft)		
Pier 1	LC1	Min.	916	2,828	1,057	3.1	6.8	1.2	7.7	17.4	NO
		Max.	1,248	2,828	1,057	2.3	8.5	0.8	8.3	17.7	NO
	LC2	Min.	1,087	2,143	1,057	2.0	9.1	1.0	8.1	14.9	NO
		Max.	1,419	2,143	1,057	1.5	10.0	0.7	8.5	16.7	NO
Pier 2	LC1	Min.	900	2,493	1,243	2.8	7.5	1.4	7.2	16.7	NO
		Max.	1,239	2,493	1,243	2.0	9.0	1.0	8.0	17.3	NO
	LC2	Min.	1,052	1,889	1,243	1.8	9.4	1.2	7.6	14.6	NO
		Max.	1,390	1,889	1,243	1.4	10.3	0.9	8.2	16.5	NO
Pier 3	LC1	Min.	917	2,544	1,377	2.8	7.5	1.5	7.0	17.6	NO
		Max.	1,261	2,544	1,377	2.0	9.0	1.1	7.8	18.0	NO
	LC2	Min.	1,072	1,927	1,377	1.8	9.4	1.3	7.4	15.3	NO
		Max.	1,416	1,927	1,377	1.4	10.3	1.0	8.1	17.1	NO
Pier 4	LC1	Min.	919	2,514	1,497	2.7	7.5	1.6	6.7	18.1	NO
		Max.	1,264	2,514	1,497	2.0	9.0	1.2	7.6	18.4	NO
	LC2	Min.	1,071	1,904	1,497	1.8	9.4	1.4	7.2	15.7	NO
		Max.	1,416	1,904	1,497	1.3	10.3	1.1	7.9	17.4	NO
Pier 8	LC1	Min.	968	2,635	1,650	2.7	7.6	1.7	6.6	19.4	NO
		Max.	1,332	2,635	1,650	2.0	9.0	1.2	7.5	19.6	NO
	LC2	Min.	1,128	1,996	1,650	1.8	9.5	1.5	7.1	16.9	NO
		Max.	1,492	1,996	1,650	1.3	10.3	1.1	7.8	18.6	NO
Pier 9	LC1	Min.	977	2,665	1,624	2.7	7.5	1.7	6.7	19.4	NO
		Max.	1,344	2,665	1,624	2.0	9.0	1.2	7.6	19.6	NO
	LC2	Min.	1,138	2,019	1,624	1.8	9.5	1.4	7.1	16.8	NO
		Max.	1,505	2,019	1,624	1.3	10.3	1.1	7.8	18.6	NO
Pier 13	LC1	Min.	985	2,700	1,936	2.7	7.5	2.0	6.1	21.6	YES
		Max.	1,355	2,700	1,936	2.0	9.0	1.4	7.1	21.0	YES
	LC2	Min.	1,149	2,046	1,936	1.8	9.4	1.7	6.6	18.4	NO
		Max.	1,518	2,046	1,936	1.3	10.3	1.3	7.4	19.8	NO
Pier 14	LC1	Min.	1,063	3,051	2,003	2.9	7.3	1.9	6.2	23.5	YES
		Max.	1,458	3,051	2,003	2.1	8.8	1.4	7.3	22.8	YES
	LC2	Min.	1,248	2,311	2,003	1.9	9.3	1.6	6.8	19.8	NO
		Max.	1,642	2,311	2,003	1.4	10.2	1.2	7.6	21.3	YES

Notes:

[1] - STR I Limit State loads provided by Figg on 6 June 2014 and are based on one continuous Multi-T beam superstructure, bridge bearing friction coefficient equal to 0.10 and unmodified HL-93 live load.

[2] - LC1 and LC2 refer to one and two lanes of live load, respectively.

[3] - Refer to AASHTO LRFD Section 10.6.1.3 - Effective Footing Dimensions.

VARIABLES/INPUT PARAMETERS:

Existing Spread Footing Dimensions: see historic bridge drawings sheet 36 of 148

Footing Width, B_x = 13 ft (transverse to bridge alignment)

Footing Length, L_y = 10 ft (parallel to bridge alignment)

Factored STR Limit State Bearing Resistance = 20.0 ksf (see PGDR)

Prepared By:	BCS	6/27/2014
Checked By:	JLL	8/15/2014
Reviewed By:	WAC	9/4/2014

STRENGTH LIMIT STATE SPREAD FOOTING ECCENTRICITY EVALUATION (MULTI-T SUPERSTRUCTURE ALTERNATIVE)

ROUTE 1 VIADUCT BRIDGE REHABILITATION

MAINEDOT WIN 19273.00

BATH, MAINE

REF.: AASHTO LRFD SECTION 10.6.3.3 - ECCENTRIC LOAD LIMITATIONS

Substructure Location	Strength I Limit State Loads[1]					Eccentricity[3] (e_x, ft)	Exceed Limiting Value[4]	Eccentricity[3] (e_y, ft)	Exceed Limiting Value[4]
	Load Combination[2]	Max. or Min.	P_u (kip)	M_{ut} (kip-ft)	M_{ul} (kip-ft)				
Pier 1	LC1	Min.	916	2,828	1,057	3.09	NO	1.15	NO
		Max.	1,248	2,828	1,057	2.27	NO	0.85	NO
	LC2	Min.	1,087	2,143	1,057	1.97	NO	0.97	NO
		Max.	1,419	2,143	1,057	1.51	NO	0.74	NO
Pier 2	LC1	Min.	900	2,493	1,243	2.77	NO	1.38	NO
		Max.	1,239	2,493	1,243	2.01	NO	1.00	NO
	LC2	Min.	1,052	1,889	1,243	1.80	NO	1.18	NO
		Max.	1,390	1,889	1,243	1.36	NO	0.89	NO
Pier 3	LC1	Min.	917	2,544	1,377	2.77	NO	1.50	NO
		Max.	1,261	2,544	1,377	2.02	NO	1.09	NO
	LC2	Min.	1,072	1,927	1,377	1.80	NO	1.28	NO
		Max.	1,416	1,927	1,377	1.36	NO	0.97	NO
Pier 4	LC1	Min.	919	2,514	1,497	2.74	NO	1.63	NO
		Max.	1,264	2,514	1,497	1.99	NO	1.18	NO
	LC2	Min.	1,071	1,904	1,497	1.78	NO	1.40	NO
		Max.	1,416	1,904	1,497	1.34	NO	1.06	NO
Pier 8	LC1	Min.	968	2,635	1,650	2.72	NO	1.70	NO
		Max.	1,332	2,635	1,650	1.98	NO	1.24	NO
	LC2	Min.	1,128	1,996	1,650	1.77	NO	1.46	NO
		Max.	1,492	1,996	1,650	1.34	NO	1.11	NO
Pier 9	LC1	Min.	977	2,665	1,624	2.73	NO	1.66	NO
		Max.	1,344	2,665	1,624	1.98	NO	1.21	NO
	LC2	Min.	1,138	2,019	1,624	1.77	NO	1.43	NO
		Max.	1,505	2,019	1,624	1.34	NO	1.08	NO
Pier 13	LC1	Min.	985	2,700	1,936	2.74	NO	1.97	NO
		Max.	1,355	2,700	1,936	1.99	NO	1.43	NO
	LC2	Min.	1,149	2,046	1,936	1.78	NO	1.68	NO
		Max.	1,518	2,046	1,936	1.35	NO	1.28	NO
Pier 14	LC1	Min.	1,063	3,051	2,003	2.87	NO	1.88	NO
		Max.	1,458	3,051	2,003	2.09	NO	1.37	NO
	LC2	Min.	1,248	2,311	2,003	1.85	NO	1.60	NO
		Max.	1,642	2,311	2,003	1.41	NO	1.22	NO

VARIABLES/INPUT PARAMETERS:

Existing Spread Footing Dimensions: see historic bridge drawings sheet 36 of 148

Footing Width, B_x = 13 ft (transverse to bridge alignment)

Footing Length, L_y = 10 ft (parallel to bridge alignment)

Calculation of Limiting Eccentricity[4]: refer to AASHTO LRFD Section 10.6.3.3 - Eccentric Load Limitations

Limiting Eccentricty, $e_{x(limit)}$ = 5.85

Limiting Eccentricty, $e_{y(limit)}$ = 4.50

Notes:

[1] - STR I Limit State loads provided by Figg on 6 June 2014 and are based on one continuous Multi-T beam superstructure, bridge bearing friction coefficient equal to 0.10 and unmodified HL-93 live load.

[2] - LC1 and LC2 refer to one and two lanes of live load, respectively.

[3] - Refer to AASHTO LRFD Section 10.6.1.3 - Effective Footing Dimensions.

[4] - Eccentricity is limited to 0.45 of the corresponding footing dimensions B or L, for footings on rock.

Prepared By:	BCS	6/27/2014
Checked By:	JLL	8/15/2014
Reviewed By:	WAC	9/4/2014

STRENGTH LIMIT STATE SPREAD FOOTING SLIDING RESISTANCE EVALUATION (MULTI-T SUPERSTRUCTURE ALTERNATIVE)

ROUTE 1 VIADUCT BRIDGE REHABILITATION

MAINEDOT WIN 19273.00

BATH, MAINE

REF.: AASHTO LRFD SECTION 10.6.3.4 - FAILURE BY SLIDING

Substructure Location	Strength I Limit State Loads[1]						Factored Sliding Resistance = P_u x tan(δ) (kips)	Is Factored Resistance > V_{uy}?
	Load Combination[2]	Max. or Min.	P_u (kip)	M_{ut} (kip-ft)	M_{ul} (kip-ft)	V_{uy} (kip)		
Pier 1	LC1	Min.	916	2,828	1,057	58	223	YES
		Max.	1,248	2,828	1,057	58	304	YES
	LC2	Min.	1,087	2,143	1,057	58	265	YES
		Max.	1,419	2,143	1,057	58	346	YES
Pier 2	LC1	Min.	900	2,493	1,243	59	219	YES
		Max.	1,239	2,493	1,243	59	302	YES
	LC2	Min.	1,052	1,889	1,243	59	257	YES
		Max.	1,390	1,889	1,243	59	339	YES
Pier 3	LC1	Min.	917	2,544	1,377	59	224	YES
		Max.	1,261	2,544	1,377	59	308	YES
	LC2	Min.	1,072	1,927	1,377	59	261	YES
		Max.	1,416	1,927	1,377	59	345	YES
Pier 4	LC1	Min.	919	2,514	1,497	59	224	YES
		Max.	1,264	2,514	1,497	59	308	YES
	LC2	Min.	1,071	1,904	1,497	59	261	YES
		Max.	1,416	1,904	1,497	59	345	YES
Pier 8	LC1	Min.	968	2,635	1,650	56	236	YES
		Max.	1,332	2,635	1,650	56	325	YES
	LC2	Min.	1,128	1,996	1,650	56	275	YES
		Max.	1,492	1,996	1,650	56	364	YES
Pier 9	LC1	Min.	977	2,665	1,624	55	238	YES
		Max.	1,344	2,665	1,624	55	328	YES
	LC2	Min.	1,138	2,019	1,624	55	278	YES
		Max.	1,505	2,019	1,624	55	367	YES
Pier 13	LC1	Min.	985	2,700	1,936	66	240	YES
		Max.	1,355	2,700	1,936	66	330	YES
	LC2	Min.	1,149	2,046	1,936	66	280	YES
		Max.	1,518	2,046	1,936	66	370	YES
Pier 14	LC1	Min.	1,063	3,051	2,003	68	259	YES
		Max.	1,458	3,051	2,003	68	356	YES
	LC2	Min.	1,248	2,311	2,003	68	304	YES
		Max.	1,642	2,311	2,003	68	400	YES

VARIABLES/INPUT PARAMETERS:

Existing Spread Footing Dimensions: see historic bridge drawings sheet 36 of 148

Footing Width, B_x = 13 ft (transverse to bridge alignment)

Footing Length, L_y = 10 ft (parallel to bridge alignment)

Interface friction between concrete and rock = 26.0 deg.

tan (δ) = 0.49

Notes:

[1] - STR I Limit State loads provided by Figg on 6 June 2014 and are based on one continuous Multi-T beam superstructure, bridge bearing friction coefficient equal to 0.10 and unmodified HL-93 live load.

[2] - LC1 and LC2 refer to one and two lanes of live load, respectively.

[3] - Refer to AASHTO LRFD Section 10.6.1.3 - Effective Footing Dimensions.

Prepared By:	BCS	6/27/2014
Checked By:	JLL	8/15/2014
Reviewed By:	WAC	9/4/2014

EXTREME EVENT LIMIT STATE SPREAD FOOTING BEARING RESISTANCE EVALUATION (MULTI-T SUPERSTRUCTURE ALTERNATIVE)

ROUTE 1 VIADUCT BRIDGE REHABILITATION

MAINEDOT WIN 19273.00

BATH, MAINE

Substructure Location	Extreme II Limit State Loads[1]							Effective Footing Dimensions[3]				Applied Bearing Pressure (ksf)	Exceed Limiting Value (Yes/No)
	Load Combination[2]	Max. or Min.	P_u (kip)	M_{ut} (kip-ft)	M_{ul} (kip-ft)	V_{ux} (kip)	V_{uy} (kip)	Effective Footing Width		Effective Footing Length			
								Eccentricity (e_x, ft)	Width (B', ft)	Eccentricity (e_y, ft)	Length (L', ft)		
Pier 1	LC1	Min.	732	808	1,675	0	150	1.1	10.8	2.3	5.4	12.5	NO
		Max.	1,064	808	1,675	0	150	0.8	11.5	1.6	6.9	13.5	NO
	LC2	Min.	781	612	1,675	0	150	0.8	11.4	2.1	5.7	12.0	NO
		Max.	1,113	612	1,675	0	150	0.5	11.9	1.5	7.0	13.4	NO
Pier 2	LC1	Min.	739	712	1,675	0	150	1.0	11.1	2.3	5.5	12.2	NO
		Max.	1,077	712	1,675	0	150	0.7	11.7	1.6	6.9	13.4	NO
	LC2	Min.	782	540	1,675	0	150	0.7	11.6	2.1	5.7	11.8	NO
		Max.	1,120	540	1,675	0	150	0.5	12.0	1.5	7.0	13.3	NO
Pier 3	LC1	Min.	752	727	1,625	0	150	1.0	11.1	2.2	5.7	12.0	NO
		Max.	1,096	727	1,625	0	150	0.7	11.7	1.5	7.0	13.3	NO
	LC2	Min.	796	551	1,625	0	150	0.7	11.6	2.0	5.9	11.6	NO
		Max.	1,140	551	1,625	0	150	0.5	12.0	1.4	7.1	13.3	NO
Pier 4	LC1	Min.	756	718	1,638	0	150	0.9	11.1	2.2	5.7	12.0	NO
		Max.	1,101	718	1,638	0	150	0.7	11.7	1.5	7.0	13.4	NO
	LC2	Min.	799	544	1,638	0	150	0.7	11.6	2.1	5.9	11.6	NO
		Max.	1,144	544	1,638	0	150	0.5	12.0	1.4	7.1	13.3	NO
Pier 8	LC1	Min.	797	753	1,588	0	150	0.9	11.1	2.0	6.0	11.9	NO
		Max.	1,161	753	1,588	0	150	0.6	11.7	1.4	7.3	13.7	NO
	LC2	Min.	843	570	1,588	0	150	0.7	11.6	1.9	6.2	11.6	NO
		Max.	1,206	570	1,588	0	150	0.5	12.1	1.3	7.4	13.6	NO
Pier 9	LC1*	Min.	804	761	1,625	0	150	0.9	11.1	2.0	6.0	12.2	NO
		Max.	1,170	761	1,625	0	150	0.7	11.7	1.4	7.2	13.8	NO
	LC2*	Min.	850	577	1,625	0	150	0.7	11.6	1.9	6.2	11.8	NO
		Max.	1,217	577	1,625	0	150	0.5	12.1	1.3	7.3	13.8	NO
	LC1**	Min.	804	2,386	1,624	150	55	3.0	7.1	2.0	6.0	19.1	NO
		Max.	1,170	2,386	1,624	150	55	2.0	8.9	1.4	7.2	18.2	NO
	LC2**	Min.	850	2,202	1,624	150	55	2.6	7.8	1.9	6.2	17.6	NO
		Max.	1,217	2,202	1,624	150	55	1.8	9.4	1.3	7.3	17.7	NO
Pier 13	LC1	Min.	810	771	1,600	0	150	1.0	11.1	2.0	6.0	12.1	NO
		Max.	1,179	771	1,600	0	150	0.7	11.7	1.4	7.3	13.8	NO
	LC2	Min.	856	584	1,600	0	150	0.7	11.6	1.9	6.3	11.7	NO
		Max.	1,226	584	1,600	0	150	0.5	12.0	1.3	7.4	13.8	NO
Pier 14	LC1*	Min.	865	872	1,613	0	150	1.0	11.0	1.9	6.3	12.6	NO
		Max.	1,259	872	1,613	0	150	0.7	11.6	1.3	7.4	14.6	NO
	LC2*	Min.	918	660	1,613	0	150	0.7	11.6	1.8	6.5	12.2	NO
		Max.	1,312	660	1,613	0	150	0.5	12.0	1.2	7.5	14.5	NO
	LC1**	Min.	865	2,484	2,003	150	68	2.9	7.3	2.3	5.4	22.2	NO
		Max.	1,259	2,484	2,003	150	68	2.0	9.1	1.6	6.8	20.4	NO
	LC2**	Min.	918	2,273	2,003	150	68	2.5	8.0	2.2	5.6	20.2	NO
		Max.	1,312	2,273	2,003	150	68	1.7	9.5	1.5	6.9	19.8	NO

VARIABLES/INPUT PARAMETERS:

Existing Spread Footing Dimensions: see historic bridge drawings sheet 36 of 148

Footing Width, B_x = 13 ft (transverse to bridge alignment)

Footing Length, L_y = 10 ft (parallel to bridge alignment)

Factored STR Limit State Bearing Resistance = 44.0 ksf (see PGDR)

Notes:

[1] - STR I Limit State loads provided by Figg on 6 June 2014 and are based on one continuous Multi-T beam superstructure, bridge bearing friction coefficient equal to 0.10 and unmodified HL-93 live load.

[2] - LC1 and LC2 refer to one and two lanes of live load, respectively.

[3] - Refer to AASHTO LRFD Section 10.6.1.3 - Effective Footing Dimensions.

Prepared By:	BCS	6/27/2014
Checked By:	JLL	8/15/2014
Reviewed By:	WAC	9/4/2014

EXTREME EVENT LIMIT STATE SPREAD FOOTING ECCENTRICITY EVALUATION (MULTI-T SUPERSTRUCTURE ALTERNATIVE)

ROUTE 1 VIADUCT BRIDGE REHABILITATION

MAINEDOT WIN 19273.00

BATH, MAINE

REF.: AASHTO LRFD SECTION 10.6.3.3 - ECCENTRIC LOAD LIMITATIONS

Substructure Location	Extreme II Limit State Loads[1]					Eccentricity (e_x, ft)	Exceed Limiting Value	Eccentricity (e_y, ft)	Exceed Limiting Value
	Load Combination[2]	Max. or Min.	P_u (kip)	M_{ut} (kip-ft)	M_{ul} (kip-ft)				
Pier 1	LC1	Min.	732	808	1,675	1.10	NO	2.29	NO
		Max.	1,064	808	1,675	0.76	NO	1.57	NO
	LC2	Min.	781	612	1,675	0.78	NO	2.14	NO
		Max.	1,113	612	1,675	0.55	NO	1.50	NO
Pier 2	LC1	Min.	739	712	1,675	0.96	NO	2.27	NO
		Max.	1,077	712	1,675	0.66	NO	1.56	NO
	LC2	Min.	782	540	1,675	0.69	NO	2.14	NO
		Max.	1,120	540	1,675	0.48	NO	1.50	NO
Pier 3	LC1	Min.	752	727	1,625	0.97	NO	2.16	NO
		Max.	1,096	727	1,625	0.66	NO	1.48	NO
	LC2	Min.	796	551	1,625	0.69	NO	2.04	NO
		Max.	1,140	551	1,625	0.48	NO	1.43	NO
Pier 4	LC1	Min.	756	718	1,638	0.95	NO	2.17	NO
		Max.	1,101	718	1,638	0.65	NO	1.49	NO
	LC2	Min.	799	544	1,638	0.68	NO	2.05	NO
		Max.	1,144	544	1,638	0.48	NO	1.43	NO
Pier 8	LC1	Min.	797	753	1,588	0.94	NO	1.99	NO
		Max.	1,161	753	1,588	0.65	NO	1.37	NO
	LC2	Min.	843	570	1,588	0.68	NO	1.88	NO
		Max.	1,206	570	1,588	0.47	NO	1.32	NO
Pier 9	LC1*	Min.	804	761	1,625	0.95	NO	2.02	NO
		Max.	1,170	761	1,625	0.65	NO	1.39	NO
	LC2*	Min.	850	577	1,625	0.68	NO	1.91	NO
		Max.	1,217	577	1,625	0.47	NO	1.34	NO
	LC1**	Min.	804	2,386	1,624	2.97	NO	2.02	NO
		Max.	1,170	2,386	1,624	2.04	NO	1.39	NO
	LC2**	Min.	850	2,202	1,624	2.59	NO	1.91	NO
		Max.	1,217	2,202	1,624	1.81	NO	1.33	NO
Pier 13	LC1	Min.	810	771	1,600	0.95	NO	1.98	NO
		Max.	1,179	771	1,600	0.65	NO	1.36	NO
	LC2	Min.	856	584	1,600	0.68	NO	1.87	NO
		Max.	1,226	584	1,600	0.48	NO	1.31	NO
Pier 14	LC1*	Min.	865	872	1,613	1.01	NO	1.86	NO
		Max.	1,259	872	1,613	0.69	NO	1.28	NO
	LC2*	Min.	918	660	1,613	0.72	NO	1.76	NO
		Max.	1,312	660	1,613	0.50	NO	1.23	NO
	LC1**	Min.	865	2,484	2,003	2.87	NO	2.32	NO
		Max.	1,259	2,484	2,003	1.97	NO	1.59	NO
	LC2**	Min.	918	2,273	2,003	2.48	NO	2.18	NO
		Max.	1,312	2,273	2,003	1.73	NO	1.53	NO

VARIABLES/INPUT PARAMETERS:

Existing Spread Footing Dimensions: see historic bridge drawings sheet 36 of 148

Footing Width, B_x = 13 ft (transverse to bridge alignment)

Footing Length, L_y = 10 ft (parallel to bridge alignment)

Calculation of Limiting Eccentricity[4]: refer to AASHTO LRFD Section 10.6.3.3 - Eccentric Load Limitations

Limiting Eccentricty, $e_{x(limit)}$ = 5.85

Limiting Eccentricty, $e_{y(limit)}$ = 4.50

Notes:

[1] - EE II Limit State loads provided by Figg on 6 June 2014 and are based on one continuous Multi-T beam superstructure, bridge bearing friction coefficient equal to 0.10 and unmodified HL-93 live load.

[2] - LC1 and LC2 refer to one and two lanes of live load, respectively.

[3] - Refer to AASHTO LRFD Section 10.6.1.3 - Effective Footing Dimensions.

[4] - Eccentricity is limited to 0.45 of the corresponding footing dimensions B or L, for footings on rock.

Prepared By:	BCS	6/27/2014
Checked By:	JLL	8/15/2014
Reviewed By:	WAC	9/4/2014

EXTREME EVENT LIMIT STATE SPREAD FOOTING SLIDING RESISTANCE EVALUATION (MULTI-T SUPERSTRUCTURE ALTERNATIVE)

ROUTE 1 VIADUCT BRIDGE REHABILITATION

MAINEDOT WIN 19273.00

BATH, MAINE

REF.: AASHTO LRFD SECTION 10.6.3.4 - FAILURE BY SLIDING

Substructure Location	Extreme II Limit State Loads[1]							Sliding Resistance = P_u x tan(δ)		Is Factored Resistance > V_{uy}?
	Load Combination[2]	Max. or Min.	P_u (kip)	M_{ut} (kip-ft)	M_{ul} (kip-ft)	V_{ux} (kip)	V_{uy} (kip)	Nominal Resistance (kip)	Factored Resistance (kip)	
Pier 1	LC1	Min.	732	808	1,675	0	150	357	179	YES
		Max.	1,064	808	1,675	0	150	519	259	YES
	LC2	Min.	781	612	1,675	0	150	381	190	YES
		Max.	1,113	612	1,675	0	150	543	271	YES
Pier 2	LC1	Min.	739	712	1,675	0	150	360	180	YES
		Max.	1,077	712	1,675	0	150	525	263	YES
	LC2	Min.	782	540	1,675	0	150	381	191	YES
		Max.	1,120	540	1,675	0	150	546	273	YES
Pier 3	LC1	Min.	752	727	1,625	0	150	367	183	YES
		Max.	1,096	727	1,625	0	150	535	267	YES
	LC2	Min.	796	551	1,625	0	150	388	194	YES
		Max.	1,140	551	1,625	0	150	556	278	YES
Pier 4	LC1	Min.	756	718	1,638	0	150	369	184	YES
		Max.	1,101	718	1,638	0	150	537	268	YES
	LC2	Min.	799	544	1,638	0	150	390	195	YES
		Max.	1,144	544	1,638	0	150	558	279	YES
Pier 8	LC1	Min.	797	753	1,588	0	150	389	194	YES
		Max.	1,161	753	1,588	0	150	566	283	YES
	LC2	Min.	843	570	1,588	0	150	411	206	YES
		Max.	1,206	570	1,588	0	150	588	294	YES
Pier 9	LC1*	Min.	804	761	1,625	0	150	392	196	YES
		Max.	1,170	761	1,625	0	150	571	285	YES
	LC2*	Min.	850	577	1,625	0	150	415	207	YES
		Max.	1,217	577	1,625	0	150	594	297	YES
	LC1**	Min.	804	2,386	1,624	150	55	392	196	YES
		Max.	1,170	2,386	1,624	150	55	571	285	YES
	LC2**	Min.	850	2,202	1,624	150	55	415	207	YES
		Max.	1,217	2,202	1,624	150	55	594	297	YES
Pier 13	LC1	Min.	810	771	1,600	0	150	395	198	YES
		Max.	1,179	771	1,600	0	150	575	288	YES
	LC2	Min.	856	584	1,600	0	150	417	209	YES
		Max.	1,226	584	1,600	0	150	598	299	YES
Pier 14	LC1*	Min.	865	872	1,613	0	150	422	211	YES
		Max.	1,259	872	1,613	0	150	614	307	YES
	LC2*	Min.	918	660	1,613	0	150	448	224	YES
		Max.	1,312	660	1,613	0	150	640	320	YES
	LC1**	Min.	865	2,484	2,003	150	68	422	211	YES
		Max.	1,259	2,484	2,003	150	68	614	307	YES
	LC2**	Min.	918	2,273	2,003	150	68	448	224	YES
		Max.	1,312	2,273	2,003	150	68	640	320	YES

VARIABLES/INPUT PARAMETERS:

Existing Spread Footing Dimensions: see historic bridge drawings sheet 36 of 148

Footing Width, B_x = 13 ft (transverse to bridge alignment)

Footing Length, L_y = 10 ft (parallel to bridge alignment)

Interface friction between concrete and rock = 26.0 deg.

tan (δ) = 0.49

Notes:

[1] - EE II Limit State loads provided by Figg on 6 June 2014 and are based on one continuous Multi-T beam superstructure, bridge bearing friction coefficient equal to 0.10 and unmodified HL-93 live load.

[2] - LC1 and LC2 refer to one and two lanes of live load, respectively.

[3] - Refer to AASHTO LRFD Section 10.6.1.3 - Effective Footing Dimensions.

Prepared By:	BCS	6/27/2014
Checked By:	JLL	8/15/2014
Reviewed By:	WAC	9/4/2014

STRENGTH LIMIT STATE SPREAD FOOTING BEARING RESISTANCE EVALUATION (NEXT SUPERSTRUCTURE ALTERNATIVE)

ROUTE 1 VIADUCT BRIDGE REHABILITATION

MAINEDOT WIN 19273.00

BATH, MAINE

Substructure Location	Strength I Limit State Loads[1]					Effective Footing Dimensions[3]				Applied Bearing Pressure (ksf)	Exceed Limiting Value (Yes/No)
						Effective Footing Width		Effective Footing Length			
	Load Combination[2]	Max. or Min.	P_u (kip)	M_{ut} (kip-ft)	M_{ul} (kip-ft)	Eccentricity (e_x, ft)	Width (B', ft)	Eccentricity (e_y, ft)	Length (L', ft)		
Pier 1	LC1	Min.	978	2,828	1,183	2.9	7.2	1.2	7.6	17.9	NO
		Max.	1,335	2,828	1,183	2.1	8.8	0.9	8.2	18.5	NO
	LC2	Min.	1,150	2,143	1,183	1.9	9.3	1.0	7.9	15.6	NO
		Max.	1,506	2,143	1,183	1.4	10.2	0.8	8.4	17.6	NO
Pier 2	LC1	Min.	964	2,493	1,390	2.6	7.8	1.4	7.1	17.3	NO
		Max.	1,327	2,493	1,390	1.9	9.2	1.0	7.9	18.2	NO
	LC2	Min.	1,115	1,889	1,390	1.7	9.6	1.2	7.5	15.5	NO
		Max.	1,478	1,889	1,390	1.3	10.4	0.9	8.1	17.4	NO
Pier 3	LC1	Min.	981	2,544	1,546	2.6	7.8	1.6	6.8	18.3	NO
		Max.	1,350	2,544	1,546	1.9	9.2	1.1	7.7	19.0	NO
	LC2	Min.	1,135	1,927	1,546	1.7	9.6	1.4	7.3	16.2	NO
		Max.	1,504	1,927	1,546	1.3	10.4	1.0	7.9	18.1	NO
Pier 4	LC1	Min.	982	2,514	1,673	2.6	7.9	1.7	6.6	18.9	NO
		Max.	1,352	2,514	1,673	1.9	9.3	1.2	7.5	19.4	NO
	LC2	Min.	1,134	1,904	1,673	1.7	9.6	1.5	7.0	16.7	NO
		Max.	1,504	1,904	1,673	1.3	10.5	1.1	7.8	18.5	NO
Pier 8	LC1	Min.	1,034	2,635	1,702	2.5	7.9	1.6	6.7	19.5	NO
		Max.	1,424	2,635	1,702	1.9	9.3	1.2	7.6	20.1	YES
	LC2	Min.	1,194	1,996	1,702	1.7	9.7	1.4	7.1	17.3	NO
		Max.	1,584	1,996	1,702	1.3	10.5	1.1	7.9	19.3	NO
Pier 9	LC1	Min.	1,044	2,665	1,696	2.6	7.9	1.6	6.8	19.6	NO
		Max.	1,436	2,665	1,696	1.9	9.3	1.2	7.6	20.2	YES
	LC2	Min.	1,205	2,019	1,696	1.7	9.6	1.4	7.2	17.4	NO
		Max.	1,598	2,019	1,696	1.3	10.5	1.1	7.9	19.4	NO
Pier 13	LC1	Min.	1,053	2,700	2,006	2.6	7.9	1.9	6.2	21.6	YES
		Max.	1,449	2,700	2,006	1.9	9.3	1.4	7.2	21.6	YES
	LC2	Min.	1,216	2,046	2,006	1.7	9.6	1.6	6.7	18.8	NO
		Max.	1,612	2,046	2,006	1.3	10.5	1.2	7.5	20.5	YES
Pier 14	LC1	Min.	1,136	3,051	2,242	2.7	7.6	2.0	6.1	24.6	YES
		Max.	1,560	3,051	2,242	2.0	9.1	1.4	7.1	24.1	YES
	LC2	Min.	1,321	2,311	2,242	1.7	9.5	1.7	6.6	21.0	YES
		Max.	1,745	2,311	2,242	1.3	10.4	1.3	7.4	22.7	YES

Notes:

[1] - SER I Limit State loads provided by Figg on 6 June 2014 and are based on one continuous Multi-T beam superstructure, bridge bearing friction coefficient equal to 0.10 and unmodified HL-93 live load.

[2] - LC1 and LC2 refer to one and two lanes of live load, respectively.

[3] - Refer to AASHTO LRFD Section 10.6.1.3 - Effective Footing Dimensions.

VARIABLES/INPUT PARAMETERS:

Existing Spread Footing Dimensions: see historic bridge drawings sheet 36 of 148

Footing Width, B_x = 13 ft (transverse to bridge alignment)

Footing Length, L_y = 10 ft (parallel to bridge alignment)

Factored STR Limit State Bearing Resistance = 20.0 ksf (see PGDR)

Prepared By:	BCS	6/27/2014
Checked By:	JLL	8/15/2014
Reviewed By:	WAC	9/4/2014

STRENGTH LIMIT STATE SPREAD FOOTING ECCENTRICITY EVALUATION (NEXT SUPERSTRUCTURE ALTERNATIVE)

ROUTE 1 VIADUCT BRIDGE REHABILITATION

MAINEDOT WIN 19273.00

BATH, MAINE

REF.: AASHTO LRFD SECTION 10.6.3.3 - ECCENTRIC LOAD LIMITATIONS

Substructure Location	Strength I Limit State Loads[1]					Eccentricity[3] (e_x, ft)	Exceed Limiting Value[4]	Eccentricity[3] (e_y, ft)	Exceed Limiting Value[4]
	Load Combination[2]	Max. or Min.	P_u (kip)	M_{ut} (kip-ft)	M_{ul} (kip-ft)				
Pier 1	LC1	Min.	978	2,828	1,183	2.89	NO	1.21	NO
		Max.	1,335	2,828	1,183	2.12	NO	0.89	NO
	LC2	Min.	1,150	2,143	1,183	1.86	NO	1.03	NO
		Max.	1,506	2,143	1,183	1.42	NO	0.79	NO
Pier 2	LC1	Min.	964	2,493	1,390	2.59	NO	1.44	NO
		Max.	1,327	2,493	1,390	1.88	NO	1.05	NO
	LC2	Min.	1,115	1,889	1,390	1.69	NO	1.25	NO
		Max.	1,478	1,889	1,390	1.28	NO	0.94	NO
Pier 3	LC1	Min.	981	2,544	1,546	2.59	NO	1.58	NO
		Max.	1,350	2,544	1,546	1.88	NO	1.15	NO
	LC2	Min.	1,135	1,927	1,546	1.70	NO	1.36	NO
		Max.	1,504	1,927	1,546	1.28	NO	1.03	NO
Pier 4	LC1	Min.	982	2,514	1,673	2.56	NO	1.70	NO
		Max.	1,352	2,514	1,673	1.86	NO	1.24	NO
	LC2	Min.	1,134	1,904	1,673	1.68	NO	1.48	NO
		Max.	1,504	1,904	1,673	1.27	NO	1.11	NO
Pier 8	LC1	Min.	1,034	2,635	1,702	2.55	NO	1.65	NO
		Max.	1,424	2,635	1,702	1.85	NO	1.20	NO
	LC2	Min.	1,194	1,996	1,702	1.67	NO	1.43	NO
		Max.	1,584	1,996	1,702	1.26	NO	1.07	NO
Pier 9	LC1	Min.	1,044	2,665	1,696	2.55	NO	1.62	NO
		Max.	1,436	2,665	1,696	1.86	NO	1.18	NO
	LC2	Min.	1,205	2,019	1,696	1.68	NO	1.41	NO
		Max.	1,598	2,019	1,696	1.26	NO	1.06	NO
Pier 13	LC1	Min.	1,053	2,700	2,006	2.56	NO	1.91	NO
		Max.	1,449	2,700	2,006	1.86	NO	1.38	NO
	LC2	Min.	1,216	2,046	2,006	1.68	NO	1.65	NO
		Max.	1,612	2,046	2,006	1.27	NO	1.24	NO
Pier 14	LC1	Min.	1,136	3,051	2,242	2.69	NO	1.97	NO
		Max.	1,560	3,051	2,242	1.96	NO	1.44	NO
	LC2	Min.	1,321	2,311	2,242	1.75	NO	1.70	NO
		Max.	1,745	2,311	2,242	1.32	NO	1.28	NO

VARIABLES/INPUT PARAMETERS:

Existing Spread Footing Dimensions: see historic bridge drawings sheet 36 of 148

Footing Width, B_x = 13 ft (transverse to bridge alignment)

Footing Length, L_y = 10 ft (parallel to bridge alignment)

Calculation of Limiting Eccentricity[4]: refer to AASHTO LRFD Section 10.6.3.3 - Eccentric Load Limitations

Limiting Eccentricty, $e_{x(limit)}$ = 5.85

Limiting Eccentricty, $e_{y(limit)}$ = 4.50

Notes:

[1] - SER I Limit State loads provided by Figg on 6 June 2014 and are based on one continuous Multi-T beam superstructure, bridge bearing friction coefficient equal to 0.10 and unmodified HL-93 live load.

[2] - LC1 and LC2 refer to one and two lanes of live load, respectively.

[3] - Refer to AASHTO LRFD Section 10.6.1.3 - Effective Footing Dimensions.

[4] - Eccentricity is limited to 0.45 of the corresponding footing dimensions B or L, for footings on rock.

Prepared By:	BCS	6/27/2014
Checked By:	JLL	8/15/2014
Reviewed By:	WAC	9/4/2014

STRENGTH LIMIT STATE SPREAD FOOTING SLIDING RESISTANCE EVALUATION (NEXT SUPERSTRUCTURE ALTERNATIVE)

ROUTE 1 VIADUCT BRIDGE REHABILITATION

MAINEDOT WIN 19273.00

BATH, MAINE

REF.: AASHTO LRFD SECTION 10.6.3.4 - FAILURE BY SLIDING

Substructure Location	Strength I Limit State Loads[1]						Factored Sliding Resistance = P_u x tan(δ) (kips)	Is Factored Resistance > V_{uy}?
	Load Combination[2]	Max. or Min.	P_u (kip)	M_{ut} (kip-ft)	M_{ul} (kip-ft)	V_{uy} (kip)		
Pier 1	LC1	Min.	978	2,828	1,183	65	239	YES
		Max.	1,335	2,828	1,183	65	326	YES
	LC2	Min.	1,150	2,143	1,183	65	280	YES
		Max.	1,506	2,143	1,183	65	367	YES
Pier 2	LC1	Min.	964	2,493	1,390	65	235	YES
		Max.	1,327	2,493	1,390	65	324	YES
	LC2	Min.	1,115	1,889	1,390	65	272	YES
		Max.	1,478	1,889	1,390	65	360	YES
Pier 3	LC1	Min.	981	2,544	1,546	66	239	YES
		Max.	1,350	2,544	1,546	66	329	YES
	LC2	Min.	1,135	1,927	1,546	66	277	YES
		Max.	1,504	1,927	1,546	66	367	YES
Pier 4	LC1	Min.	982	2,514	1,673	65	239	YES
		Max.	1,352	2,514	1,673	65	330	YES
	LC2	Min.	1,134	1,904	1,673	65	277	YES
		Max.	1,504	1,904	1,673	65	367	YES
Pier 8	LC1	Min.	1,034	2,635	1,702	58	252	YES
		Max.	1,424	2,635	1,702	58	347	YES
	LC2	Min.	1,194	1,996	1,702	58	291	YES
		Max.	1,584	1,996	1,702	58	386	YES
Pier 9	LC1	Min.	1,044	2,665	1,696	57	255	YES
		Max.	1,436	2,665	1,696	57	350	YES
	LC2	Min.	1,205	2,019	1,696	57	294	YES
		Max.	1,598	2,019	1,696	57	390	YES
Pier 13	LC1	Min.	1,053	2,700	2,006	68	257	YES
		Max.	1,449	2,700	2,006	68	353	YES
	LC2	Min.	1,216	2,046	2,006	68	297	YES
		Max.	1,612	2,046	2,006	68	393	YES
Pier 14	LC1	Min.	1,136	3,051	2,242	76	277	YES
		Max.	1,560	3,051	2,242	76	380	YES
	LC2	Min.	1,321	2,311	2,242	76	322	YES
		Max.	1,745	2,311	2,242	76	426	YES

VARIABLES/INPUT PARAMETERS:

Existing Spread Footing Dimensions: see historic bridge drawings sheet 36 of 148

Footing Width, B_x = 13 ft (transverse to bridge alignment)

Footing Length, L_y = 10 ft (parallel to bridge alignment)

Interface friction between concrete and rock = 26.0 deg.

tan (δ) = 0.49

Notes:

[1] - SER I Limit State loads provided by Figg on 6 June 2014 and are based on one continuous Multi-T beam superstructure, bridge bearing friction coefficient equal to 0.10 and unmodified HL-93 live load.

[2] - LC1 and LC2 refer to one and two lanes of live load, respectively.

Prepared By:	BCS	6/27/2014
Checked By:	JLL	8/15/2014
Reviewed By:	WAC	9/4/2014

EXTREME EVENT LIMIT STATE SPREAD FOOTING BEARING RESISTANCE EVALUATION (NEXT SUPERSTRUCTURE ALTERNATIVE)

ROUTE 1 VIADUCT BRIDGE REHABILITATION

MAINEDOT WIN 19273.00

BATH, MAINE

Substructure Location	Extreme II Limit State Loads[1]							Effective Footing Dimensions[3]				Applied Bearing Pressure (ksf)	Exceed Limiting Value (Yes/No)
	Load Combination[2]	Max. or Min.	P_u (kip)	M_{ut} (kip-ft)	M_{ul} (kip-ft)	V_{ux} (kip)	V_{uy} (kip)	Effective Footing Width		Effective Footing Length			
								Eccentricity (e_x, ft)	Width (B', ft)	Eccentricity (e_y, ft)	Length (L', ft)		
Pier 1	LC1	Min.	795	808	1,675	0	150	1.0	11.0	2.1	5.8	12.5	NO
		Max.	1,151	808	1,675	0	150	0.7	11.6	1.5	7.1	14.0	NO
	LC2	Min.	844	612	1,675	0	150	0.7	11.5	2.0	6.0	12.1	NO
		Max.	1,200	612	1,675	0	150	0.5	12.0	1.4	7.2	13.9	NO
Pier 2	LC1	Min.	802	712	1,675	0	150	0.9	11.2	2.1	5.8	12.3	NO
		Max.	1,165	712	1,675	0	150	0.6	11.8	1.4	7.1	13.9	NO
	LC2	Min.	845	540	1,675	0	150	0.6	11.7	2.0	6.0	11.9	NO
		Max.	1,208	540	1,675	0	150	0.4	12.1	1.4	7.2	13.8	NO
Pier 3	LC1	Min.	816	727	1,625	0	150	0.9	11.2	2.0	6.0	12.1	NO
		Max.	1,185	727	1,625	0	150	0.6	11.8	1.4	7.3	13.9	NO
	LC2	Min.	860	551	1,625	0	150	0.6	11.7	1.9	6.2	11.8	NO
		Max.	1,229	551	1,625	0	150	0.4	12.1	1.3	7.4	13.8	NO
Pier 4	LC1	Min.	819	718	1,638	0	150	0.9	11.2	2.0	6.0	12.1	NO
		Max.	1,188	718	1,638	0	150	0.6	11.8	1.4	7.2	13.9	NO
	LC2	Min.	862	544	1,638	0	150	0.6	11.7	1.9	6.2	11.8	NO
		Max.	1,232	544	1,638	0	150	0.4	12.1	1.3	7.3	13.9	NO
Pier 8	LC1	Min.	863	753	1,588	0	150	0.9	11.3	1.8	6.3	12.1	NO
		Max.	1,253	753	1,588	0	150	0.6	11.8	1.3	7.5	14.2	NO
	LC2	Min.	909	570	1,588	0	150	0.6	11.7	1.7	6.5	11.9	NO
		Max.	1,299	570	1,588	0	150	0.4	12.1	1.2	7.6	14.2	NO
Pier 9	LC1*	Min.	871	761	1,625	0	150	0.9	11.3	1.9	6.3	12.3	NO
		Max.	1,263	761	1,625	0	150	0.6	11.8	1.3	7.4	14.4	NO
	LC2*	Min.	917	577	1,625	0	150	0.6	11.7	1.8	6.5	12.1	NO
		Max.	1,309	577	1,625	0	150	0.4	12.1	1.2	7.5	14.4	NO
	LC1**	Min.	871	2,386	1,696	150	57	2.7	7.5	1.9	6.1	19.0	NO
		Max.	1,263	2,386	1,696	150	57	1.9	9.2	1.3	7.3	18.7	NO
	LC2**	Min.	917	2,202	1,696	150	57	2.4	8.2	1.8	6.3	17.8	NO
		Max.	1,309	2,202	1,696	150	57	1.7	9.6	1.3	7.4	18.3	NO
Pier 13	LC1	Min.	877	771	1,600	0	150	0.9	11.2	1.8	6.4	12.3	NO
		Max.	1,273	771	1,600	0	150	0.6	11.8	1.3	7.5	14.4	NO
	LC2	Min.	924	584	1,600	0	150	0.6	11.7	1.7	6.5	12.0	NO
		Max.	1,320	584	1,600	0	150	0.4	12.1	1.2	7.6	14.4	NO
Pier 14	LC1*	Min.	938	872	1,613	0	150	0.9	11.1	1.7	6.6	12.8	NO
		Max.	1,362	872	1,613	0	150	0.6	11.7	1.2	7.6	15.2	NO
	LC2*	Min.	991	660	1,613	0	150	0.7	11.7	1.6	6.7	12.6	NO
		Max.	1,414	660	1,613	0	150	0.5	12.1	1.1	7.7	15.2	NO
	LC1**	Min.	938	2,484	2,242	150	76	2.6	7.7	2.4	5.2	23.3	NO
		Max.	1,362	2,484	2,242	150	76	1.8	9.4	1.6	6.7	21.7	NO
	LC2**	Min.	991	2,273	2,242	150	76	2.3	8.4	2.3	5.5	21.5	NO
		Max.	1,414	2,273	2,242	150	76	1.6	9.8	1.6	6.8	21.2	NO

VARIABLES/INPUT PARAMETERS:

Existing Spread Footing Dimensions: see historic bridge drawings sheet 36 of 148

Footing Width, B_x = 13 ft (transverse to bridge alignment)

Footing Length, L_y = 10 ft (parallel to bridge alignment)

Factored STR Limit State Bearing Resistance = 44.0 ksf (see PGDR)

Notes:

[1] - SER I Limit State loads provided by Figg on 6 June 2014 and are based on one continuous Multi-T beam superstructure, bridge bearing friction coefficient equal to 0.10 and unmodified HL-93 live load.

[2] - LC1 and LC2 refer to one and two lanes of live load, respectively.

[3] - Refer to AASHTO LRFD Section 10.6.1.3 - Effective Footing Dimensions.

Prepared By:	BCS	6/27/2014
Checked By:	JLL	8/15/2014
Reviewed By:	WAC	9/4/2014

EXTREME EVENT LIMIT STATE SPREAD FOOTING ECCENTRICITY EVALUATION (NEXT SUPERSTRUCTURE ALTERNATIVE)

ROUTE 1 VIADUCT BRIDGE REHABILITATION

MAINEDOT WIN 19273.00

BATH, MAINE

REF.: AASHTO LRFD SECTION 10.6.3.3 - ECCENTRIC LOAD LIMITATIONS

Substructure Location	Extreme II Limit State Loads[1]					Eccentricity (e_x, ft)	Exceed Limiting Value	Eccentricity (e_y, ft)	Exceed Limiting Value
	Load Combination[2]	Max. or Min.	P_u (kip)	M_{ut} (kip-ft)	M_{ul} (kip-ft)				
Pier 1	LC1	Min.	795	808	1,675	1.02	NO	2.11	NO
		Max.	1,151	808	1,675	0.70	NO	1.46	NO
	LC2	Min.	844	612	1,675	0.73	NO	1.98	NO
		Max.	1,200	612	1,675	0.51	NO	1.40	NO
Pier 2	LC1	Min.	802	712	1,675	0.89	NO	2.09	NO
		Max.	1,165	712	1,675	0.61	NO	1.44	NO
	LC2	Min.	845	540	1,675	0.64	NO	1.98	NO
		Max.	1,208	540	1,675	0.45	NO	1.39	NO
Pier 3	LC1	Min.	816	727	1,625	0.89	NO	1.99	NO
		Max.	1,185	727	1,625	0.61	NO	1.37	NO
	LC2	Min.	860	551	1,625	0.64	NO	1.89	NO
		Max.	1,229	551	1,625	0.45	NO	1.32	NO
Pier 4	LC1	Min.	819	718	1,638	0.88	NO	2.00	NO
		Max.	1,188	718	1,638	0.60	NO	1.38	NO
	LC2	Min.	862	544	1,638	0.63	NO	1.90	NO
		Max.	1,232	544	1,638	0.44	NO	1.33	NO
Pier 8	LC1	Min.	863	753	1,588	0.87	NO	1.84	NO
		Max.	1,253	753	1,588	0.60	NO	1.27	NO
	LC2	Min.	909	570	1,588	0.63	NO	1.75	NO
		Max.	1,299	570	1,588	0.44	NO	1.22	NO
Pier 9	LC1*	Min.	871	761	1,625	0.87	NO	1.87	NO
		Max.	1,263	761	1,625	0.60	NO	1.29	NO
	LC2*	Min.	917	577	1,625	0.63	NO	1.77	NO
		Max.	1,309	577	1,625	0.44	NO	1.24	NO
	LC1**	Min.	871	2,386	1,696	2.74	NO	1.95	NO
		Max.	1,263	2,386	1,696	1.89	NO	1.34	NO
	LC2**	Min.	917	2,202	1,696	2.40	NO	1.85	NO
		Max.	1,309	2,202	1,696	1.68	NO	1.30	NO
Pier 13	LC1	Min.	877	771	1,600	0.88	NO	1.82	NO
		Max.	1,273	771	1,600	0.61	NO	1.26	NO
	LC2	Min.	924	584	1,600	0.63	NO	1.73	NO
		Max.	1,320	584	1,600	0.44	NO	1.21	NO
Pier 14	LC1*	Min.	938	872	1,613	0.93	NO	1.72	NO
		Max.	1,362	872	1,613	0.64	NO	1.18	NO
	LC2*	Min.	991	660	1,613	0.67	NO	1.63	NO
		Max.	1,414	660	1,613	0.47	NO	1.14	NO
	LC1**	Min.	938	2,484	2,242	2.65	NO	2.39	NO
		Max.	1,362	2,484	2,242	1.82	NO	1.65	NO
	LC2**	Min.	991	2,273	2,242	2.29	NO	2.26	NO
		Max.	1,414	2,273	2,242	1.61	NO	1.59	NO

VARIABLES/INPUT PARAMETERS:

Existing Spread Footing Dimensions: see historic bridge drawings sheet 36 of 148

Footing Width, B_x = 13 ft (transverse to bridge alignment)

Footing Length, L_y = 10 ft (parallel to bridge alignment)

Calculation of Limiting Eccentricity[4]: refer to AASHTO LRFD Section 10.6.3.3 - Eccentric Load Limitations

Limiting Eccentricty, $e_{x(limit)}$ = 5.85

Limiting Eccentricty, $e_{y(limit)}$ = 4.50

Notes:

[1] - SER I Limit State loads provided by Figg on 6 June 2014 and are based on one continuous Multi-T beam superstructure, bridge bearing friction coefficient equal to 0.10 and unmodified HL-93 live load.

[2] - LC1 and LC2 refer to one and two lanes of live load, respectively.

[3] - Refer to AASHTO LRFD Section 10.6.1.3 - Effective Footing Dimensions.

[4] - Eccentricity is limited to 0.45 of the corresponding footing dimensions B or L, for footings on rock.

Prepared By:	BCS	6/27/2014
Checked By:	JLL	8/15/2014
Reviewed By:	WAC	9/4/2014

EXTREME EVENT LIMIT STATE SPREAD FOOTING SLIDING RESISTANCE EVALUATION (NEXT SUPERSTRUCTURE ALTERNATIVE)

ROUTE 1 VIADUCT BRIDGE REHABILITATION

MAINEDOT WIN 19273.00

BATH, MAINE

REF.: AASHTO LRFD SECTION 10.6.3.4 - FAILURE BY SLIDING

Substructure Location	Extreme II Limit State Loads[1]							Factored Sliding Resistance = P_u x tan(δ) (kips)	Is Factored Resistance > V_{uy} or V_{ux}?
	Load Combination[2]	Max. or Min.	P_u (kip)	M_{ut} (kip-ft)	M_{ul} (kip-ft)	V_{ux} (kip)	V_{uy} (kip)		
Pier 1	LC1	Min.	795	808	1,675	0	150	194	YES
		Max.	1,151	808	1,675	0	150	281	YES
	LC2	Min.	844	612	1,675	0	150	206	YES
		Max.	1,200	612	1,675	0	150	293	YES
Pier 2	LC1	Min.	802	712	1,675	0	150	196	YES
		Max.	1,165	712	1,675	0	150	284	YES
	LC2	Min.	845	540	1,675	0	150	206	YES
		Max.	1,208	540	1,675	0	150	295	YES
Pier 3	LC1	Min.	816	727	1,625	0	150	199	YES
		Max.	1,185	727	1,625	0	150	289	YES
	LC2	Min.	860	551	1,625	0	150	210	YES
		Max.	1,229	551	1,625	0	150	300	YES
Pier 4	LC1	Min.	819	718	1,638	0	150	200	YES
		Max.	1,188	718	1,638	0	150	290	YES
	LC2	Min.	862	544	1,638	0	150	210	YES
		Max.	1,232	544	1,638	0	150	300	YES
Pier 8	LC1	Min.	863	753	1,588	0	150	210	YES
		Max.	1,253	753	1,588	0	150	306	YES
	LC2	Min.	909	570	1,588	0	150	222	YES
		Max.	1,299	570	1,588	0	150	317	YES
Pier 9	LC1*	Min.	871	761	1,625	0	150	212	YES
		Max.	1,263	761	1,625	0	150	308	YES
	LC2*	Min.	917	577	1,625	0	150	224	YES
		Max.	1,309	577	1,625	0	150	319	YES
	LC1**	Min.	871	2,386	1,696	150	57	212	YES
		Max.	1,263	2,386	1,696	150	57	308	YES
	LC2**	Min.	917	2,202	1,696	150	57	224	YES
		Max.	1,309	2,202	1,696	150	57	319	YES
Pier 13	LC1	Min.	877	771	1,600	0	150	214	YES
		Max.	1,273	771	1,600	0	150	310	YES
	LC2	Min.	924	584	1,600	0	150	225	YES
		Max.	1,320	584	1,600	0	150	322	YES
Pier 14	LC1*	Min.	938	872	1,613	0	150	229	YES
		Max.	1,362	872	1,613	0	150	332	YES
	LC2*	Min.	991	660	1,613	0	150	242	YES
		Max.	1,414	660	1,613	0	150	345	YES
	LC1**	Min.	938	2,484	2,242	150	76	229	YES
		Max.	1,362	2,484	2,242	150	76	332	YES
	LC2**	Min.	991	2,273	2,242	150	76	242	YES
		Max.	1,414	2,273	2,242	150	76	345	YES

VARIABLES/INPUT PARAMETERS:

Existing Spread Footing Dimensions: see historic bridge drawings sheet 36 of 148

Footing Width, B_x = 13 ft (transverse to bridge alignment)

Footing Length, L_y = 10 ft (parallel to bridge alignment)

Interface friction between concrete and rock = 26.0 deg.

tan (δ) = 0.49

Notes:

[1] - SER I Limit State loads provided by Figg on 6 June 2014 and are based on one continuous Multi-T beam superstructure, bridge bearing friction coefficient equal to 0.10 and unmodified HL-93 live load.

[2] - LC1 and LC2 refer to one and two lanes of live load, respectively.

[3] - Refer to AASHTO LRFD Section 10.6.1.3 - Effective Footing Dimensions.

Prepared By:	BCS	6/27/2014
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Reviewed By:	WAC	9/4/2014