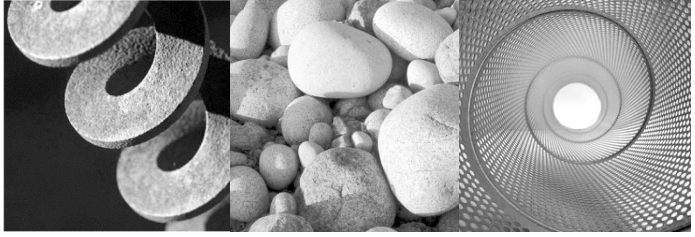

GEI Consultants

Consulting
Engineers and
Scientists

Geotechnical Report

Bridge No. 03240

Route 69 over Mad River
Wolcott, Connecticut

Submitted to:
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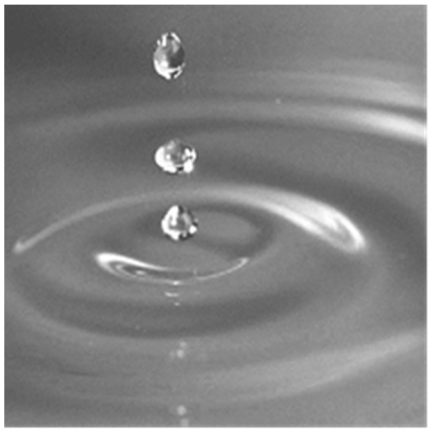
April 2, 2020
Project No. 125810



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1. Introduction

1.1 Project Summary

The project consists of replacement of Bridge No. 03240, which carries Route 69 (Wolcott Road) over the Mad River in Wolcott, Connecticut. The project location and current structure are shown on Figures 1 and 2.

GEI Consultants, Inc. (GEI) was retained by CME Associates, Inc., to perform a subsurface exploration program and prepare this Geotechnical Report in support of the design efforts. This report presents the results of the subsurface explorations, our evaluation of the existing subsurface conditions, and our geotechnical recommendations for design and construction.

1.2 Scope of Services

GEI's scope of work for this project included the following:

1. Reviewed available published geologic data, existing plans, and proposed bridge design information provided to us.
2. Developed a subsurface exploration program consisting of one boring near each abutment (2 total) and three augered probes (6 total) at varying distances behind each abutment.
3. Provided full-time observation of the test borings and augered probes and classified recovered samples in general accordance with ConnDOT Geotechnical Engineering Manual.
4. Assigned and coordinated laboratory testing on samples collected from the test borings.
5. Reviewed the results of the geotechnical explorations, prepared a subsurface profile, and developed soil properties for analyses.
6. Prepared foundation recommendations for the proposed bridge.
7. Presented the results of the explorations, our analyses, and our recommendations in this report.

1.3 Authorization

Our work was performed in accordance with the terms and conditions of our Sub-consultant Agreement with CME and our Notice to Proceed (NTP) effective August 1, 2017.

1.4 Datum

Ground surface elevations at the boring locations have been estimated from a topographic survey provided by CME in NAVD 88 datum. Coordinates, stations, and offsets from the design plans have been transcribed to the boring logs.

2. Site and Project Description

2.1 Site Description

Bridge No. 03240 is a single-span, 24-foot long concrete slab structure originally constructed in 1900. The bridge has a skew angle of approximately 30 degrees and the clear span (hydraulic opening) is approximately 15.25 feet wide. The bridge carries two lanes of traffic along Route 69 (Wolcott Road) over the Mad River, in the Town of Wolcott, Connecticut, as shown in Figure 1.

The original stone masonry abutments, constructed around 1900, remain under the western portion of the bridge. No record drawings are available for these substructures; therefore, the dimensions and bearing depths are unknown at present. The bridge was widened, as part of State Project No. 166-13, from 24.0 feet to 35.7 feet wide in 1935. Reinforced concrete abutments were built as part of this work, connecting to the masonry abutments and continuing to the east beneath the deck. These portions of the abutments are approximately 15 to 16 feet in height and appear to be founded on rock, based on the results of this investigation. Stone masonry and reinforced concrete wingwalls are flared out from each corner of the bridge.

2.2 Project Description

GEI was provided a copy of the 60 percent design plans dated January 2020 prepared by RHS Consulting Design, LLC. We understand the bridge replacement will occur within a 2-month (approx.) full detour period.

The project will involve full replacement of the existing bridge with a longer (37’-7” feet span) and wider (38’-10” feet out-to-out) structure supported on new reinforced concrete abutments. The vertical profile will be raised slightly (approximately 1.5 feet) while the horizontal alignment will remain the same. Short U-type wingwalls will be constructed on three sides of the bridge. A longer curved wingwall will be constructed off the southeast corner, abutting Hillside Drive.

There are several underground utilities that will be impacted by the project. These considerations are discussed in further detail in Section 6.2.

3. Exploration Procedures

3.1 Test Borings

New England Boring Contractors, Inc., under subcontract to CME, drilled two borings (B-1, B-2) in the northbound lane of the bridge, and advanced seven auger probes (A-1 through A-7) in the southbound lane of the bridge, between September 19 and September 21, 2017. A GEI representative was on-site full time to observe the drilling procedures and classify the soil samples.

Each boring was advanced using solid stem augers (SSA) to depths of 10 to 12 feet, then drive and wash techniques to refusal at the top of bedrock. Rock core samples, approximately 10 feet in length, were obtained from each boring.

Standard Penetration tests and split-spoon sampling in general accordance with ASTM D1586 were taken continuously in the upper 12 to 14 feet of the borings. The truck-mounted drilling rig used was equipped with a 140-lb safety hammer. The depth to groundwater was measured in each boring prior to backfilling with drill cuttings. After each boring was completed, the holes were backfilled with drill cuttings, supplemented with ¾" stone, and patched at the surface using cold patch asphalt.

Each auger probe was advanced to refusal at shallow depths (up to 10 feet below grade) or into inferred native soils or weathered rock below the assumed bearing depth of existing abutments (greater than about 12 feet below grade). No soil or rock samples were obtained from the auger probes.

Approximate boring and auger probe locations relative to existing conditions are shown in Figure 2. Soil test boring logs are attached in Appendix A.

3.2 Laboratory Testing

Laboratory testing was performed on selected samples collected during the exploration program. Testing was performed by GeoTesting Express, under subcontract to CME. Grain size analyses and moisture contents were performed to aid in soil classification and estimation of engineering properties.

Laboratory testing results are presented in Appendix B.

4. Subsurface Conditions

4.1 Geologic Setting

The bridge site lies within the narrow, primarily north-south Mad River valley, cutting through glacial till uplands immediately to the east and west. Post-glacial alluvial deposits are likely present within the valley and areas immediately adjacent.

Bedrock geology is mapped as part of the Taine Mountain Formation, described as gray, medium-grained, well-laminated gneissic granofels.

4.2 Subsurface Conditions

Based on our review of the available geotechnical information, the general subsurface strata are as follows, beginning at the ground surface. The subsurface conditions are known only at the exploration locations. Borings B-1 and B-2 were advanced in the northbound lane, approximately 8 to 9 feet behind the front face of the existing bridge abutments. Conditions between boring locations may differ significantly from those described below.

Roadway Section – The borings encountered 6 to 7 inches of asphalt pavement over 10 to 18 inches of processed stone base.

I. Sand and Gravel Fill – Fill was encountered in both borings on the west side of the bridge to a depth of 10 to 11 feet below grade. The fill was comprised primarily of medium- to coarse-grained sand with gravel (up to 1-¼-inch), although some oversize material was encountered.

In boring B-1, difficult drilling was noted within a 10-inch interval starting at 3 feet, likely signifying a cobble or other small obstruction within the fill. Auger Probes A-2 and A-3, advanced behind the old masonry abutments beneath the east side of the bridge, also encountered obstructions at approximately 4 feet below grade. These obstructions imply that cobbles, possibly concrete, and other debris may be present within the bridge backfill, particularly at shallow depths.

Uncorrected SPT N-values in the fill varied from 4 to 28 blows per foot (bpf), indicating very loose- to medium-dense conditions.

II. Alluvial Organic Soils – A thin layer of river-laid organic soils was encountered below the fill in boring B-1, directly behind the north abutment. Between depths of about 11 feet

and 12.5 feet, black and brown sand interbedded with organic silts and fibrous organics were encountered. These soils are very loose in place.

There is a severe bend in the river just to the west of the bridge site. Also, from review of the surrounding topography, it appears as if the roadway embankment partially constricts the natural channel, particularly on the north side. Based on our experience, we would expect these organic soils to be present extending north within the approximate limits of the former channel, estimated to be about 20 feet back from the existing abutment face.

III. Sand and Gravel – Sand and gravel deposits were encountered below the fill in boring B-2, and below the alluvial organic soils in boring B-1, extending to bedrock below. Recovered samples were generally classified as reddish brown widely graded sand to gravel with a trace of silty fines. Uncorrected SPT N-values ranged from 14 to 61 bpf, increasing with depth, indicating medium-dense to very dense conditions.

Bedrock – Bedrock of the Taine Mountain formation was encountered in boring B-1 at 16.5 feet below grade and in boring B-2 at 15.0 feet below grade. In boring B-1, the roller-bit was advanced from 16.5 feet to 20.0 feet through presumed moderately weathered rock before obtaining a core sample. Recovered core samples were generally described as gray to dark gray, medium to coarse-grained, massively bedded gneissic granofels. Weathering is generally slight, with moderately weathered seams, particularly near the top of the stratum. Joints were generally oriented at less than a 20-degree dip from horizontal.

Table 1 - Summary of Bedrock Conditions

Boring ID	Depth to Top (ft)	Elevation (ft)	RQD (%)	Apparent Weathering	Classification
B-1	20	574.5	67	Slightly Weathered	GNEISSIC GRANOFELS
	25	569.5	91		
B-2	15	578.5	60	Slightly to Moderately Weathered	GNEISSIC GRANOFELS
	20	573.5	88		

4.3 Groundwater Conditions

Water levels were measured in the boreholes prior to starting the drive and wash drilling. Water levels ranged from about 9.8 feet to 11.3 feet below roadway grade.

Groundwater level measurements represent conditions at the times and locations indicated. Significantly different groundwater levels may occur at other times and locations.

4.4 Auger Probe Results

Auger probes A-1 through A-7 were advanced in the southbound lane of the bridge, at the noted approximate distances from the front face of the existing stone masonry abutments. The southbound lane of the bridge includes the original portion of the Bridge No. 03240 structure, constructed in 1900. The dimensions and layout of the newer portions of the bridge are shown on the 1935 repair plans; however, the dimensions and layout of the original 1900 stone masonry abutments are not. The purpose of the auger probes was to establish the overall abutment geometry, heading back from the face.

Table 2 – Auger Probe Results

Probe No.	Location	Approx. Dist. behind Abutment Front Face (ft)	Termination Depth (ft)	Inferred Termination Material
A-1	South	2.5	2.5	Abutment
A-2	South	5.0	4.0	Abutment or Obstruction
A-3	South	8.0	4.0	Abutment or Obstruction
A-4	South	11.0	12.0	Dense native soil/weathered rock
A-5	North	2.5	2.8	Abutment
A-6	North	5.0	13.0	Dense native soil/weathered rock
A-7	North	8.0	14.0	Dense native soil/weathered rock

<u>North Abutment</u>
Probe A-5 encountered what was likely the back portion of abutment at 2.5 feet behind the face. Probe A-6, at a distance of 5 feet behind the north (masonry) abutment face, did not encounter the abutment, advancing into dense native soil or weathered rock at a depth of 13 feet below roadway grade. Probe A-7, further behind the face, encountered similar results to A-6. Based on these results, the masonry abutment on the north side is likely between 3.0 and 4.5 feet in thickness, as measured from the face.

<u>South Abutment</u>
Similar to the North abutment, the first probe on the south side (A-1) encountered the back of abutment at about 2.5 feet below grade. The next two probes (A-2 and A-3) encountered shallow refusal at a depth of about 4 feet, extending back to at least 8 feet behind the face. It is unclear what obstruction(s) caused this refusal, possibly an old approach slab, boulder-laden backfill, debris, etc. Although possible, we do not believe the abutment itself extends

back this far. It is more likely that the south masonry abutment is of similar size and shape to the north one.

4.5 In-place Soil Properties

Recommended in-place soil properties for the site are presented below. We estimated these values based on published correlations to SPT N-values and visual soil descriptions.

Table 3 – In-Place Soil Properties

STRATUM	Angle of Internal Friction ($\phi°$)	Cohesion (c) (psf)	Moist (Total) Unit Weight (γ_T) (lb/ft^3)	Active Earth Pressure Coeff. (K_a)	Passive Earth Pressure Coeff. (K_p)
I. Existing Fill	32	0	125	0.31	3.25
II. Alluvial Organic Soils	28	0	115	0.36	2.77
III. Sand and Gravel	36	0	125	0.26	3.85

5. Design Recommendations

Recommendations presented herein are based on the 30 percent plans provided by CME. The design criteria presented herein should be reviewed by GEI for continued applicability if, and when, revisions are made by the design team concerning configuration, design loads, etc.

5.1 Code Reference

Our services were performed in general conformance with the ConnDOT Geotechnical Engineering Manual and our approved scope dated November 21, 2016. Project design parameters and computations generally follow those described in the relevant sections of the AASHTO LRFD Bridge Design Specifications (8th Ed., 2017, with interims), supplemented by the most recent editions of the ConnDOT Bridge Design Manual and Geotechnical Engineering Manual.

5.2 Foundation Support

The new abutments will require deep foundations extended into rock to provide sufficient vertical and lateral resistance. Micropiles are well-suited for this project, primarily due to schedule concerns within the relatively short detour window, as they can completed in a single installation. Design and construction recommendations for micropiles are provided below.

Proposed wingwalls can be supported on conventional shallow foundations bearing on subgrades prepared in accordance with Section 6.3. We understand that, as controlled by scour concerns, the base of the abutments and wingwalls are to be at an elevation of approximately 584.5 ft. Per Section 6.3, we recommend installation of a 12-inch (min.) working platform of 3/8-inch Crushed Stone beneath wingwalls and abutments to improve constructability and to provide sufficient frost depth resistance.

5.3 Micropile Design

To provide adequate support for project loadings, micropiles should be extended into the bedrock underlying the site.

Micropiles should be designed by a Connecticut-registered professional engineer retained by the contractor. The piles should be designed in accordance with the structural requirements in Article 10.9.3.10 of the AASHTO LRFD Bridge Design Specifications. We recommend that permanent steel casing be seated a minimum of six inches into rock. The bending capacity at the joints is significantly less than the capacity of the intact casing, so we also recommend that no casing joints be present within 10 feet of the bottom of the pile cap.

The contractor's design should be checked with one (1) pre-production verification pile load test on a sacrificial pile and one (1) proof test at each abutment. The load test may be conducted at an off-line location, a maximum of 15 feet from the nearest production micropile, to allow for it to be conducted and results analyzed before the shutdown period. Off the east corner of the south abutment near the terminus of Hillside Drive appears to be one suitable location. The bond zone may be designed based on a resistance factor of 0.70 (Strength Limit) per Table 10.5.5.2.5-1 of the AASHTO Specifications. A resistance factor of 1.0 should be used for the Extreme Event limit state.

Micropile spacing should be at least three times the pile diameter or 2.5 ft, whichever is greater, to limit group interaction effects. The piles should extend at least 18 inches into the pile cap.

For preliminary design and costing, we have estimated the capacity of two common rock-socketed micropile sizes, as shown below.

Table 4 – Estimated Micropile Geotechnical Resistance

Pile Type & Size	Nominal Axial Resistance (kips)	Factored Axial Resistance (Strength) (kips)
9.625-inch O.D. casing, 0.50-inch wall; 8.5-inch dia., 10-ft rock socket	334	233
10.75-inch O.D. casing, 0.50-inch wall; 9.5-inch dia., 10-ft rock socket	373	261

The values given in Table 4 assume a nominal bond strength in rock of 15 ksf, ignoring any contribution from the cased soil. The final pile design by the contractor will incorporate the actual diameter of the drilling bit to be used. Our conceptual design assumed 80 ksi (API-N80) permanent casing, a single No. 18 Grade 75 center reinforcing bar, a corrosion allowance of 1/16-inch on the exterior surface of the casing, and a minimum grout strength of 4,000 psi. Structural details and capacity of the micropile section used for construction should be determined by the micropile designer. Micropile capacity was controlled by geotechnical grout-to-rock bond capacity in our preliminary evaluation, using the parameters noted above.

We recommend that the micropiles have a minimum rock socket length of 10 feet to account for variability of rock quality through the socket. This minimum should apply regardless of the bond strength measured during the pile load test.

Vertical profile adjustments across the bridge will be relatively minor. Therefore, induced settlements adjacent to the piles are expected to be negligible, and downdrag does not need to be considered in the micropile design.

Because the micropiles will be achieve their capacity in the bedrock, we estimate that the pile foundations will settle less than one-half inch due to the applied structure loads.

We performed lateral capacity analysis of the micropile foundations using the software LPILE, assuming both free-head and fixed-head conditions within the pile cap. For the free-head condition, lateral capacity was evaluated at displacements of ½-inch and 1-inch. For the fixed-head condition, the expected shear load per pile was input, assuming 10 micropiles per abutment and the provided lateral loads.

Table 5 – Lateral Resistance – 9-5/8" Micropile

Pile Type & Size	Deflection (in.)	Max. Shear (kips)	Max. Moment (kips-in)
Free Head Condition (Deflection Input)			
9.625-inch O.D. casing, 0.50-inch wall; 8.5-inch dia., 10-ft rock socket	0.50	15.0	675
	1.00	24.0	1,325
Fixed Head Condition (Shear Input)			
9.625-inch O.D. casing, 0.50-inch wall; 8.5-inch dia., 10-ft rock socket	0.16	18.4	645

Table 6 – Lateral Resistance – 10-3/4" Micropile

Pile Type & Size	Deflection (in.)	Max. Shear (kips)	Max. Moment (kip-in)
Free Head Condition (Deflection Input)			
10.75-inch O.D. casing, 0.50-inch wall; 9.5-inch dia., 10-ft rock socket	0.50	19.5	1,005
	1.00	31.0	1,880
Fixed Head Condition (Shear Input)			
10.75-inch O.D. casing, 0.50-inch wall; 9.5-inch dia., 10-ft rock socket	0.11	18.4	720

5.4 Wingwalls

U-type wingwalls will be constructed on three sides of the bridge. A longer curved wingwall will be constructed off the southeast corner, abutting Hillside Drive. Including embedment and endblocks, these walls are shown on the current plans as 10 to 18 feet in height.

The organic alluvial soils in place beneath the north side of the bridge are not suitable for support of wingwalls. If these soils are left in place, excess settlements may occur, particularly the wingwalls differentially to the pile-supported abutments.

All wingwall footings should bear on a working platform consisting of at least 12 inches of 3/8-inch Crushed Stone *(Item #0728008A)* underlain by geotextile fabric *(Separation – High Survivability)*. According to the test boring results, up to 18 inches of organic alluvial soils may extend below the proposed working platform base elevation. As discussed in Section 6.3, these soils should be removed where encountered and the Crushed Stone working platform thickened as necessary.

The base of the granular working platform supporting the wingwall footings should be embedded at least 4 feet below finished surrounding grade for frost protection. Wingwalls should be evaluated for bearing at the strength and service limit states according to the attached Figure 5, which provides recommended values for varying footing widths. Note the service limit states correspond to approximately 1 inch of settlement.

5.5 Lateral Earth Pressures

The new abutments and wingwalls should be designed to withstand lateral earth pressures according to the lateral earth coefficients given below. Assuming the abutments and wingwalls will be backfilled per Section 5.6 of the ConnDOT Bridge Design Manual, a unit weight of 125 pcf and an internal friction angle (ϕ) of 35 degrees may be assumed.

Table 7 – Lateral Coefficients for Wingwalls and Abutments

Support Condition	Lateral Coefficient
Active Condition	0.27
At-rest Condition	0.43
Passive Condition	3.69

Earth pressures should be applied as shown on Fig. 3.11.5.3-1 of the AASHTO LRFD manual. Design of abutments and walls should also include a live load surcharge, in accordance with AASHTO 3.11.6.4.

For calculating nominal sliding resistance for footings bearing on 3/8-inch Crushed Stone, we recommend the following:

Table 8 – Sliding Resistance

Footing Type	Coefficient of Friction (Table 3.11.5.3-1)	Resistance Factor (Table 10.5.5.2.2-1)
Cast-in-place Concrete	0.55	0.80

The passive resistance provided by any soils in front of the wingwalls should be ignored.

5.6 Seismic Design Information

The recommended alternative for this bridge is a full replacement. According to the ConnDOT Bridge Design Manual, abutments and wingwalls on new single span bridges do not need to be designed for seismic forces; however, superstructure seat widths and restraint forces shall conform to the requirements of the AASHTO specifications.

In accordance with Article 3.10.3.1 of the AASHTO Bridge Specifications, we recommend that the project be classified as Site Class C.

We recommend the following seismic design criteria, based on a seven percent probability of exceedance in a 75-year period (approximate return period of 1,000 years) per Section 3.10 of the *AASHTO LRFD Bridge Design Specifications*.

<u>Mapped Seismic Hazard (Site Class C)</u>
Peak Ground Acceleration Coefficient (PGA) = 0.064g
Spectral Acceleration Coefficient (period = 0.2 sec) (S_S) = 0.136g
Spectral Acceleration Coefficient (period = 1.0 sec) (S_1) = 0.036g

The resulting parameters for calculation of the design response spectrum are:

Site-corrected Peak Ground Acceleration (A_S) = 0.077g
Short Period Response Acceleration (S_{DS}) = 0.163 g
Long Period Response Acceleration (S_{D1}) = 0.062 g
Seismic Zone: 1 (per Table 3.10.6.1)

As per AASHTO 10.5.4.2, we did not evaluate the potential for liquefaction because the bridge is in Seismic Zone 1.

5.7 Approach Embankments

The project will include widening and vertical profile adjustment, resulting in minor modifications to the current embankment slopes. We do not expect these activities to result in detrimental settlements or slope instability.

Finished slopes adjacent to the river channel and extending up the side of the abutments should be constructed at no steeper than a 2H:1V grade and covered with suitable riprap where required for scour protection.

6. Construction Considerations

6.1 Excavation and Dewatering (TERS)

At a minimum, all excavations should be made in accordance with Occupational Safety and Health Administration (OSHA) standards.

Current plans show the existing abutments and wingwalls to be left in place and cut down where necessary. Excavations to install new abutments and wingwalls will extend approximately 4.5 to 6 feet below the cutoff elevation. We expect these excavations will occur largely above the groundwater table and can be accomplished without the need for dedicated TERS/cofferdams beyond the substructures left in place. To facilitate excavations in the dry, minor items such as sandbags, sumps, and diversion of surface water should be expected for temporary conditions such as removal and replacement of unsuitable organic soils on the north side or after large rainfall events.

Maximum cut slope rates for establishing limits of temporary lateral support (TERS) should be 1.5H:1V.

6.2 Utility Considerations

Utility impacts are expected to be relatively minor under the current replacement scheme, consisting primarily of gas and water mains crossing just north of the bridge before heading south, and a sanitary sewer main to the south of the bridge.

The existing underground gas main and water main will likely need to be protected in-place, or relocated north of the construction limits, due to the proximity of roadway and structure excavation. The sewer main south of the bridge has a manhole on Hillside Drive, where pavement reconstruction is expected. However, this buried main is not anticipated to be impacted by the construction. One utility pole supporting overhead wires is anticipated to be relocated.

6.3 Subgrade Preparation

The organic alluvial soils in place beneath the north side of the bridge, as noted in boring B-1, are not suitable for support of wingwalls. If these soils are left in place, excess settlements may occur, particularly the wingwalls differentially to the pile-supported abutments.

All wingwall footings should bear on a working platform consisting of at least 12 inches of 3/8-inch Crushed Stone *(Item #0728008A)* underlain by geotextile fabric *(Separation – High*

Survivability). According to the test boring results, up to 18 inches of organic alluvial soils may extend below the proposed working platform base elevation on the north side. These soils should be removed where encountered and the Crushed Stone working platform thickened as necessary. A detail should be added to the plans to call attention to expected remedial excavations.

We recommend that the 3/8-inch Crushed Stone working platform (also underlain by the same geotextile fabric) be extended in kind under each abutment, with similar unsuitable material excavations noted beneath the north abutment. This will improve and stabilize the conditions for heavy equipment working within this area.

We recommend that all unsuitable excavations and replacement take place in the dry. This will allow for inspection of the subgrade, feasible placement of the geotextile, and confirmation that unsuitable materials have been removed to a sufficient depth. Temporary control/diversion of surface water and dewatering of subgrades will likely be required to conduct undercutting operations in the dry.

The finished subgrade should be prepared in accordance with ConnDOT specifications. Direct bearing surfaces should be free of standing water, frost, and loose soil. Areas of the subgrade disturbed by traffic or surface water should be re-compacted.

Zones within the foundation soils may be frost susceptible. Therefore, if construction is performed during freezing weather, special precautions will be required to prevent the subgrade soils from freezing. Freezing of the soil beneath the foundation during construction may result in subsequent settlement of the structure. All subgrades should be free of frost. Frost-susceptible subgrade soils that have frozen should be removed and replaced in accordance with ConnDOT standard specifications.

Pervious Structure Backfill should be placed behind abutments and wingwalls to the limits described in Section 5.6 of the ConnDOT Bridge Design Manual and compacted in accordance with the Standard Specifications.

7. Limitations

Our recommendations are based on the project information provided to us at the time of this report and may require modification if there are any changes in the nature, design, or location of the proposed construction. We recommend that GEI be engaged to review the final plans and specifications to judge whether changes in the project affect the validity of our recommendations and whether our recommendations have been properly implemented in the design.

The recommendations in this report are based in part on the data obtained from the borings. The nature and extent of variations between borings may not become evident until construction. If variations from the anticipated conditions are encountered, it may be necessary to revise the recommendations in this report. Therefore, we recommend that GEI be engaged to make site visits during construction to: a) check that the subsurface conditions exposed during construction are in general conformance with our design assumptions and b) ascertain that, in general, the geotechnical aspects of the work are being performed in compliance with the contract documents.

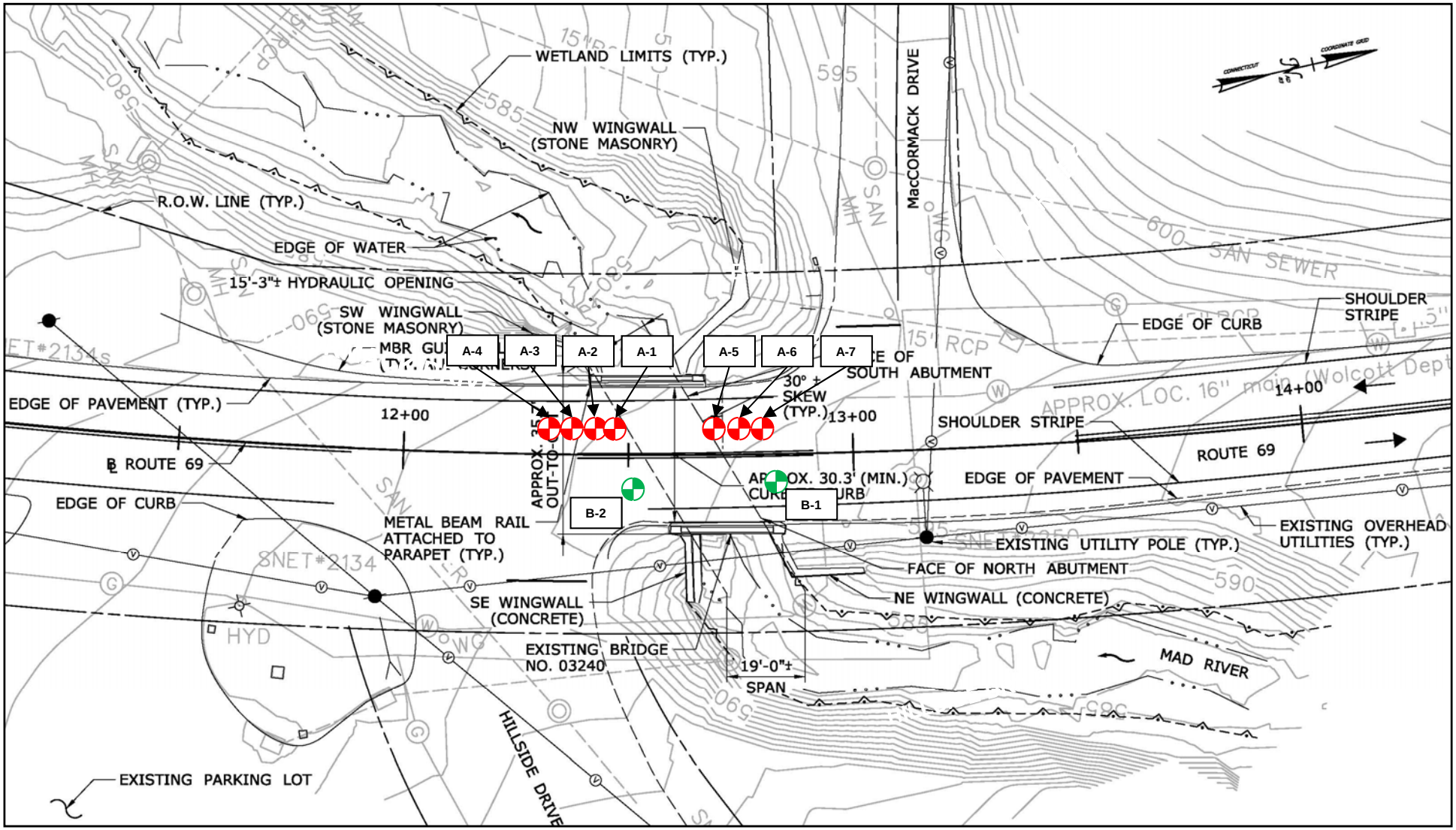
Our professional services for this project have been performed in accordance with generally accepted engineering practices; no warranty, express or implied, is made.

Figures

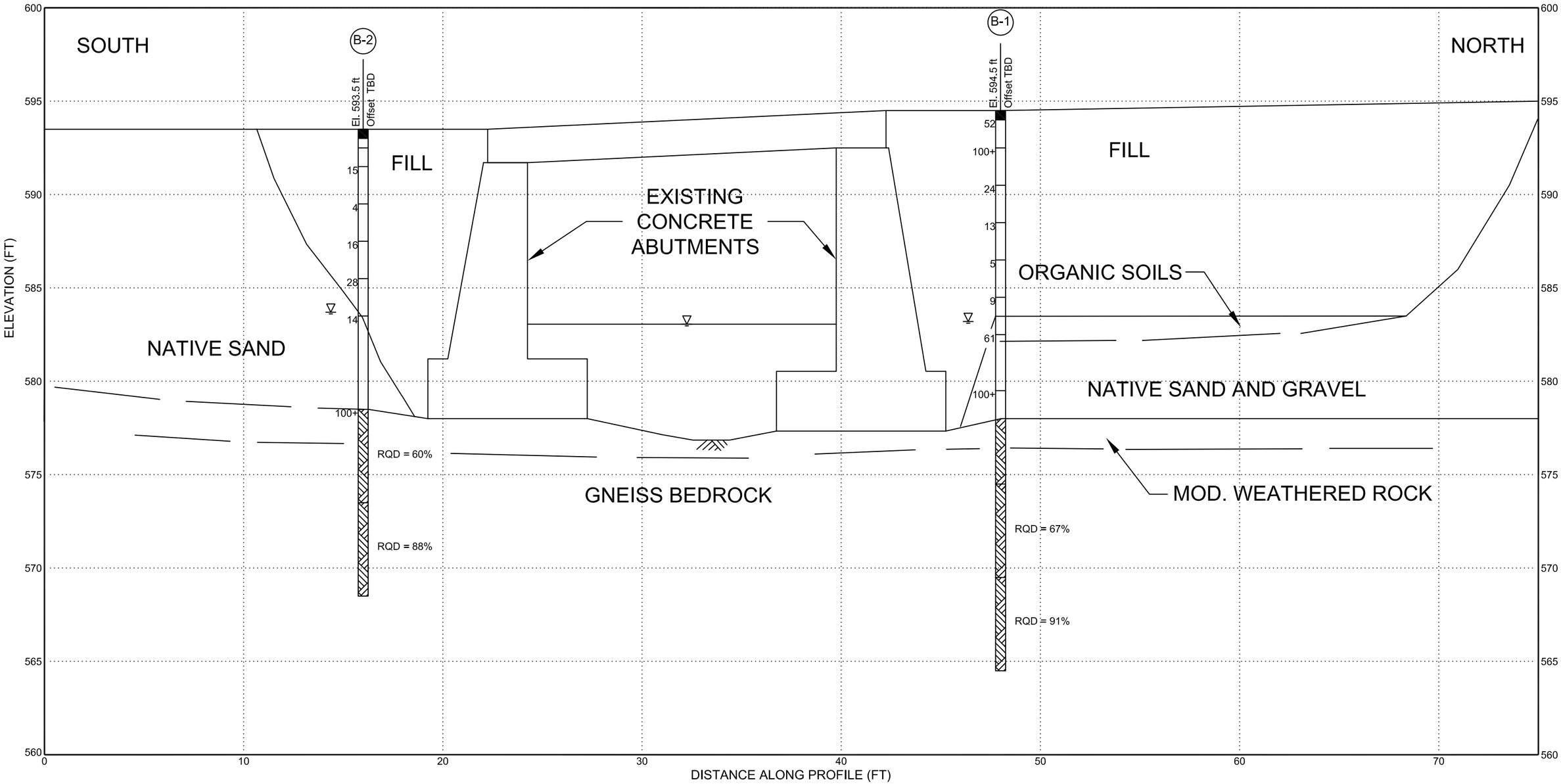
SCALE:	NTS	GEI Consultants	**SITE LOCATION PLAN** CT DOT BRIDGE NO. 03240 - ROUTE 69 OVER MAD RIVER WOLCOTT, CONNECTICUT	FIGURE NO. 1
DRAWN BY:	SC			
CHECKED BY:	--			
DATE:	10/19/2017		STATE PROJECT NO: **166-103**	

LEGEND

- Approximate Boring Location
- Approximate Auger Probe Location



SCALE:	NTS	GEI Consultants	**BORING LOCATION PLAN** CT DOT BRIDGE NO. 03240 - ROUTE 69 OVER MAD RIVER WOLCOTT, CONNECTICUT	FIGURE NO. 2
DRAWN BY:	SC			
CHECKED BY:	--			
DATE:	10/19/2017		STATE PROJECT NO: **166-103**	



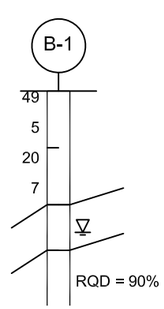
NOTES:

1. SUBSURFACE CONDITIONS BETWEEN BORING LOCATIONS MAY DIFFER. STRATUM BOUNDARIES ARE APPROXIMATE.

0 5 10

SCALE: 1" = 5'

LEGEND:



BORING NUMBER

EXISTING GROUND SURFACE

N-VALUE FROM STANDARD PENETRATION TEST

INTERPRETED STRATUM BOUNDARY

GROUNDWATER LEVEL MEASURED DURING DRILLING

ROCK QUALITY DESIGNATION

Bridge No. 03240 Route 69 Over Mad River Wolcott, Connecticut	GEI Consultants	SUBSURFACE PROFILE
CME Associates East Hartford, Connecticut	Project 125810	October 2017 Fig. 3

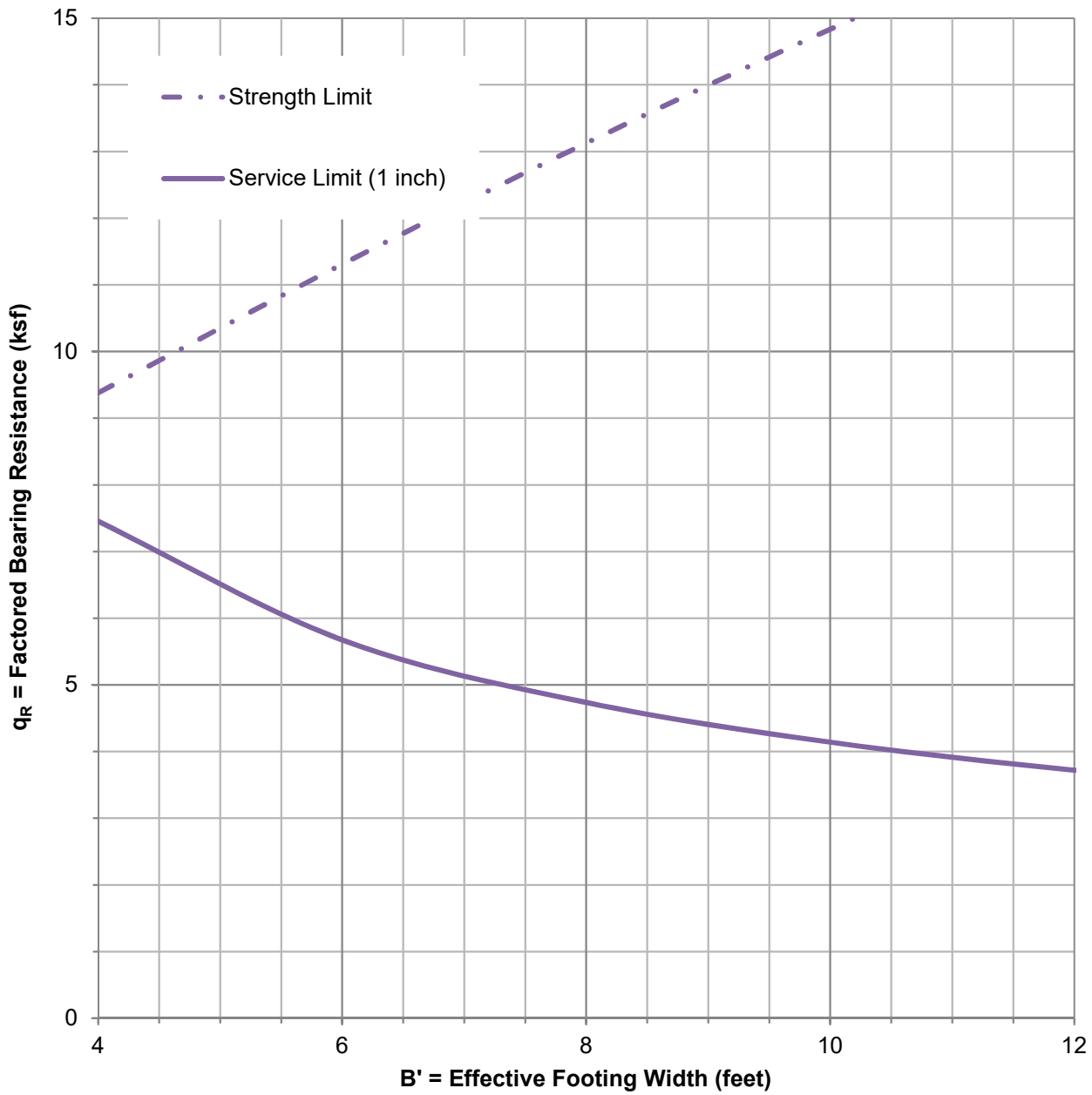
---- H:\TECH\project\CME Associates\125810 - CTDOT Bridge Liaison Program\03240 Route 69 Wolcott\Field Work\bridge no. 03240 draft sub-profile.dwg - 10/19/2017

Figure 4

GEI Consultants, Inc.

PHOTOGRAPHIC RECORD –

CT DOT BRIDGE NO. 03240 WOLCOTT, CT



Notes:
1. B' represents the smallest dimension (i.e. effective footing width).
2. Groundwater was assumed to be even with the ground surface at the front face of the wall.
3. The strength limit values are based on a resistance factor of 0.55 for wall footings, and the service limit values are based on a resistance factor of 1.0.
4. An embedment depth of 4 ft. was assumed.

Route 69 over Mad River Bridge No. 03240 Wolcott, CT	GEI Consultants	Factored Bearing Resistance vs. Effective Footing Width - Wingwall Footings
	State Project 166-103	Nov-19 Fig. 5

Appendix A

Boring Logs

Driller:	M. St. John	**Connecticut DOT Boring Report**		Hole No.:	B-1
Inspector:	L. McKiernan	Town:	Wolcott, Connecticut	Stat./Offset:	2+85/6'R
Engineer:	GEI	Project No.:	166-103	Northing:	778624.7
Start Date:	9-19-17	Route No.:	Route 69	Easting:	935585.8
Finish Date:	9-19-17	Bridge No.:	03240	Surface Elevation:	594.5

Project Description: CT DOT Bridge No. 03240 - Route 69 Over Mad River

Casing Size/Type: " ID / 4.0" OD	Sampler Type/Size: / " OD	Core Barrel Type: NX
Hammer Type: Safety Hammer	Hammer Wt.: 140 lb Fall: 30 in.	

Groundwater Observations: 11.3ft.

Depth (ft)	SAMPLES Sample Type/No.	Blows on Sampler per 6 inches	Pen. (in.)	Rec. (in.)	RQD %	Generalized Strata Description	Material Description and Notes	Elevation (ft)
0	S1	68 32 20	18	11		ROAD SECTION	7" ASPHALT 10" BASE - Gray and black f-c GRAVEL and ASPHALT PIECES up to 1.5" and f-c SAND, trace silt, dry.	
	S2	16 75 75/0"	12	8		FILL	S1B: Brown f-c SAND, some f-c gravel up to 1", trace silt, dry. S2: Brown to gray m-c SAND and f-c GRAVEL up to 1", trace silt, dry. Difficult drilling @ 3.0 ft. (Approx. 10" cobble)	
	S3	35 16 8 10	24	15			S3A (0-6"): Light gray f-m SAND, some f-m gravel up to 3/4", little silt, dry. COBBLE FRAGS. S3B (6-15"): Brown f-c SAND and f-c GRAVEL up to 1.25", trace silt, dry.	590
5	S4	12 4 9 11	24	10			S4: Brown f-c SAND, some f-m gravel up to 3/4", little silt, dry.	
	S5	4 3 2 2	24	13			S5: SIMILAR TO S4, damp.	
								585
10	S6	11 7 2 1	24	10			S6A (0-5"): Brown f-c GRAVEL up to 1.5", some f-c sand, trace silt, damp.	
	S7	3 31 30 27	24	12		ALLUVIAL SOILS	S6B (5-10"): Black and brown f-c SAND and Organic SILT (alternating 0.5"-1" layers), moist. Organic odor noted. S7A (0-6"): SIMILAR TO S6B, layers 1"-2" thick, some fibrous organics.	
						SAND AND GRAVEL	S7B (6-12"): Red/brown f-c GRAVEL up to 1.5", some f-c sand, trace nonplastic fines, wet.	
								580
15	S8	10 35 100/5"	17	11			S8: Red/brown and gray f-c SAND, some f-c GRAVEL up to 1", little nonplastic fines, moist to wet.	

Sample Type: S = Split Spoon R = Rock Core T = Undisturbed Piston V = Vane Shear Test

Proportions Used: Trace = 1 - 10%, Little = 10 - 20%, Some = 20 - 35%, And = 35 - 50%

Total Penetration in Earth: ft. Rock: ft.	NOTES: Driller advanced borehole with solid-stem auger (SSA) to 12 ft below grade and advanced further drive & wash with 4" OD casing. SPT samples were advanced using a safety hammer and spooling winch.	Sheet 1 of 2
No. of Soil Samples: 10 No. of Core Runs: 2		SM-001-M REV. 1/02

Driller: M. St. John	**Connecticut DOT Boring Report**		Hole No.: B-1
Inspector: L. McKiernan	Town: Wolcott, Connecticut		Stat./Offset: 2+85/6'R
Engineer: GEI	Project No.: 166-103		Northing: 778624.7
Start Date: 9-19-17	Route No.: Route 69		Easting: 935585.8
Finish Date: 9-19-17	Bridge No.: 03240		Surface Elevation: 594.5
Project Description: CT DOT Bridge No. 03240 - Route 69 Over Mad River			
Casing Size/Type: " ID / 4.0" OD	Sampler Type/Size: / " OD		Core Barrel Type: NX
Hammer Type: Safety Hammer	Hammer Wt.: 140 lb Fall: 30 in.		
Groundwater Observations: 11.3ft.			

Depth (ft)	SAMPLES: Sample Type/No.	SAMPLES: Blows on Sampler per 6 inches	SAMPLES: Pen. (in.)	SAMPLES: Rec. (in.)	SAMPLES: RQD %	Generalized Strata Description	Material Description and Notes	Elevation (ft)
						BEDROCK	Hard, steady resistance @ 16.5 ft, likely top of rock (mod. weathered). Advance with roller bit to 20.0 ft.	
								575
20	C1		60	52	67		C1 (20-25.0 ft): Dark gray GNEISSIC GRANOFELS, medium to coarse-grained, strong, highly fractured, joints slightly weathered at 5-20 deg. dip. CORE TIMES: 5:00, 6:00, 6:00, 5:00, 5:00	
								570
25	C2		58	59	91		C2 (25.0-29.8 ft): Dark gray GNEISSIC GRANOFELS, medium to coarse-grained, strong, moderate to highly fractured, joints slightly weathered at 5-15 deg. dip. CORE TIMES: 5:00, 5:30, 5:30, 6:00, 4:30	
								565
							End of Boring at 29.833 ft. Planned Extent. Borehole backfilled with cuttings and brought to grade with cold-patch asphalt.	

Sample Type: S = Split Spoon R = Rock Core T = Undisturbed Piston V = Vane Shear Test

Proportions Used: Trace = 1 - 10%, Little = 10 - 20%, Some = 20 - 35%, And = 35 - 50%

Total Penetration in Earth: ft. Rock: ft.	NOTES: Driller advanced borehole with solid-stem auger (SSA) to 12 ft below grade and advanced further drive & wash with 4" OD casing. SPT samples were advanced using a safety hammer and spooling winch.	Sheet 2 of 2
No. of Soil Samples: 10 No. of Core Runs: 2		SM-001-M REV. 1/02

Driller:	M. St. John	**Connecticut DOT Boring Report**		Hole No.:	B-2
Inspector:	L. McKiernan	Town:	Wolcott, Connecticut	Stat./Offset:	2+55/6'R
Engineer:	GEI	Project No.:	166-103	Northing:	778590.2
Start Date:	9-20-17	Route No.:	Route 69	Easting:	935582.7
Finish Date:	9-20-01	Bridge No.:	03240	Surface Elevation:	593.5

Project Description: CT DOT Bridge No. 03240 - Route 69 Over Mad River

Casing Size/Type: " ID / 4.0" OD	Sampler Type/Size: / " OD	Core Barrel Type: NX
Hammer Type: Safety Hammer	Hammer Wt.: 140 lb Fall: 30 in.	

Groundwater Observations: 9.8ft.

Depth (ft)	SAMPLES: Sample Type/No.	Blows on Sampler per 6 inches	Pen. (in.)	Rec. (in.)	RQD %	Generalized Strata Description	Material Description and Notes	Elevation (ft)
0						ROAD SECTION	6" ASPHALT	
	S1	32 36	12	5			18" BASE - S1: Gray to light gray f-c GRAVEL up to 1.5" and f-c SAND, trace silt, dry.	
	S2	20 9 6 4	24	15		FILL	S2: Brown f-c SAND, little f-m gravel up to 3/4", trace silt, dry.	
								590
	S3	4 2 2 6	24	9			S3: SIMILAR TO S2.	
5								
	S4	26 10 6 36	24	9			S4A (0-5"): SIMILAR TO S2.	
							S4B (5-9"): Red/brown f-c SAND, little nonplastic to low plasticity fines, trace f gravel up to 1/4", damp.	
	S5	2 8 20 30	24	6			S5: Gray/brown f-c SAND, some f-m gravel up to 3/4", little silt, dry. Crushed cobble up to 1.5" in final 2-inches of sample.	585
10	S6	8 10 4 4	24	8		NATIVE SAND	S6: Red/brown f-c SAND, some nonplastic to low plasticity fines, trace f gravel up to 1/4", trace organic fibers, damp.	
								580

Sample Type: S = Split Spoon R = Rock Core T = Undisturbed Piston V = Vane Shear Test

Proportions Used: Trace = 1 - 10%, Little = 10 - 20%, Some = 20 - 35%, And = 35 - 50%

Total Penetration in Earth: ft. Rock: ft.	NOTES: Driller advanced borehole with solid-stem auger (SSA) to 10 ft below grade and advanced further drive & wash with 4" OD casing. SPT samples were advanced using a safety hammer and spooling winch.	Sheet 1 of 2
No. of Soil Samples: 9 No. of Core Runs: 2		SM-001-M REV. 1/02

Driller: M. St. John	**Connecticut DOT Boring Report**		Hole No.: B-2
Inspector: L. McKiernan	Town: Wolcott, Connecticut		Stat./Offset: 2+55/6'R
Engineer: GEI	Project No.: 166-103		Northing: 778590.2
Start Date: 9-20-17	Route No.: Route 69		Easting: 935582.7
Finish Date: 9-20-01	Bridge No.: 03240		Surface Elevation: 593.5
Project Description: CT DOT Bridge No. 03240 - Route 69 Over Mad River			
Casing Size/Type: " ID / 4.0" OD	Sampler Type/Size: / " OD		Core Barrel Type: NX
Hammer Type: Safety Hammer	Hammer Wt.: 140 lb	Fall: 30 in.	
Groundwater Observations: 9.8ft.			

Depth (ft)	SAMPLES Sample Type/No.	Blows on Sampler per 6 inches	Pen. (in.)	Rec. (in.)	RQD %	Generalized Strata Description	Material Description and Notes	Elevation (ft)
15	S7 C1	75/0"	0 60	0 52	60	BEDROCK	C1 (15-20.0 ft): Dark gray GNEISSIC GRANOFELS, medium- to coarse-grained, strong, slightly weathered at joints, with moderately weathered and fractured zone @ 15.5 - 17.0 ft. CORE TIMES: 4:30, 4:30, 4:00, 4:00, 4:30	
								575
20	C2		60	60	88		C2 (20-25.0 ft): Dark gray GNEISSIC GRANOFELS, medium- to coarse-grained, strong, highly fractured, joints slightly weathered at 5-20 deg. dip. CORE TIMES: 4:00, 5:00, 6:00, 6:00, 3:30	
								570
25							End of Boring at 25 ft. Planned Extent. Borehole backfilled with cuttings and brought to grade with cold-patch asphalt.	

Sample Type: S = Split Spoon R = Rock Core T = Undisturbed Piston V = Vane Shear Test

Proportions Used: Trace = 1 - 10%, Little = 10 - 20%, Some = 20 - 35%, And = 35 - 50%

Total Penetration in Earth: ft. Rock: ft.	NOTES: Driller advanced borehole with solid-stem auger (SSA) to 10 ft below grade and advanced further drive & wash with 4" OD casing. SPT samples were advanced using a safety hammer and spooling winch.	Sheet 2 of 2
No. of Soil Samples: 9 No. of Core Runs: 2		SM-001-M REV. 1/02

Appendix B

Laboratory Test Results

Laboratory Test Results
CT DOT Bridge No. 03240
Route 69 over Mad River
Wolcott, Connecticut

Boring ID	Sample Number	Top Depth (feet)	Bottom Depth (feet)	USCS Symbol	Soil Stratum	Natural Moisture Content (%)	Grain Size Analyses		
							Gravel (%)	Sand (%)	Fines (%)
B-1	S4	6	8	SM	FILL	6.8	27.7	60.3	12.0
B-1	S8	15	16.5	SW-SM	NATIVE	11.7	29.0	60.3	10.7
B-2	S5	8	10	SM	FILL	5.1	26.4	57.0	16.6
B-2	S6	10	12	SC	NATIVE	41.5	0.7	75.6	23.7

Client:	GEI Consultants, Inc.				
Project:	CME-CONNDOT Bridge Liaison Contrac-1011				
Location:	Bridge 03240			Project No:	GTX-307078
Boring ID:	---	Sample Type:	---	Tested By:	GA
Sample ID:	---	Test Date:	10/06/17	Checked By:	emm
Depth :	---	Test Id:	426721		

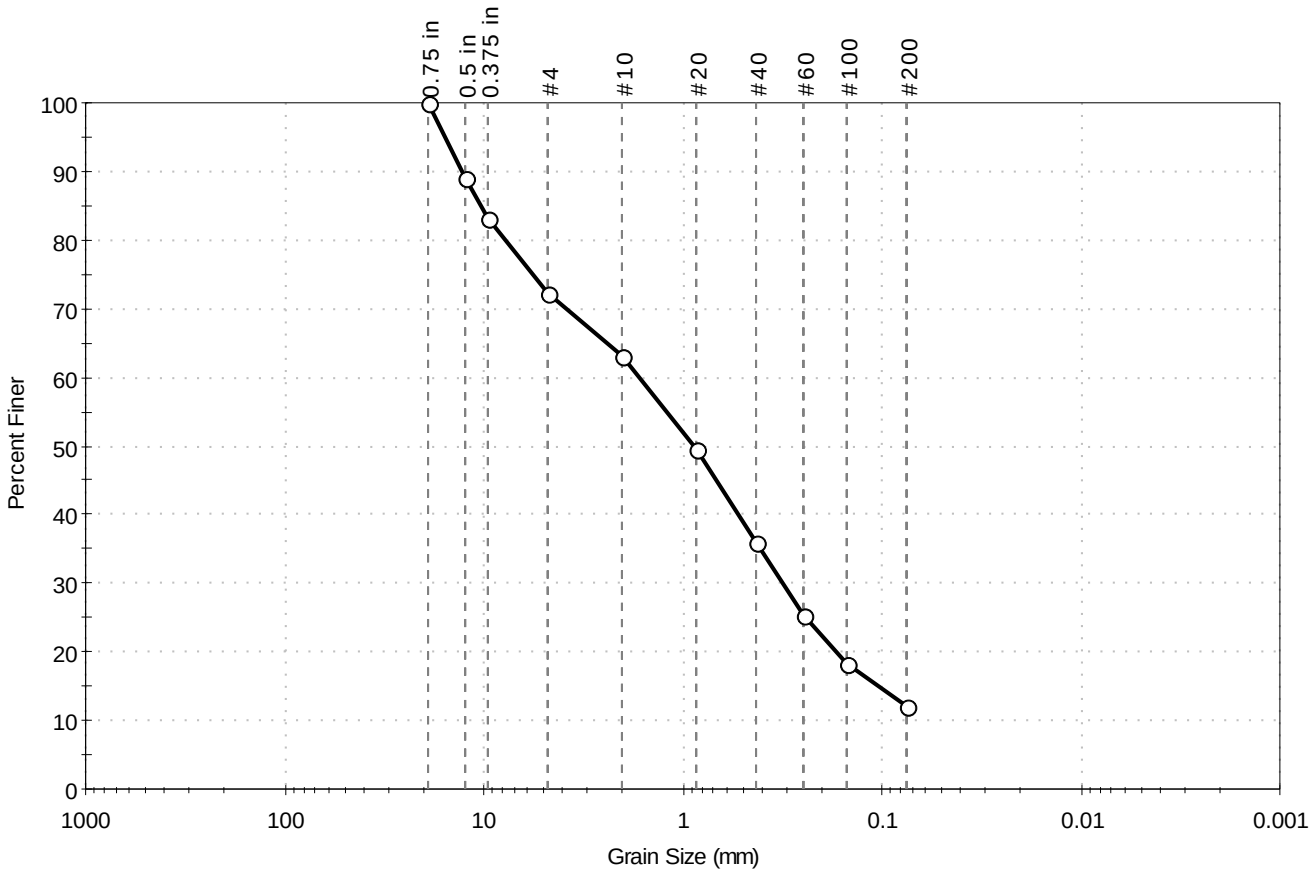
Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content, %
B-1	S4	6-8 ft	Moist, olive brown silty sand with gravel	6.8
B-1	S8	15-16.5 ft	Moist, dark brown sand with silt and gravel	11.7
B-2	S5	8-10 ft	Moist, olive brown silty sand with gravel	5.1
B-2	S6	10-12 ft	Moist, dark brown clayey sand	41.5

Notes: Temperature of Drying : 110º Celsius

Client:	GEI Consultants, Inc.				
Project:	CME-CONNDOT Bridge Liaison Contrac-1011				
Location:	Bridge 03240			Project No:	GTX-307078
Boring ID:	B-1	Sample Type:	bag	Tested By:	GA
Sample ID:	S4	Test Date:	10/06/17	Checked By:	emm
Depth :	6-8 ft	Test Id:	426722		
Test Comment:	---				
Visual Description:	Moist, olive brown sand with silt and gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	27.7	60.3	12.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	89		
0.375 in	9.50	83		
#4	4.75	72		
#10	2.00	63		
#20	0.85	50		
#40	0.42	36		
#60	0.25	25		
#100	0.15	18		
#200	0.075	12		

Coefficients

D_{85} = 10.3103 mm D_{30} = 0.3132 mm

D_{60} = 1.6327 mm D_{15} = 0.1053 mm

D_{50} = 0.8666 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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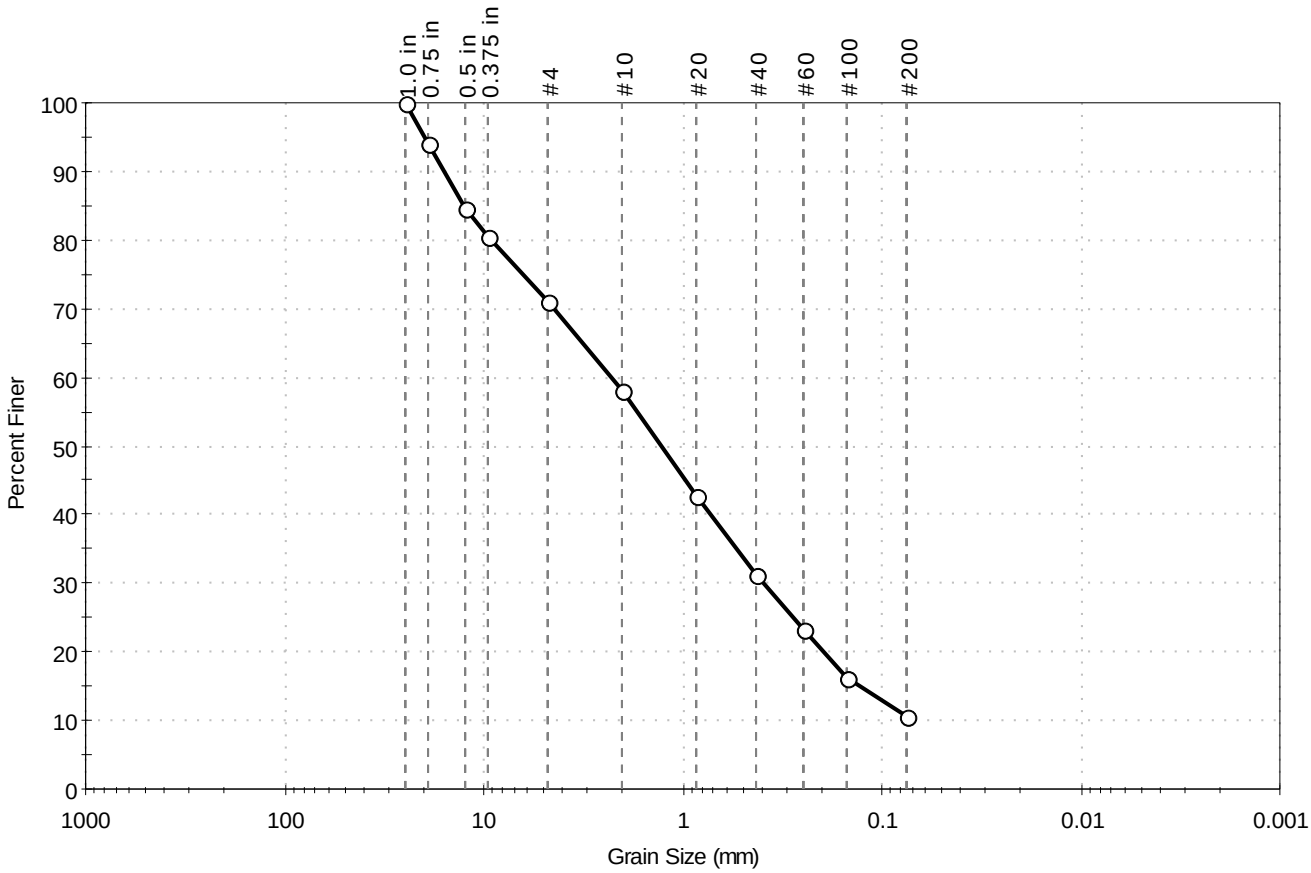
Client:	GEI Consultants, Inc.				
Project:	CME-CONNDOT Bridge Liaison Contrac-1011				
Location:	Bridge 03240			Project No:	GTX-307078
Boring ID:	B-1	Sample Type:	bag	Tested By:	GA
Sample ID:	S8	Test Date:	10/06/17	Checked By:	emm
Depth :	15-16.5 ft	Test Id:	426723		
Test Comment:	---				
Visual Description:	Moist, dark brown sand with silt and gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	29.0	60.3	10.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.0 in	25.00	100		
0.75 in	19.00	94		
0.5 in	12.50	85		
0.375 in	9.50	81		
#4	4.75	71		
#10	2.00	58		
#20	0.85	43		
#40	0.42	31		
#60	0.25	23		
#100	0.15	16		
#200	0.075	11		

Coefficients

D_{85} = 12.6747 mm D_{30} = 0.3888 mm

D_{60} = 2.2875 mm D_{15} = 0.1279 mm

D_{50} = 1.2780 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

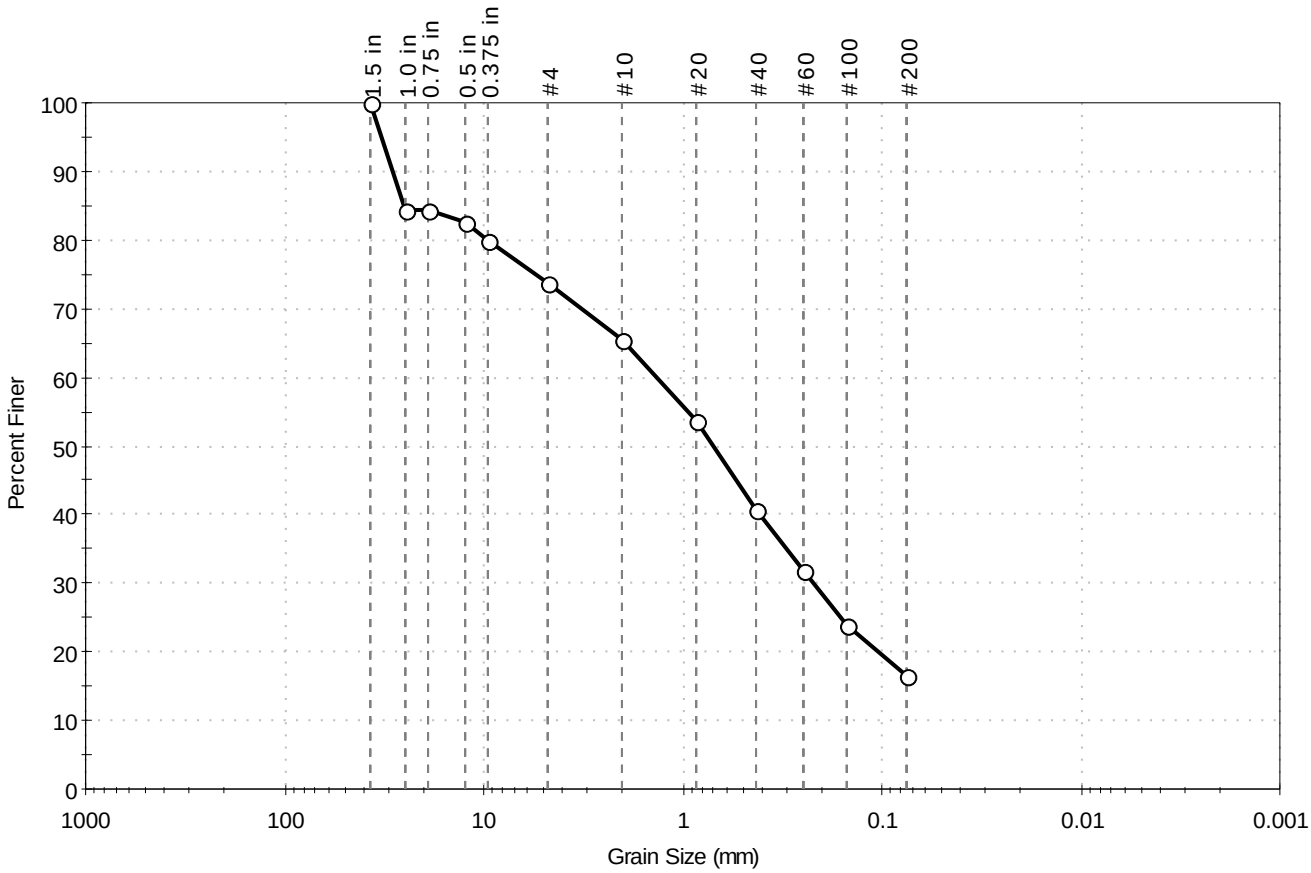
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	GEI Consultants, Inc.				
Project:	CME-CONNDOT Bridge Liaison Contrac-1011				
Location:	Bridge 03240			Project No:	GTX-307078
Boring ID:	B-2	Sample Type:	bag	Tested By:	GA
Sample ID:	S5	Test Date:	10/06/17	Checked By:	emm
Depth :	8-10 ft	Test Id:	426724		
Test Comment:	---				
Visual Description:	Moist, olive brown silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	26.4	57.0	16.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1.0 in	25.00	84		
0.75 in	19.00	84		
0.5 in	12.50	83		
0.375 in	9.50	80		
#4	4.75	74		
#10	2.00	65		
#20	0.85	54		
#40	0.42	41		
#60	0.25	32		
#100	0.15	24		
#200	0.075	17		

Coefficients

D_{85} = 25.4741 mm D_{30} = 0.2228 mm

D_{60} = 1.3410 mm D_{15} = N/A

D_{50} = 0.6948 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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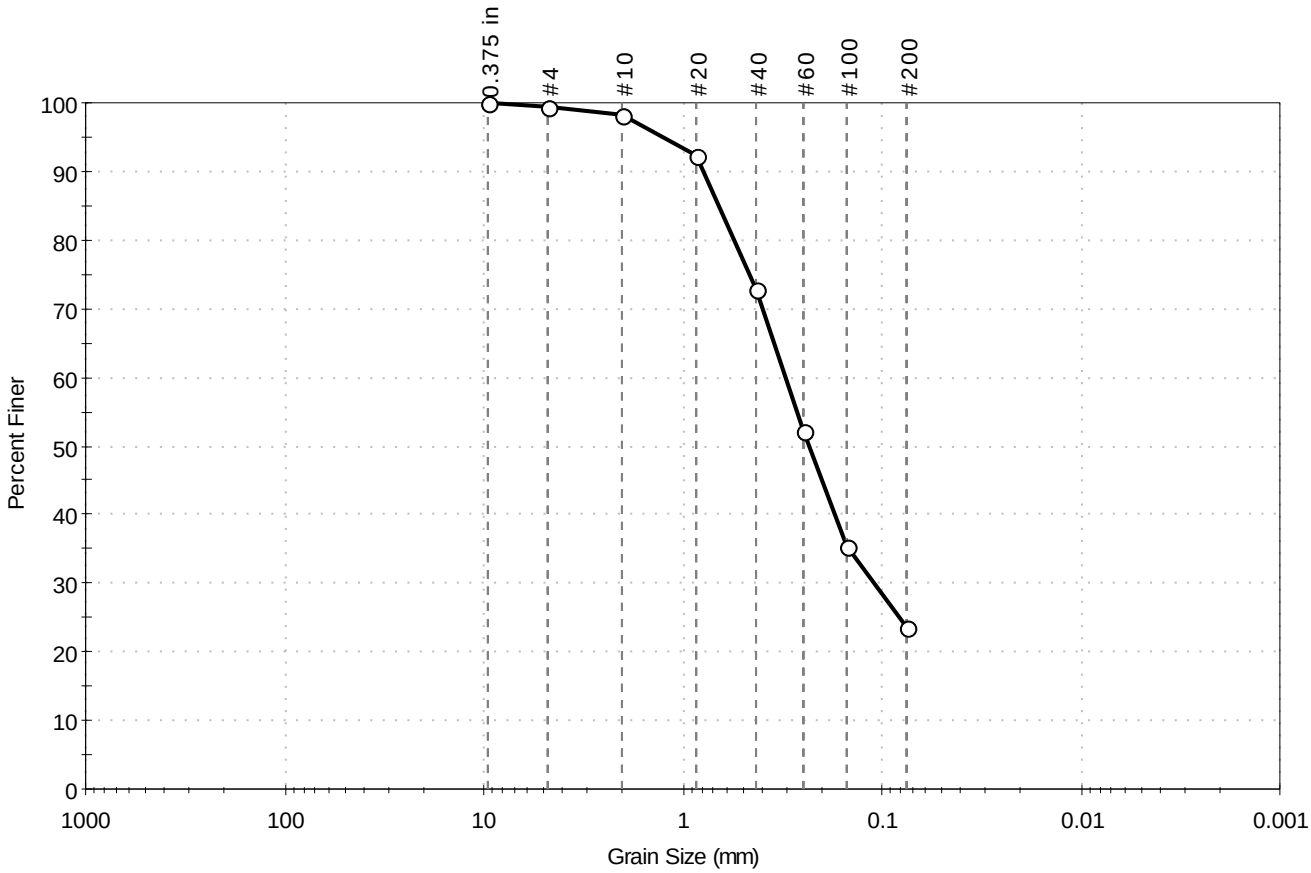
Client:	GEI Consultants, Inc.				
Project:	CME-CONNDOT Bridge Liaison Contrac-1011				
Location:	Bridge 03240			Project No:	GTX-307078
Boring ID:	B-2	Sample Type:	bag	Tested By:	GA
Sample ID:	S6	Test Date:	10/06/17	Checked By:	emm
Depth :	10-12 ft	Test Id:	426725		
Test Comment:	---				
Visual Description:	Moist, dark brown clayey sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	0.7	75.6	23.7

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	99		
#10	2.00	98		
#20	0.85	92		
#40	0.42	73		
#60	0.25	52		
#100	0.15	35		
#200	0.075	24		

Coefficients

D_{85} = 0.6556 mm D_{30} = 0.1093 mm

D_{60} = 0.3051 mm D_{15} = N/A

D_{50} = 0.2339 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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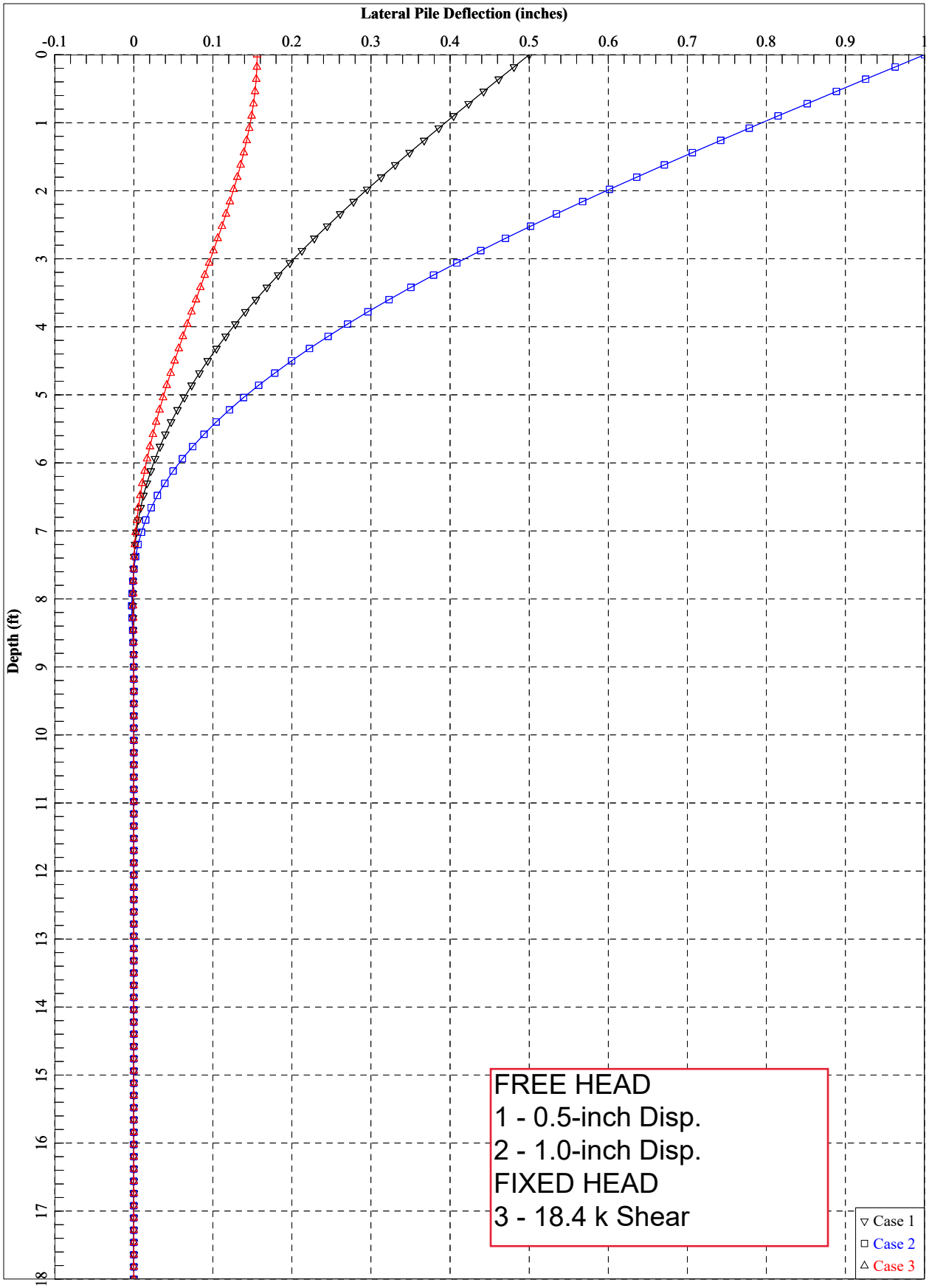
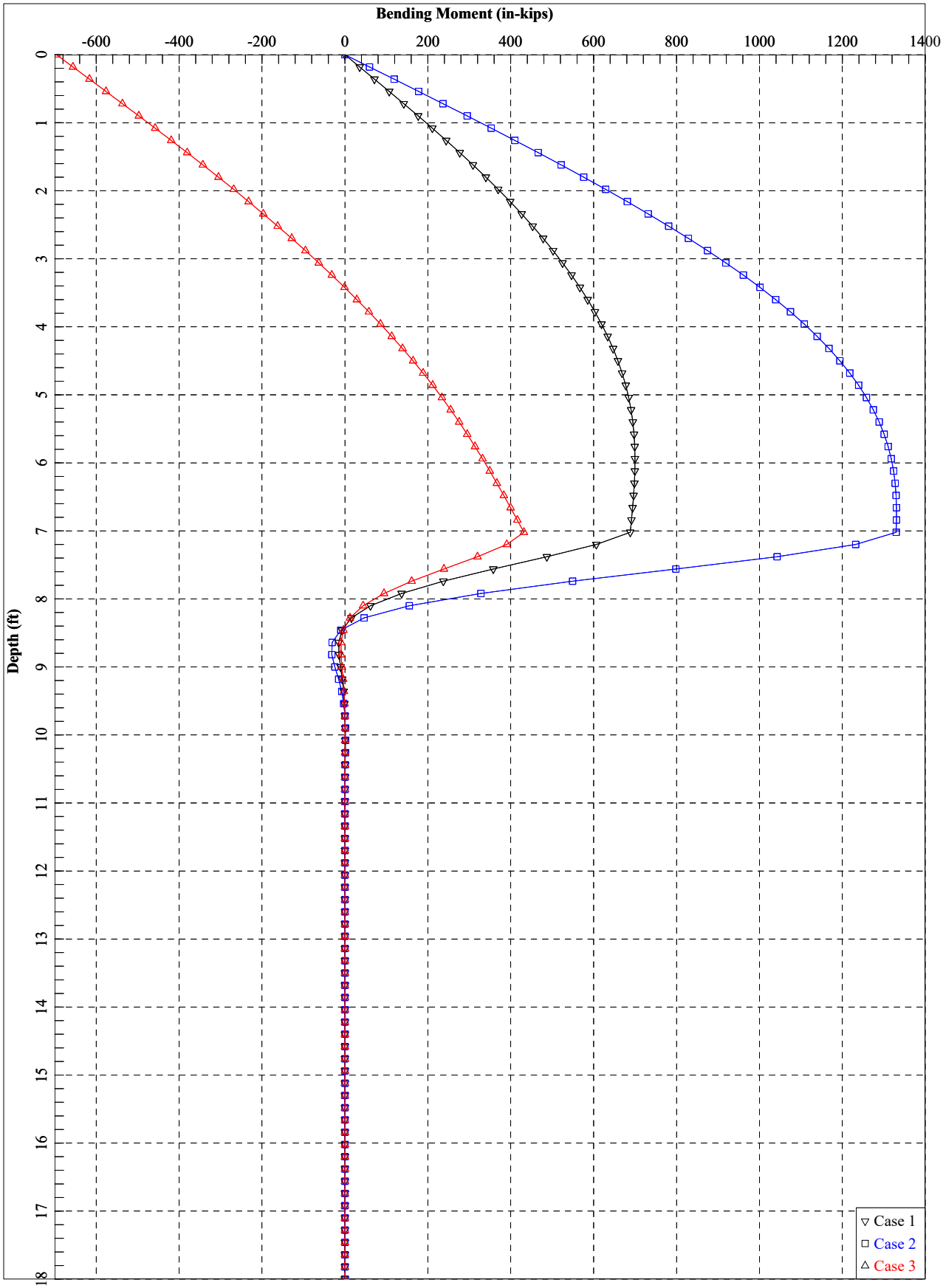
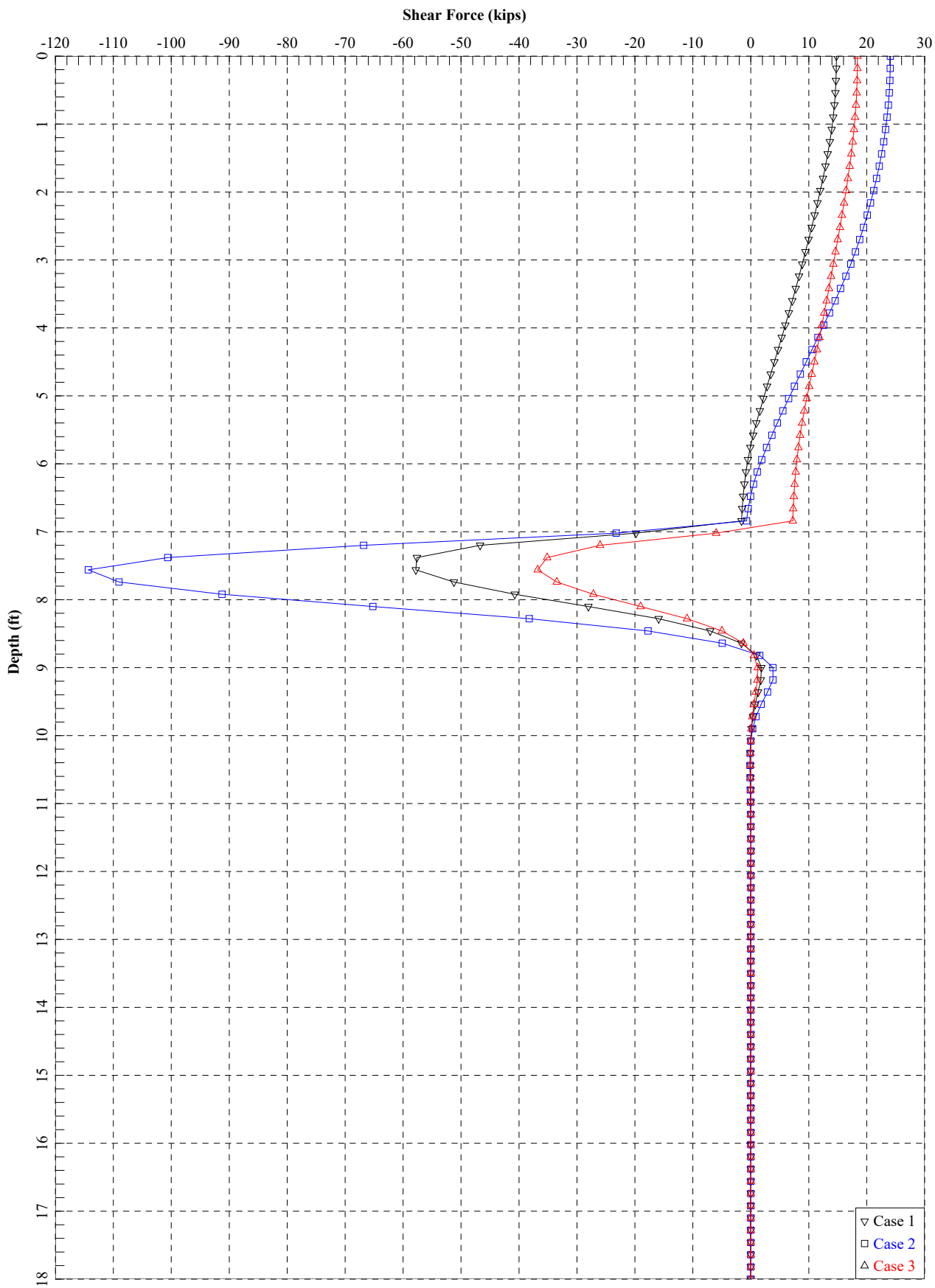
Appendix C

Geotechnical Calculations

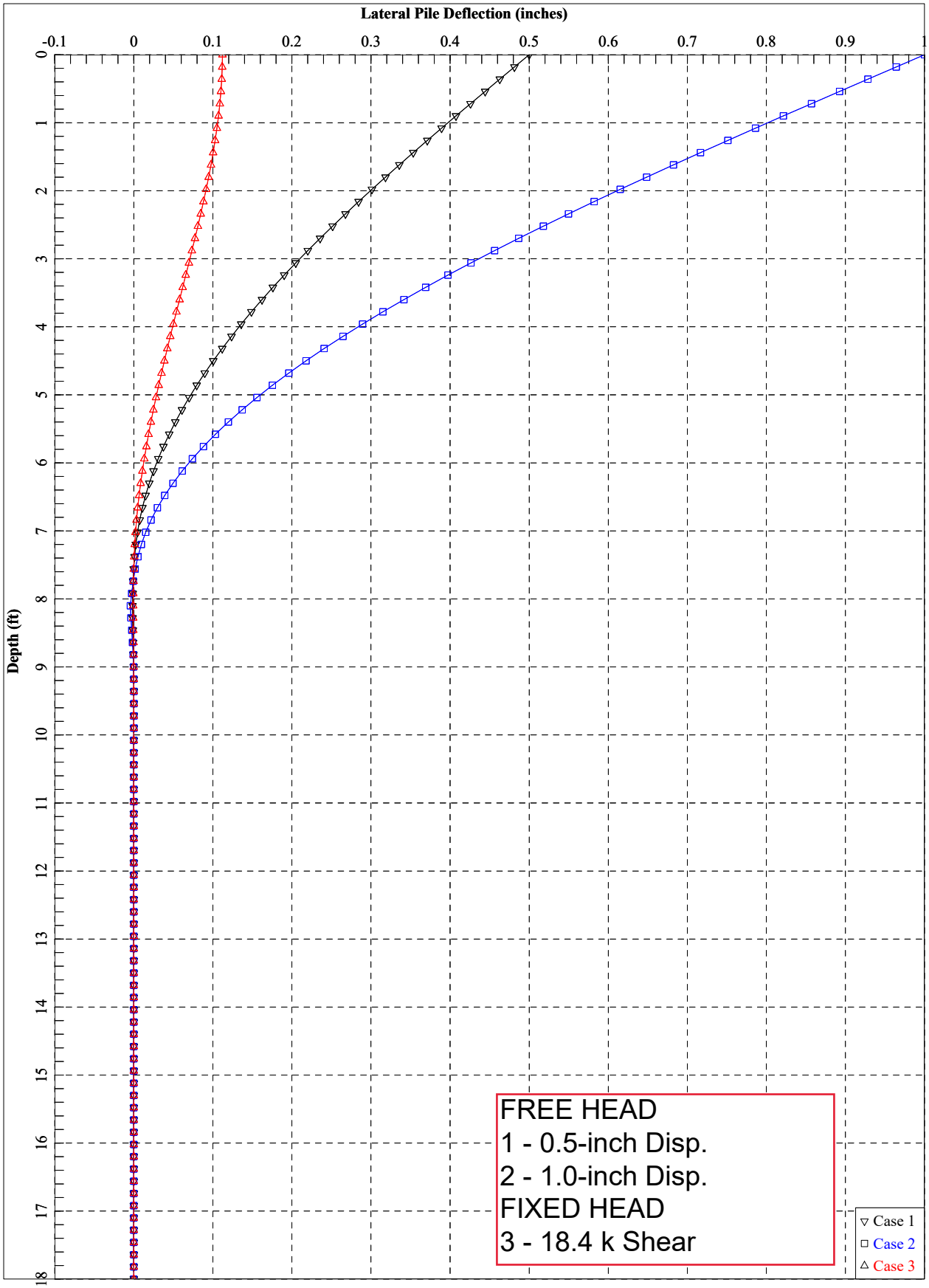
9.625-inch Micropile

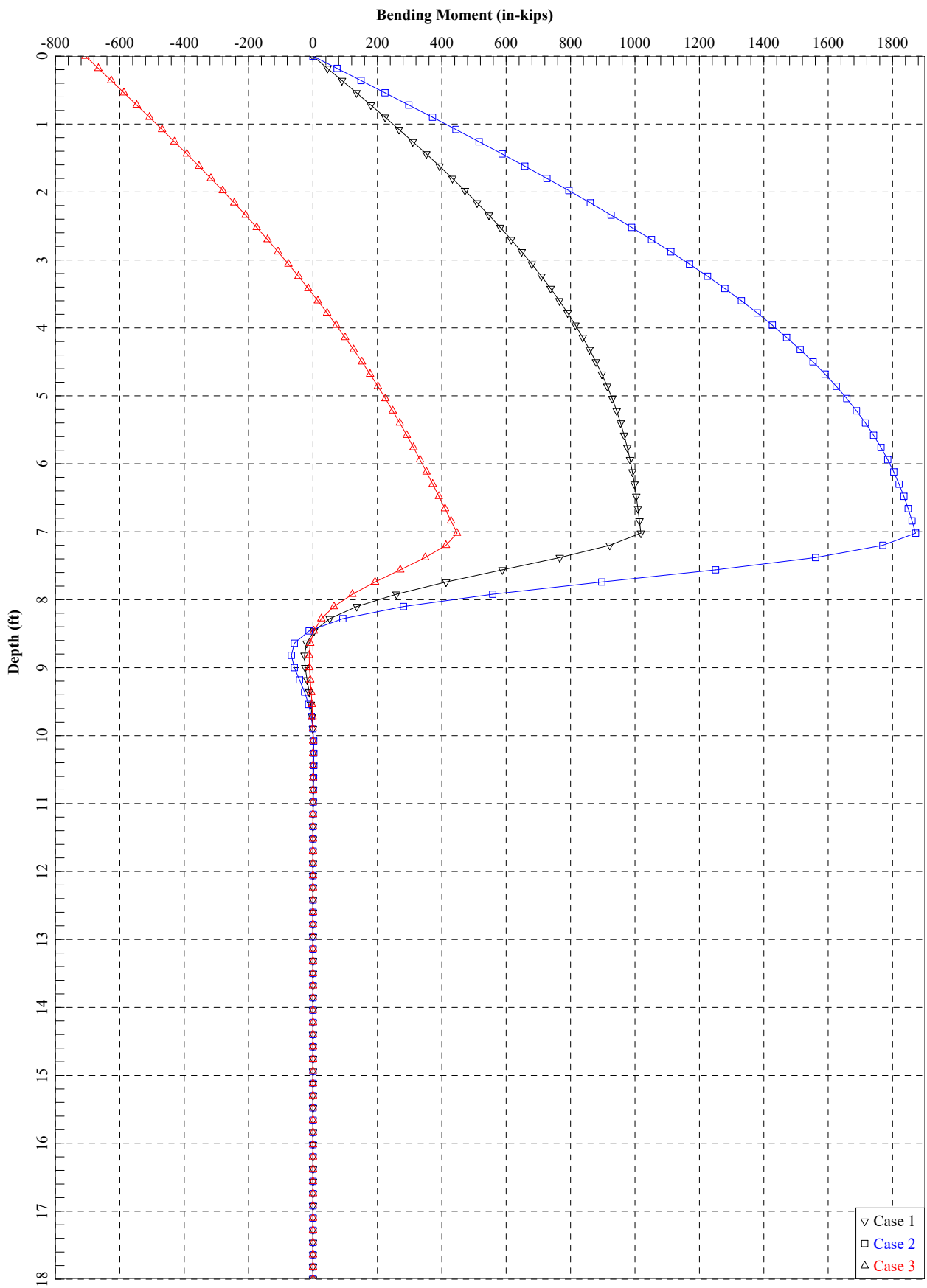


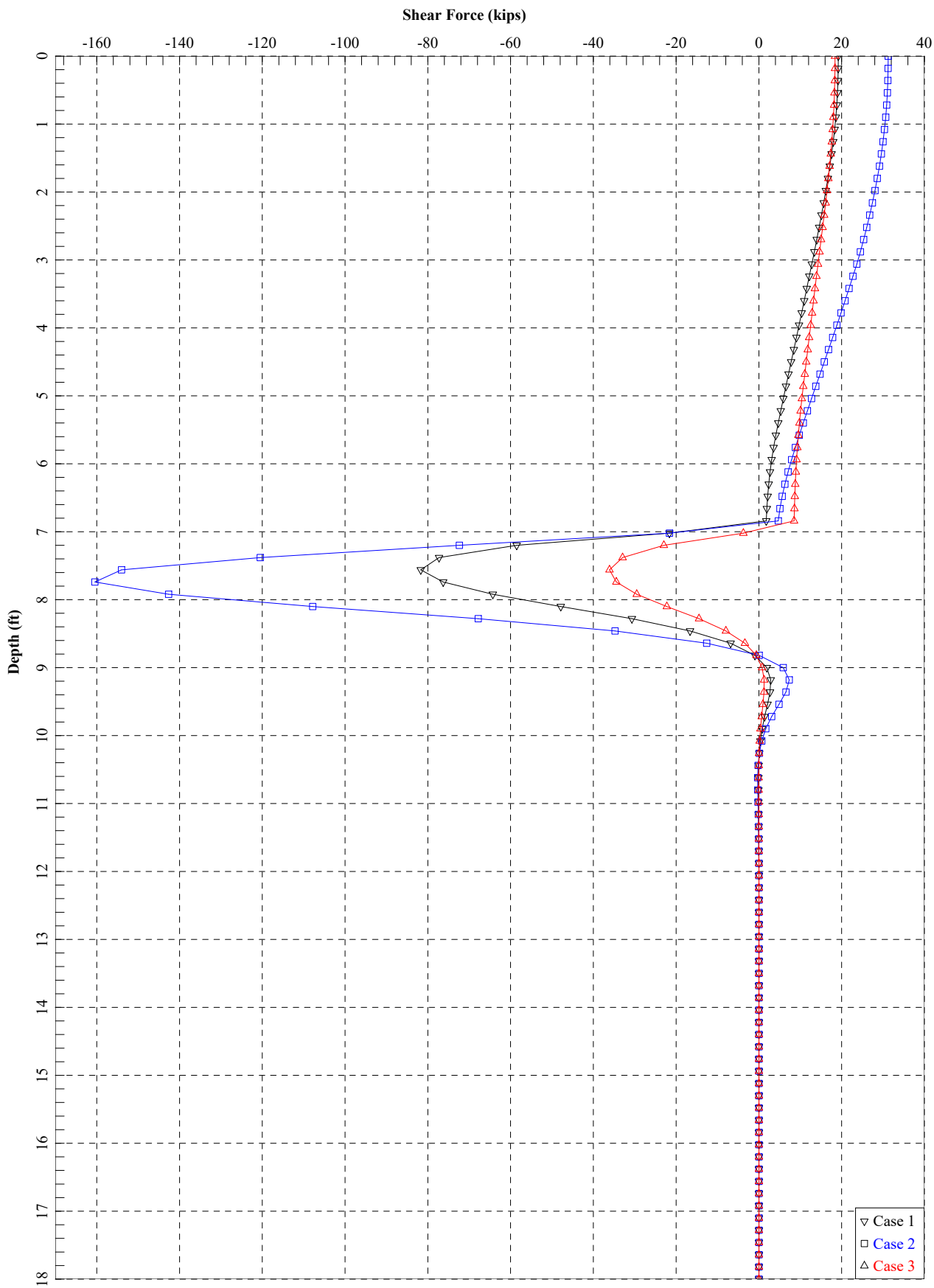

Bending Moment (in-kips)
-600
-400
-200
0
200
400
600
800
1000
1200
1400
Depth (ft)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
Case 1
Case 2
Case 3


Shear Force (kips)
-120
-110
-100
-90
-80
-70
-60
-50
-40
-30
-20
-10
0
10
20
30
Depth (ft)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
Case 1
Case 2
Case 3

10.75-inch Micropile




Bending Moment (in-kips)
-800
-600
-400
-200
0
200
400
600
800
1000
1200
1400
1600
1800
Depth (ft)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
Case 1
Case 2
Case 3


Shear Force (kips)
-160
-140
-120
-100
-80
-60
-40
-20
0
20
40
Depth (ft)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
Case 1
Case 2
Case 3