Ward Geotechnical
Consulting, PLLC

August 11, 2011
Project 11210

Mr. Edward L. Chase, P.E.
The Louis Berger Group, Inc.
100 Commercial Street, 2nd Floor North
Manchester, New Hampshire 03101

Subject: Foundation Investigation
Whittier Street Bridge over the Cocheco River
Dover, New Hampshire

Dear Mr. Chase:

Ward Geotechnical Consulting, PLLC (WGC) has prepared this letter report to summarize the results of the foundation investigation conducted for the proposed replacement of the Whittier Street Bridge over the Cocheco River in Dover, New Hampshire. Our work on the project was authorized by the subconsultant agreement between The Louis Berger Group, Inc. (LBG) and WGC, dated April 18, 2011.

PROJECT AND SITE DESCRIPTION

The project involves the replacement of the Whittier Street Bridge over the Cocheco River in Dover, New Hampshire. The location of the site is shown on Figure 1 and a site plan is shown on Figures 2A and 2B.

The existing bridge is a steel girder structure with a concrete deck. The bridge has two spans totaling about 120 feet. The superstructure is supported on stone masonry abutments and a cast-in-place concrete center pier. The center pier was constructed when the bridge was last replaced circa 1962. The stone masonry abutments predate the c. 1962 bridge replacement, but the date of construction of the abutments is not known. Based on our review of the design plans for the existing bridge, which were prepared by Wesley L. Haynes, P.E., dated July 2, 1962, it appears that the center pier was cast on bedrock. Several bedrock outcrops are visible in the river, both upstream and downstream of the existing bridge. Foundation conditions and the base widths and depths of the existing stone masonry abutments and wing walls are not known.

LBG is considering two options for the new bridge. One option is to construct a single span structure supported on new cast-in-place concrete abutments located either at or behind the existing stone masonry abutments. The other option being considered by LBG is a two span structure. The existing center pier would be modified or replaced at its current location to provide intermediate support for the new bridge. New cast-in-place concrete abutments would be constructed at or behind the existing stone masonry abutments. For either option, the total length of the bridge span(s) would probably be on the order of 130 to 155 feet. We understand that the replacement bridge will be a few feet wider than the existing bridge and the alignment will be slightly changed to improve the horizontal alignment of the approach roadway. Minor changes might also be made to the vertical alignment of the bridge and roadway. Current plans include the reconstruction of approximately 600 feet of approach roadway on either side of the bridge.

SUBSURFACE INVESTIGATION

Boring Program

WGC engaged New Hampshire Boring, Inc. to drill 13 borings at the site. The borings were drilled from April 27 through May 3, 2011. Boring logs are provided in Appendix A.

The locations of the borings are shown on the Boring Location Plan provided on Figures 2A and 2B. The site plan used to prepare the Boring Location Plan was provided in AutoCAD format by LBG. Most of the boring locations and ground surface elevations were surveyed by others after the borings were drilled. One boring (B103) was missed in the survey and its location was determined by WGC by measuring ties to features shown on the site plan. The ground surface elevation at B103 was estimated based on nearby spot elevations shown on the site plan. The elevations are referenced to the National Geodetic Vertical Datum of 1929 (NGVD29).

Nine of the borings were drilled in the vicinity of the existing abutments (B101 through B109) to depths ranging from 8 feet (B109) to 35 feet (B103) below the existing ground surface using cased wash boring drilling techniques. Split-spoon soil sampling with standard penetration tests (SPTs) was conducted at depth intervals ranging from 3 to 5 feet in all borings except B105A. B105 was abandoned at a depth of 19 feet due to difficulties with advancing the casing through boulders (or possibly masonry blocks). B105A was subsequently drilled about 2.5 feet east of B105 to obtain subsurface information below 19 feet, and split-spoon sampling was not performed in the upper 19 feet. Bedrock core samples were drilled in five of the borings (B101, B103, B105A, B107, and B109). The aggregate length of bedrock core sample drilled in each of the five borings ranged from 5 to 11.3 feet.

Four of the borings were drilled in Whittier Street about 200 to 500 feet from the bridge abutments to obtain information concerning existing base and subgrade soils in the portions of the approach roadway that will be reconstructed. These borings were advanced to depths ranging from 5 to 6.5 feet below the existing roadway surface using hollow-stem augers. Split-spoon soil sampling with SPTs was conducted at depth intervals of 2 feet, or less. The uppermost split-spoon sample in each of these borings was obtained using a 3-inch-diameter split-spoon (oversize) driven using a 300-pound hammer with a 24 inch drop. The oversize spoon was used to obtain larger, more representative samples of the existing base soils for visual classification and mechanical grain size analyses.

Mechanical grain size analyses were conducted on four soil samples obtained from the borings. The results of the grain size analyses are provided in Appendix B.

SUBSURFACE CONDITIONS

The subsurface conditions encountered in the borings drilled in the vicinity of the abutments, the borings drilled in the approach roadway (about 200 to 500 feet from the bridge abutments), and borings and probes drilled in the vicinity of the intermediate pier (for the 1962 design of the bridge replacement) are described below, from the ground surface down. Conditions at the borings drilled in the vicinity of the existing bridge abutments are also shown on the subsurface profiles on Figures 3A through 4B. Subsurface conditions are known only at the boring locations. Conditions at other locations may differ.

Vicinity of Existing Abutments (B101 through B109)

Pavement – The asphalt pavement at the borings drilled in the road behind the abutments (B101 through B108) ranges from about 7.5 to 11 inches thick.

Fill (Unified Soil Classification SW, SW-SM, SP-SM, SM, ML) – Fill was encountered in all of the borings drilled in the vicinity of the abutments (B101 through B109). The fill in these borings extends to depths of about 2 to 19 feet below the roadway surface, with fill thickness decreasing with distance from the abutment walls. The fill consists of a wide variety of soils, including sand with gravel, sand with silt and gravel, silty sand, silty sand with gravel, sandy silt, and sandy silt with gravel. Several cobbles and boulders were also encountered in the fill. SPT N-values in the fill typically ranged from 5 to 28 blows per foot, indicating that the fill is loose to medium dense. An SPT N-value of 37 was obtained in the fill in B105 (S3). However, the split-spoon sampler was obstructed by a boulder or cobble (sampler deflected and was bent), and the SPT N-value is not considered representative of the relative density of the soil.

The fill layer encountered beneath the pavement in the borings drilled in the road behind the abutments (B101 through B108) includes pavement base soils. These base soils extend to depths ranging from about 1.5 to 3 feet below the existing roadway surface. The base soils consist primarily of sand with gravel and sand with gravel and silt, but also include silty sand with gravel in a few areas. SPT N-values in the base soils ranged from 9 to 28 blows per foot, indicating that the soils are loose to medium dense.

Sandy Silt and Silty Sand (ML, SM) – A layer of sandy silt and silty sand was encountered beneath the fill in all of the borings drilled in the roadway behind the abutments, except B105 and B105A. These borings were drilled close to the abutment and wing wall at the northeast quadrant of existing bridge, and the silty sand/sandy silt layer was probably removed during construction of the adjacent walls. Where observed, the sandy silt and silty sand layer is approximately 4 to 9.5 feet thick, with the bottom of the layer extending to depths ranging from about 8 to 17 feet below the existing roadway surface. SPT N-values in the layer ranged from 4 to 16 blows per foot, indicating that the soil is very loose to medium dense.

At several locations, the sandy silt and silty sand was observed to have a stratified structure, suggesting that it was deposited by or through water. Fine roots were observed in some of the samples. A twig was observed in one of the samples (B102-S3) and small pockets of organic soils were observed in another sample (B103-S4). The sandy silt observed in the samples has low to medium plasticity. Natural Resource Conservation Service soil mapping indicates that the area of the bridge contains glaciomarine deposits of "silt loam" (silt and silty sand per the USCS). However, the sandy silt and silty sand layer might be an alluvial or glacio-fluvial deposit. In some of the samples, the sandy silt and silty sand did not appear to be stratified, and might have been locally excavated and placed as fill during construction of the existing bridge.

Silty Sand with Gravel (SM) – A layer of silty sand with gravel containing several cobbles and boulders was encountered beneath the fill in B105A and beneath the sandy silt/silty sand layer in the other borings drilled in the roadway behind the abutments (B101 through B104 and B106 through B108). At the boring locations, the silty sand with gravel layer ranges in thickness from about 3 to 10 feet. The bottom of the layer at the boring locations ranges in depth from about 15 feet (B108) to about 24 feet (B101). SPT N-values in the layer typically ranged from 37 to 87 blows per foot, indicating the soil is dense to very dense. Several SPT refusals (N-values greater than 100 blows per foot, or greater than 50 blows per 6 inches) were also encountered, probably due to obstruction of the split-spoon sampler by cobbles and boulders. The silty sand with gravel was observed to have a heterogeneous structure and was probably deposited as glacial till.

Several SPT refusals and poor sample recoveries (sample recoveries less than half of the split-spoon penetration) were obtained in the silty sand with gravel layer due to obstruction of the split-spoon sampler by cobbles and boulders. Large gravel probably also obstructed the sampler, contributing to the poor sample recoveries. Note that the standard split-spoon sampler has an inner diameter of approximately 2 inches and the maximum particle size admitted by the sampler is practically limited to about 1.5 inches. The presence of cobbles, boulders, and large gravel must be considered when interpreting sampling and SPT results. For example, these obstructions will cause some of the SPT N-values to be larger than considered representative of the relative density of the soil (i.e., the soil might be less dense than indicated by the SPT N-value). Also, since gravel greater than 1.5 inches, cobbles, and boulders are excluded from the samples, the gradation of the in situ soil is probably coarser than observed in the samples.

Bedrock – Bedrock was encountered beneath the fill in B109, and beneath the glacial till layer in the borings drilled in the roadway behind the abutments from which rock core samples were drilled (B101, B103, B105A, and B107). In B109, which was drilled near the toe of the slope north of the west abutment, bedrock was encountered at a depth of about 3 feet, corresponding to elevation 45.5 feet. In the other borings from which rock core samples were drilled, bedrock was encountered at depths ranging from about 21 to 24 feet below the existing roadway surface, with elevations ranging from about 33.4 to 37.4 feet.

Either bedrock or boulder was encountered beneath the glacial till in each of the other borings drilled in the roadway behind the abutments (B102, B104, B106, and B108) at depths ranging from about 15 to 21 feet below the existing roadway surface (elevations ranging from 36.5 to 42.5 feet). Bedrock coring was not performed to confirm that bedrock was encountered in these borings. However, the borings were advanced about 1.6 to 4 feet below the surface of the bedrock or boulder using a tricone roller bit. The elevations at which bedrock or boulders were encountered in these borings compare well with the elevations of bedrock encountered in the borings in which bedrock was cored. Therefore, it is likely that bedrock was encountered in the borings in which bedrock was not cored. Assuming this is true, bedrock appears to slope downwards towards the river at both abutments and generally from upstream to downstream.

The bedrock observed in the core samples consists of gray, fine grained meta-sedimentary rock with quartz veins and intrusions. Geologic mapping of the area indicates that the bedrock is probably phyllite or quartzite of the Eliot formation, which is consistent with the rock observed in the core samples. Foliations in the bedrock are steep, ranging from about 45° to 90°.

Except at B103, the rock observed in the core samples is generally fresh to slightly weathered, with joints dipping from about 0° to 30° and 45° to 90° (usually along foliations) at spacing ranging from ½ to 16 inches, and Rock Quality Designations (RQDs) ranging from 52% to 75%. The rock in the two cores obtained from the upper approximately 9 feet of bedrock in B103 (B103-C1 and -C2) is moderately to highly weathered, and of the 7.3 feet of core drilled, only 2.5 feet were recovered. The joints in these core samples dip from about 0° to 20° and about 80° to 90° (along foliations), at spacings ranging from about ½ inch to 4 inches. The RQDs of these two core samples were 0%. The third rock core drilled from about 9 to 14 feet below the bedrock surface in B103 (B103-C3) was observed to be less fractured and weathered. The joints in this core sample dip approximately horizontal and at about 70° to 90°, at spacings ranging from ½ to 5.5 inches. The RQD of this core sample is about 23%.

Groundwater – Groundwater observation wells were not installed in the borings for the measurement of stabilized groundwater levels. Also, the borings were drilled using cased wash boring techniques, in which water is introduced into the borehole to flush drill cuttings. Therefore, estimation of groundwater levels based on water levels in the boreholes upon completion of drilling and sample moisture conditions would not be reliable. Based on site observations, we expect that the groundwater level at the boring locations is typically about 2 to 4 feet above the water level in the river. However, it is likely that groundwater periodically becomes perched on the sandy silt/silty sand layer (which is typically less permeable than the overlying fill) as water infiltrates down to the groundwater level.

Our groundwater level evaluation is approximate and represents the conditions at the time the borings were drilled. It should be noted that groundwater levels typically fluctuate with seasonal variations in precipitation and infiltration conditions, and may differ at other times of the year.

Approach Roadway (B110 through B113)

Pavement – The asphalt pavement at the borings drilled in the approach roadway (B110 through B113) ranges from about 4.5 to 5 inches thick.

Fill (Unified Soil Classification SW, SM) – Fill was encountered beneath the pavement in all of the borings drilled in approach roadway (B110 through B113). The fill in these borings extends to depths ranging from of about 1.5 to 4 feet below the roadway surface. In B110 through B112, the fill consists of a 1.5- to 2.5-foot-thick layer of sand with gravel pavement base soil. At B113, the fill includes an approximately 3-foot-thick layer of sand with gravel pavement base soil underlain by an approximately 1-foot-thick layer of silty sand with gravel. A boulder was encountered within the fill layer in B113. Reliable SPT N-values were not measured in the fill because the upper split-spoon sample (0.5 to 2.5-foot depth interval) was obtained using a nonstandard, 3-inch-diameter split-spoon driven using a 300 pound hammer with a 24-inch drop. The second split-spoon sample at B113 was obtained using a standard split-spoon sampler, but was probably obstructed by a boulder and the SPT N-value is probably not representative of the soil's relative density.

Sandy Silt, Clayey Silt, and Silty Sand (ML, SM) – A layer of sandy silt, clayey silt, and silty sand was encountered beneath the fill in all of the borings drilled in the approach roadway. This layer extends to depths of at least 5 to 6.5 feet, where the borings were terminated. Most of the samples obtained from the layer were observed to be stratified, similar to those obtained from the layer observed in the borings drilled near the bridge abutments. However, samples obtained from the layer in B111 and B112 contain thin varves, or laminations, of silt and clay with a few sand partings. Cobbles or boulders were encountered in the layer in B110 (rock fragments in B110-S3) and B113 (refusals in B113-S3 and S-4). SPT N-values in the layer typically ranged from 7 to 38 blows per foot, indicating that the soil is loose to dense.

Groundwater – Groundwater observation wells were not installed in the borings for the measurement of stabilized groundwater levels. However, the borings were drilled using hollow-stem augers, and the moisture conditions of soil samples provides some indication of groundwater levels. The lower portions of the silty sand samples in B110-S3 and B113-S3 were observed to be wet, indicating the presence of groundwater at depths of about 6 feet and 4.5 feet, respectively, below the pavement surface. However, it is likely that water infiltrating through the fill periodically becomes perched on the silty and clayey natural soils, which are much less permeable than the overlying fill.

Our groundwater level evaluation is approximate and represents the conditions at the time the borings were drilled. It should be noted that groundwater levels typically fluctuate with seasonal variations in precipitation and infiltration conditions, and may differ at other times of the year.

Pier (1962 Borings and Probes)

No borings were or probes were drilled in the vicinity of the center pier for this investigation. However, logs for the borings and probes drilled for the 1962 design of the previous bridge superstructure replacement indicate that bedrock was encountered beneath an approximately 0 to 9.9-foot-thick layer of overburden (described as till and nested cobbles and boulders) at elevations ranging from 26.4 to 40.6 feet. The logs for the 1962 borings do not provide much information concerning rock quality, but the descriptions of the rock core samples obtained from the 1962 borings (soft, fractured, and broken gray schist with fractures dipping 80°) are consistent with the rock core samples obtained from the borings drilled for this investigation.

BRIDGE FOUNDATION DESIGN AND CONSTRUCTION RECOMMENDATIONS

Introduction

Our recommendations for geotechnical aspects of the design of the new bridge are based on Load and Resistance Factor Design (LRFD) methodology. These recommendations were developed in general accordance with the AASHTO LRFD Bridge Design Specifications, Interim 2010 (AASHTO Specifications), and the Federal Highway Administration publication *Geotechnical Engineering Circular No. 6, Shallow Foundations*, FHWA-SA-02-054, September 2002. Note that this report was prepared during the preliminary design phase of the project, and information concerning loads, abutment and wing wall locations, and other bridge design details were not yet available. Therefore, several assumptions had to be made in the development of our recommendations. We recommend that WGC be retained during final design to check that our assumptions were reasonable.

Foundation Design

Abutments and wing walls should be supported on cast-in-place concrete spread footings bearing either on competent bedrock, or within the silty sand with gravel layer (glacial till) that was encountered below the existing fill and the sandy silt and silty sand layer in the borings. If a new or modified pier will be constructed as an intermediate support, the pier should bear on competent bedrock. Footings bearing within the glacial till layer should be underlain by a minimum 12-inch-thick layer of compacted structural fill. The footings should bear at least 4 feet below finished grade at the toes of the walls to provide frost protection. It may be necessary to embed the footings deeper than 4 feet to provide scour protection. Evaluation of scour depth and scour protection was not included in our scope of services. Riprap or other means of scour protection also should be considered.

The determination of whether abutment or wing wall footings should bear on bedrock or glacial till will depend largely on the anticipated scour depth (to be determined by others) and the depth to bedrock below finished grade at the location of the abutment or wing wall. Footing subgrade elevations should be selected such that the each footing bears entirely on either bedrock or glacial till. A footing should not bear on both bedrock and glacial till due to the potential for differential settlement and cracking. It should be noted that the bedrock surface elevations are likely to vary between borings. Therefore, if a planned footing subgrade is near the expected interface between the glacial till layer and the bedrock surface (i.e., not well above or well below the bedrock surface), the planned footing subgrade could be above the bedrock surface in some areas and below the bedrock surface in others. If this is the case, we recommend that the footings be underlain by minimum 12-inch-thick layer of compacted structural fill and be designed for soil bearing conditions. Then, in areas where bedrock is encountered above the planned subgrade elevation for the structural fill, it should be overexcavated to allow placement of the structural fill layer. The structural fill will provide a cushion between the bedrock and footings and reduce the potential for cracking due to differential settlement.

If a new or modified pier is constructed, it should be supported on a footing cast on competent bedrock. All portions of the pier footing should bear directly on bedrock.

Design bearing pressures for footings founded on bedrock and glacial till are provided below.

<u>Footings on Glacial Till</u>

Bearing capacity and settlement analyses were conducted to determine nominal bearing resistance for the strength and service limit states as a function of the effective footing width (B'_f) for footings bearing within the glacial till layer. The effective footing width is the portion of an eccentrically loaded footing over which an equivalent uniform pressure is applied for the purpose of analysis. The effective footing width is defined as follows:

$$B'_f = B_f - 2e$$

where: B_f = actual footing width
e = eccentricity

Eccentricity (e) is the distance from the resultant vertical force to the center of the footing, as determined by overturning stability analysis. The AASHTO Specifications indicate that eccentricity should be no greater than $B_f/4$. If this condition is satisfied, the effective footing width will be at least half of the actual footing width.

We recommend that the footings be designed based on the following bearing pressures:

- The nominal bearing resistance for the strength and extreme limit state conditions should be the ultimate bearing capacity calculated as follows:

  $$q_{ult} = 12.2 + 2.6B'_f$$

  where: q_{ult} = ultimate bearing capacity, kips per square foot (ksf)
  B'_f = effective footing width, feet

  Since the strength of the soil subgrade was estimated based on SPT data, a resistance factor (φ) of 0.45 should be applied.

- A nominal bearing resistance of 10 ksf should be used for the service limit state condition. This is based on settlement analyses conducted assuming that the effective footing widths (B'_f) for abutment and wing walls would fall within the range of 6 to 15 feet. Settlements for footings with effective footing widths ranging from 6 to 15 feet and designed for a nominal bearing resistance of 10 ksf are expected to be less than about ¾ inch.

We recognize that the strength limit state nominal bearing resistance value provided above is conservative for footings for which the distances from the bedrock surface to the bottoms of the footings are less than the effective footing widths. In these areas, the shallow bedrock will restrict the development of general shear failure surfaces in the soil (on which bearing capacity analysis is based). However, we expect that footing widths will be controlled by the requirement to satisfy eccentricity limits (AASHTO Specifications require the resultant force at the strength limit state to be within middle half of the footing width) rather than bearing resistance values. The actual bearing pressures for both the service and strength limit states are expected to be less than the nominal bearing resistance values provided above.

Resistance to sliding should be based only on friction along the bottoms of the footings, neglecting passive pressure at the toes of the footings. For concrete footings cast on a 12-inch-thick layer of compacted structural fill placed on the glacial till bearing layer, the nominal (or ultimate) sliding resistance for the strength limit state condition should be calculated as follows:

$$Q_T = 0.78P_V$$

where: Q_T = ultimate sliding resistance
P_V = vertical load on the footing

A resistance factor (φ) of 0.8 should be applied for cast-in-place concrete footings.

Footings on Bedrock

Footings designed to bear on bedrock should be cast directly on competent bedrock. The bearing surface should be free of all soil and weathered and fractured bedrock that can be dislodged using an excavator bucket.

The meta-sedimentary bedrock observed in most of the core samples is fresh to slightly weathered and slightly fractured to sound, with RQD values ranging from about 52% to 75%. However, core samples obtained from the upper approximately 9 feet of bedrock in B103 were found to be moderately to severely weathered and moderately fractured, with an RQD value of 0%. A subsequent core sample obtained from 9 to 14 feet below the bedrock surface in B103 is fresh to slightly weathered and moderately to slightly fractured, with an RQD value of 23%. Since it would be impractical to remove all of the fractured and weathered bedrock from this area, we recommend that the all footings be designed for conservative bearing resistance values that account for the poor condition of the rock observed at B103. That said, weathered and fractured bedrock that can be dislodged using an excavator bucket should be removed from the bearing surface (as indicated above). Also, the weathered and fractured bedrock that would remain under the footing could be susceptible to scour, and the footings should be embedded below the anticipated scour depth (to be determined by others).

Footings bearing on bedrock should be designed using a service limit state nominal bearing resistance of 20 ksf. Settlements of footings bearing on bedrock are expected to be less than ½ inch.

Footings bearing on bedrock should be designed using a strength state nominal bearing resistance of 60 ksf. A resistance factor (φ) of 0.45 should be used.

Although the service and strength limit state bearing resistances provided above are conservative, it is likely that the footing widths will be controlled by the requirement to satisfy eccentricity limits, rather than nominal bearing resistance. The actual bearing pressures for both the service and strength limit states are expected to be less than the nominal bearing resistance values provided above.

Resistance to sliding should be based only on friction along the bottoms of the footings, neglecting passive pressure at the toes of the footings. For concrete footings cast on competent bedrock, the nominal (or ultimate) sliding resistance for the strength limit state condition should be calculated as follows:

$$Q_T = 0.78P_V$$

where: Q_T = ultimate sliding resistance
P_V = vertical load on the footing

A resistance factor (φ) of 0.8 should be applied for cast-in-place concrete footings.

Seismic Parameters

Based on the results of the borings, the site is in Site Class C and Seismic Zone 1, per the AASHTO Specifications. Seismic acceleration coefficients, modified by site factors per the AASHTO Specifications, are as follows:

$As = 0.123$
$S_{DS} = 0.234$
$S_{D1} = 0.077$

Abutment and Wing Walls

We understand that new abutment and wing walls would probably be cast-in-place concrete cantilever walls. We assume that the walls will have approximately horizontal backfill surfaces.

Drains

Drains should be installed behind the abutment and wing walls. The drains should consist of minimum 4-inch-diameter weeps placed at maximum spacings of 6 feet. A minimum 16-

inch-wide by 16-inch-thick zone of crushed stone should be placed along the entire lengths of the walls behind the weeps. The crushed stone should meet the requirements of No. 67 Stone, Section 703 of the New Hampshire Department of Transportation Standard Specifications for Road and Bridge Construction, 2010 (NHDOT Specifications). The stone should be completely separated from the backfill and in situ soils by a nonwoven, needle-punched medium strength geotextile, item 593.121 of the NHDOT Specifications. The inverts of the weeps should be no more than 1 to 2 feet above the normal high water level of the adjacent section of the river. The weep holes should be screened to retain the crushed stone (maximum ½ inch square openings) and to prevent entry by animals, unless the weeps are below the finished grade in front of the wall. If the weeps outlet below the finished grade at the front of the wall, a zone of crushed stone surrounded by geotextile, similar to that recommended for behind the wall, should be placed against the wall in front of the weeps. The crushed stone in front of the walls should discharge directly to riprap placed at the toes of the walls.

Earth and Surcharge Pressures

Abutment or wing walls supported on footings bearing on the glacial till are expected to be free to rotate or displace a sufficient amount to mobilize active earth pressure. Active earth pressure should be estimated using an equivalent fluid pressure (slope of the earth pressure diagram) of 35 pounds per cubic foot (pcf) applied from finished grade behind the wall to the level of the weeps (or the design water level in the river, whichever is higher). Below the level of the weeps (or design water level), the equivalent fluid pressure (slope of the earth pressure diagram) should be decreased to 17.5 pcf to account for effective stresses. Unless approach slabs are provided, design of the abutment walls should include a uniform traffic surcharge pressure of 70 pounds per square foot (psf), which represents an equivalent 2 feet of soil (based on assumed overall wall height greater than 20 feet). This surcharge pressure should also be included in the design of wing walls if traffic can pass within a distance from the backs of the walls equal to half of the overall wall height. Passive earth pressure at the toes of the walls should be neglected in the design.

Abutment or wing walls supported on footings bearing on bedrock are not expected to rotate or displace a sufficient amount to mobilize active earth pressure and should be designed for at rest earth pressure. At rest earth pressure should be estimated using an equivalent fluid pressure (slope of the earth pressure diagram) of 55 pcf applied from finished grade behind the wall to the level of the weeps (or the design water level in the river, whichever is higher). Below the level of the weeps (or design water level), the equivalent fluid pressure (slope of the earth pressure diagram) should be decreased to 27.5 pcf to account for effective stresses. Unless approach slabs are provided, design of the abutment walls should include a uniform traffic surcharge pressure of 110 psf, which represents an equivalent 2 feet of soil (based on assumed overall wall height greater than 20 feet). This surcharge pressure should be included in the design of wing walls if traffic can pass within a distance from the backs of the walls equal to half of the overall wall height. Passive earth pressure at the toes of the walls should be neglected in the design.

The stability analysis and design of the walls should also include hydrostatic water pressures behind, in front of, and beneath the walls. The hydrostatic pressure behind the wall should be estimated assuming the water level is at the inverts of the weep holes, or the design water level in the river, whichever is higher. The hydrostatic water pressure in front of the wall should be estimated using the design water level in the river. The hydrostatic uplift pressure acting on the bottoms of the footings should be estimated assuming the water level varies linearly from the water level at the back of the wall to the water level at the front of the wall. Since hydrostatic uplift pressure will be applied at the bottoms of the footings, the total unit weight of the concrete walls (150 psf) and the soil above the wall footings (125 pcf) should be used in the stability analyses.

Backfill and Compaction

All fill placed behind the concrete abutment and wing walls, to a distance of at least 1 foot behind the heels of the wall footings, and beneath approach slabs should consist of Granular Backfill (Bridge), item 209.201 of the NHDOT Specifications. The backfill should be placed and compacted in maximum 8-inch-thick loose lifts.

Structural fill placed beneath the footings bearing on the glacial till should meet the requirements for Crushed Gravel for Structural Fill, item 508 of the NHDOT Specifications. The fill should be placed and compacted in maximum 8-inch-thick loose lifts. Clean Stone Fill for Structural Fill, per item 508 of the NHDOT Specifications, may be used beneath footings in lieu of the Crushed Gravel for Structural Fill. If Clean Stone Fill for Structural Fill is used, it should be completely separated from soil subgrades and backfill soils by a nonwoven, needle-punched medium strength geotextile, item 593.121 of the NHDOT Specifications.

Embankment fill placed beyond the limits of Granular Backfill (Bridge) should be a granular material consisting primarily of sand or sand with gravel with no more than 35% fines, and free of organic soils, construction debris, clumps of silt and clay, stones greater than 6 inches in diameter, and other deleterious materials. We expect that some of the soil excavated for wall construction will be suitable for use in the new embankment provided oversized stones and organic soils and materials are removed. However, sandy silt excavated from behind the existing abutment and wing walls is not suitable for use as embankment fill. The fill should be placed and compacted in maximum 12-inch-thick loose lifts.

Backfill should be compacted to the following criteria:

- Structural fill placed below footings should be compacted to at least 98% of maximum dry density as determined in accordance with AASHTO T 99.

- Backfill placed under approach slabs or within 10 feet of the backs of structures without approach slabs should be compacted to at least 98% of maximum dry density (AASHTO T 99).

- All other backfill materials should be compacted to at least 95% of maximum dry density (AASHTO T 99).

Heavy compaction equipment (such as vibratory rollers) should not be operated within a distance from the back of a wall equal to the half the overall wall height. Fill placement and compaction should be performed simultaneously on both sides of the walls to avoid excessive differential earth pressures.

The lift thicknesses specified above should be considered maximum values, assuming a large vibratory roller (imparting at least 30 kips combined static and dynamic force) is used. Thinner lifts will probably be needed to attain uniform compaction where smaller compaction equipment (such as a vibratory plates or mechanical tampers) is used.

River Diversion, Temporary Dewatering, and Excavation Support

Construction of the new bridge structure will require excavation of up to about 25 feet or more below the existing roadway. The excavation subgrade for abutment footings will be at least 4 feet below the bottom of the river.

Cofferdams will be needed to divert the river from areas of excavation for construction of the abutment and wing walls. For construction of the new abutment and wing walls, it may be possible to use sheet pile cofferdams, which could also be used to support the excavations and partially cutoff seepage. However, pre-excavation might be needed in some areas to clear boulders that could obstruct sheet pile driving. Moreover, due to the shallow bedrock, sheet piles might not have sufficient toe depth below the bottom of wall footing excavations to provide stability of the sheet pile cofferdam, and low levels of bracing or tiebacks might be needed. Other types of cofferdams that could be used (in conjunction with open cut excavation) to exclude surface water from the wall excavations include earthen and sand bag cofferdams, as well as proprietary cofferdams such as Port-A-Dam. Depending on the locations of the new abutment and wing walls, it also might be possible to utilize the existing abutment and wing walls to divert river water from the excavations.

A cofferdam will also be needed to divert the river from the excavation for a new or modified intermediate pier (if constructed). A sheet pile cofferdam is probably not feasible at the pier location due to insufficient overburden thickness, and an earthen or sand bag cofferdam, or a proprietary cofferdam system (such as Port-a-Dam), might be necessary.

Dewatering requirements will depend on several factors, including excavation depth, groundwater and river levels at the time of construction, and the effectiveness of sheet pile cofferdams (if used) in partially cutting off seepage. Dewatering requirements will also depend on whether the footings are supported on bedrock or on glacial till subgrade. If the footings are supported on glacial till subgrade, careful dewatering will be necessary to reduce upward seepage pressures that could disturb the soils beneath the bearing subgrade and threaten the excavation stability. Well points or deep wells might be needed to dewater and depressurize the soils below the excavation subgrades. The contract specifications should require the contractor to lower the piezometric water level in the soils below the excavation to at least 2 foot below the excavation subgrade.

Water that is intercepted by the dewatering system should be discharged in accordance with local, state, and federal requirements.

Earth support systems/cofferdams should be designed by a professional engineer licensed in New Hampshire and experienced with this type of work. All excavations should comply with OSHA regulations. Open cut excavations should have side slopes no steeper than 1.5H:1V (assuming the excavations are properly dewatered).

Preparation and Maintenance of Footing Subgrades

Excavation of the final 2 feet above the soil subgrades for the footings bearing on the glacial till should be performed using a smooth edged bucket. All loose, soft, or disturbed soils should be removed from the subgrade. Proof rolling of the subgrade with a vibratory compactor should be performed unless it causes “pumping” and disturbance of the subgrade. The period of time that the soil subgrades are left exposed should be minimized to reduce the risk of subgrade softening and disturbance. If overexcavation of the subgrade is necessary to remove disturbed soils, the overexcavation should be backfilled with compacted structural fill.

Bedrock excavation will be required for footings bearing on bedrock. We expect that mechanical rock removal methods (such as an excavator-mounted jack hammer) and/or blasting will be necessary. Care must be taken to limit overbreak or shattering of the bedrock below the planned subgrade elevation. All loose soil and fractured or weathered rock that can be dislodged using an excavator bucket should be removed from bedrock subgrades.

Soil and bedrock subgrades should be free of standing water, frost, and loose soil before placement of geotextile and/or structural fill.

Construction Vibration Monitoring

Construction vibrations caused by bedrock removal and fill compaction could cause densification of loose soils in the vicinity of the project. If loose soils underlie utilities or structures in the project area, vibration-induced densification of these loose soils could lead to settlement and damage. The contractor should be required to engage a geotechnical engineering and/or vibration monitoring firm to monitor construction vibrations at nearby structures and set vibration limits to inhibit vibration damage. Preconstruction surveys should also be conducted to document the existing condition of nearby structures so that claims of vibration damage can be assessed.

Freezing Conditions

During freezing conditions, additional care must be exercised during construction to prevent disturbance of the soil subgrades and to achieve the required degree of fill compaction. The subgrades and each lift of backfill must be compacted before the water in the subgrade or backfill can freeze.

Frozen material should not be placed as backfill, nor should backfill, foundations, pavements, or slabs be placed on frozen soil. If, during construction, the top layer of soil becomes frozen, the frozen soil should be removed before backfill, foundations, pavements, or slabs are placed on it.

When the air temperature is below 25° F the contractor should not be allowed to place fill or expose final subgrades unless special procedures, approved by the geotechnical engineer, are used to prevent freezing. If footings are built and left exposed during the winter season, precautions should be implemented to prevent damage due to frost heave.

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 structure. We cannot accept responsibility for designs based on our recommendations unless we are engaged to review the final plans and specifications to determine whether any 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 in subsurface conditions 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 WGC be engaged to make site visits during construction to:

1. Check that the subsurface conditions exposed during construction are in general conformance with our design assumptions.

2. Ascertain that, in general, the work is being performed in compliance with the contract documents and our recommendations.

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

We appreciate the opportunity to work with you on this project. Please call if you have any questions.

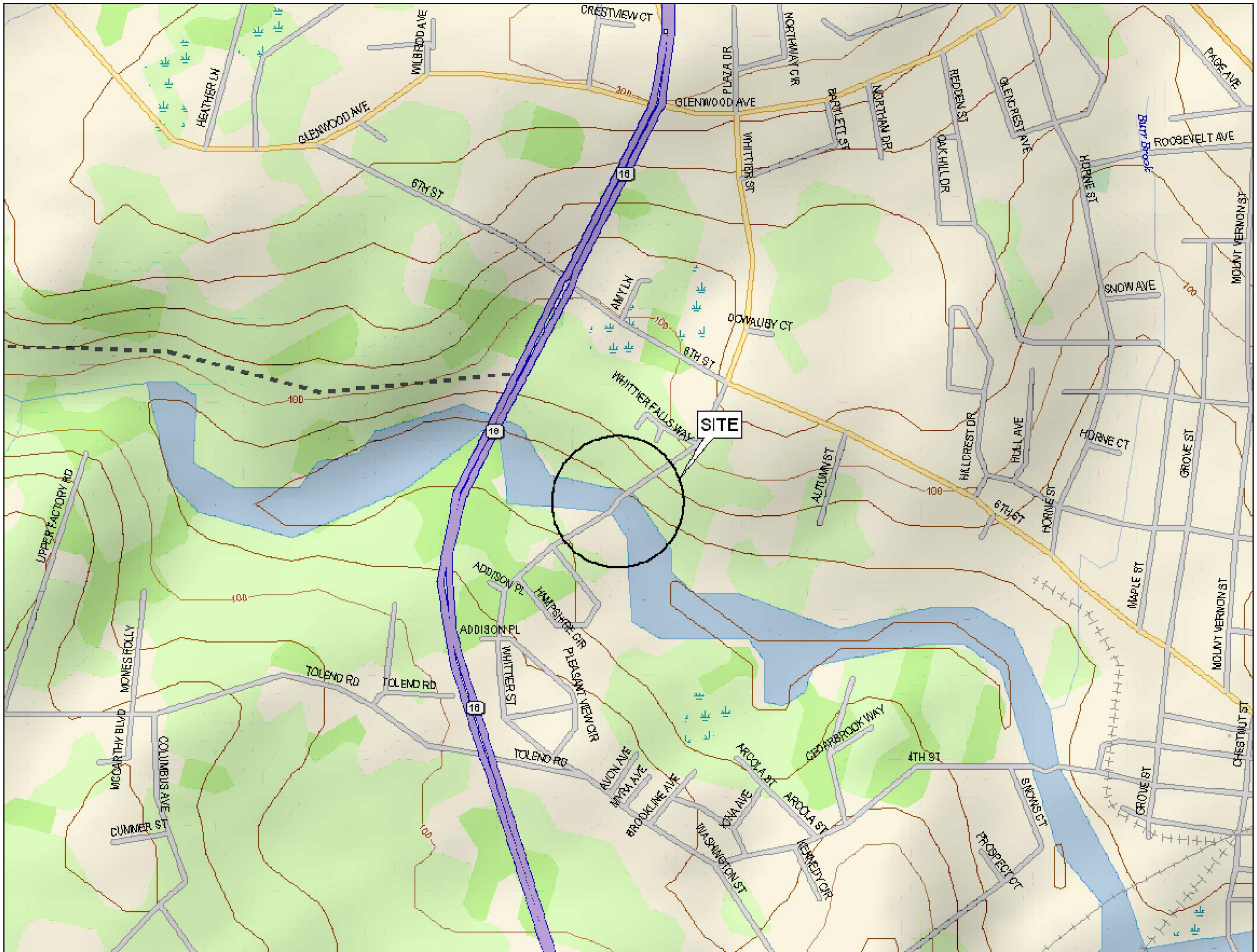
Sincerely,

Ward Geotechnical Consulting, PLLC

Craig F. Ward, P.E.
Principal

Figures 1 through 4B
Appendices A & B

CFW



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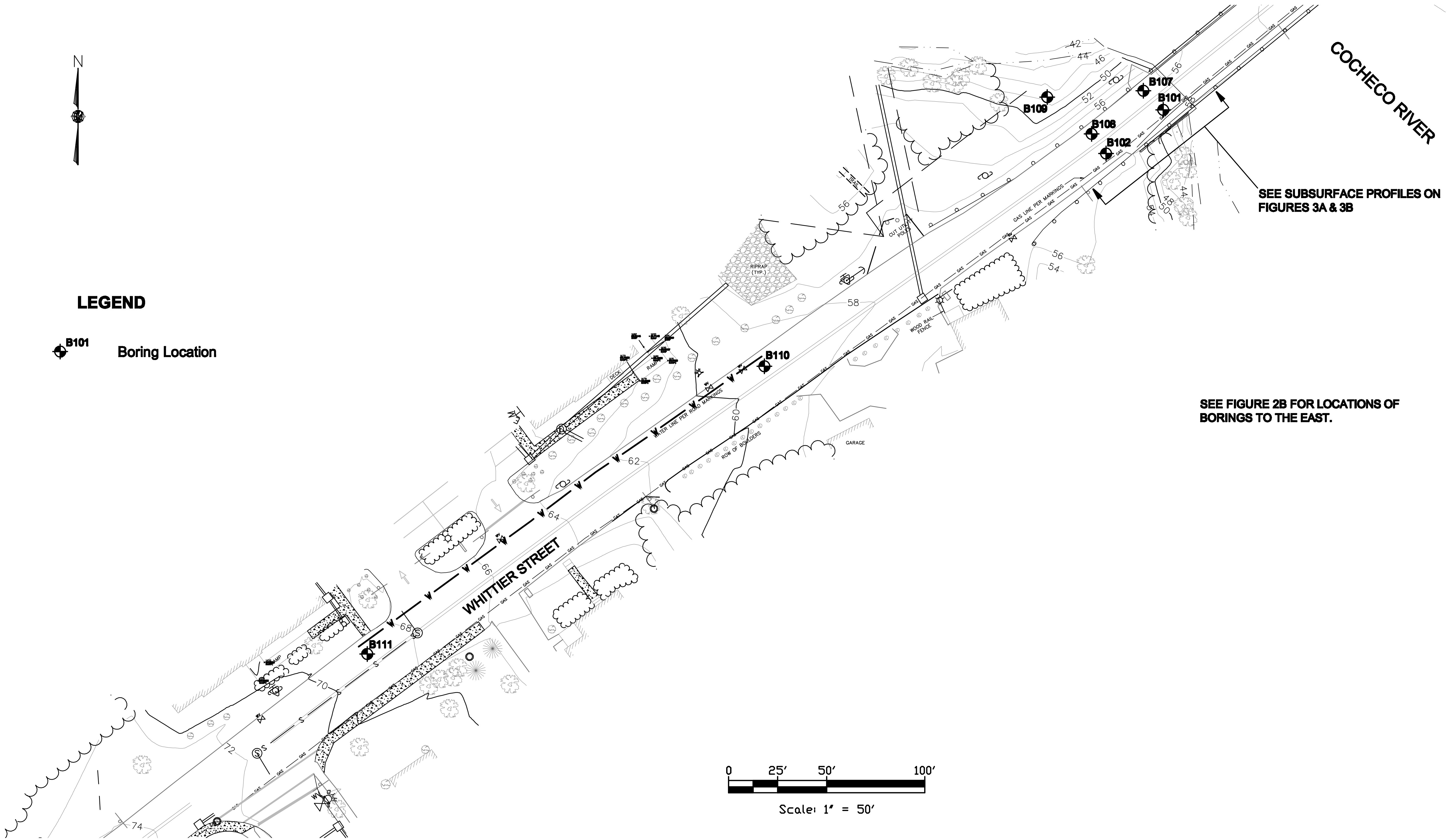
MN (15.4° W)

0 500 1000 1500 ft
Data Zoom 14-2

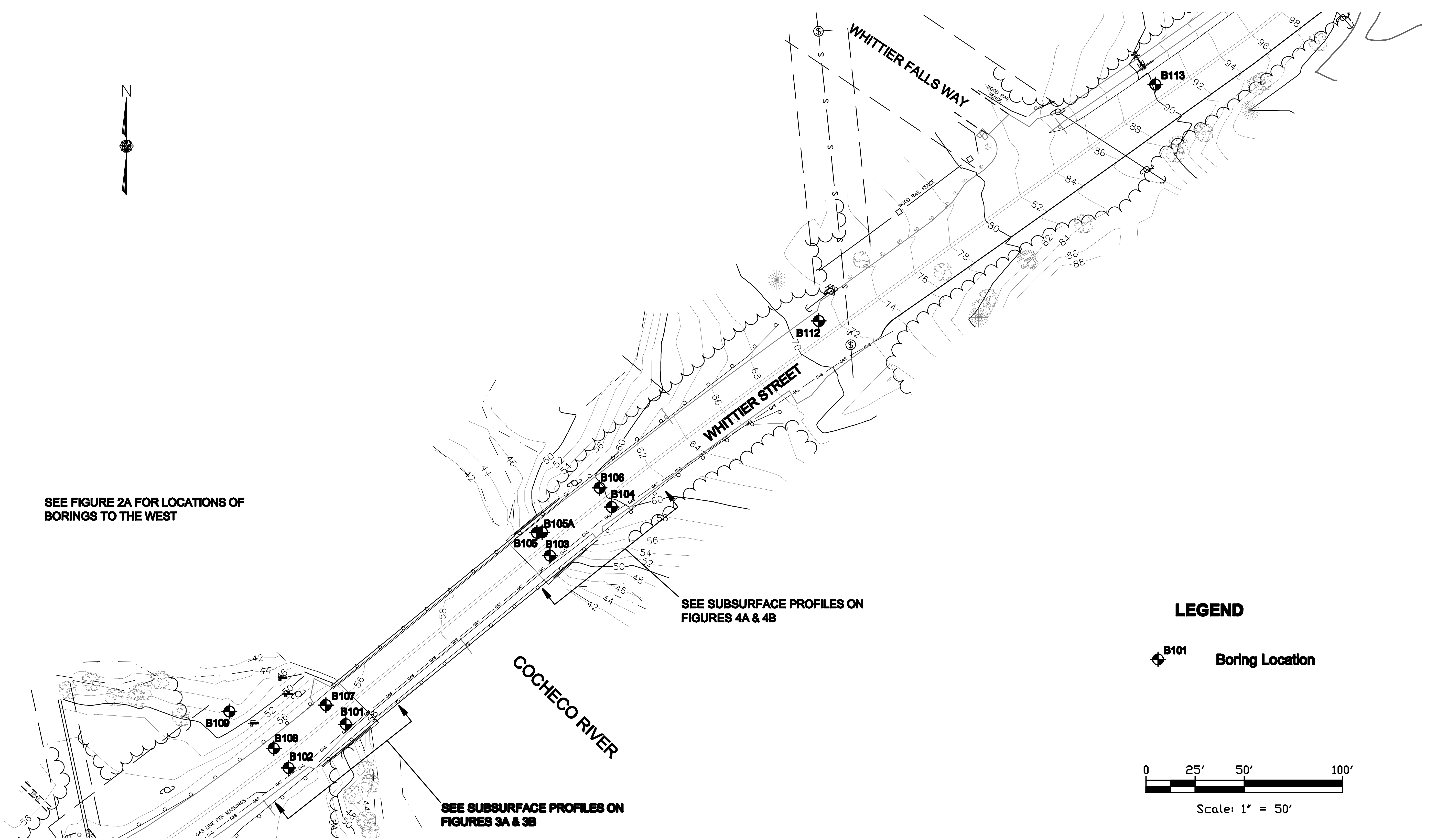
The Louis Berger Group, Inc. Manchester, New Hampshire	WHITTIER STREET BRIDGE OVER COCHECO RIVER DOVER, NEW HAMPSHIRE	SITE LOCATION MAP
Ward Geotechnical Consulting, PLLC	WGC Project 11210	August 2011 Figure 1



Notes:

1. The borings were drilled by New Hampshire Boring, Inc. and observed by Ward Geotechnical Consulting, PLLC on April 27 through May 3, 2011.
2. Base plan was provided by the Louis Berger Group, Inc. in AutoCAD format for our use in preparing this boring location plan. Elevations are referenced to the National Geodetic Vertical Datum of 1929.

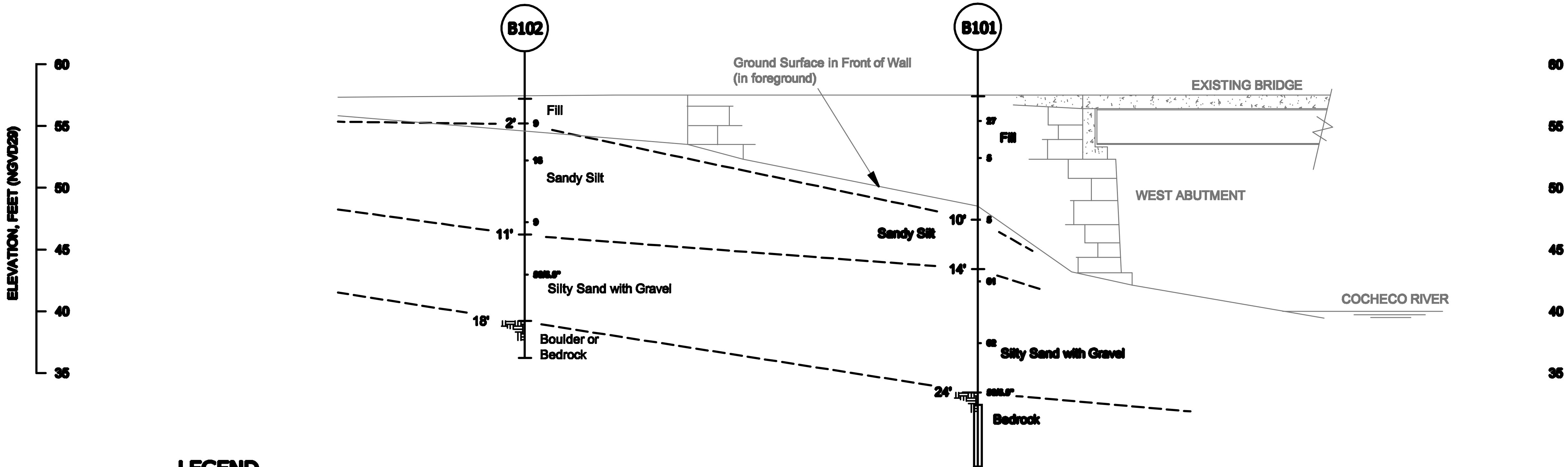
The Louis Berger Group, Inc. Manchester, New Hampshire	WHITTIER STREET BRIDGE DOVER, NEW HAMPSHIRE	BORING LOCATION PLAN - WEST
Ward Geotechnical Consulting, PLLC	WGC Project 11210	August 2011 Figure 2A



Notes:

1. The borings were drilled by New Hampshire Boring, Inc. and observed by Ward Geotechnical Consulting, PLLC on April 27 through May 3, 2011.
2. Base plan was provided by the Louis Berger Group, Inc. in AutoCAD format for our use in preparing this boring location plan. Elevations are referenced to the National Geodetic Vertical Datum of 1929.

The Louis Berger Group, Inc. Manchester, New Hampshire	WHITTIER STREET BRIDGE DOVER, NEW HAMPSHIRE	BORING LOCATION PLAN - EAST
Ward Geotechnical Consulting, PLLC	WGC Project 11210	August 2011 Figure 2B



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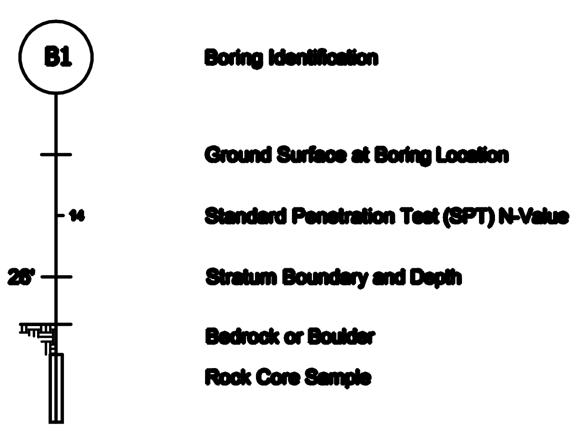
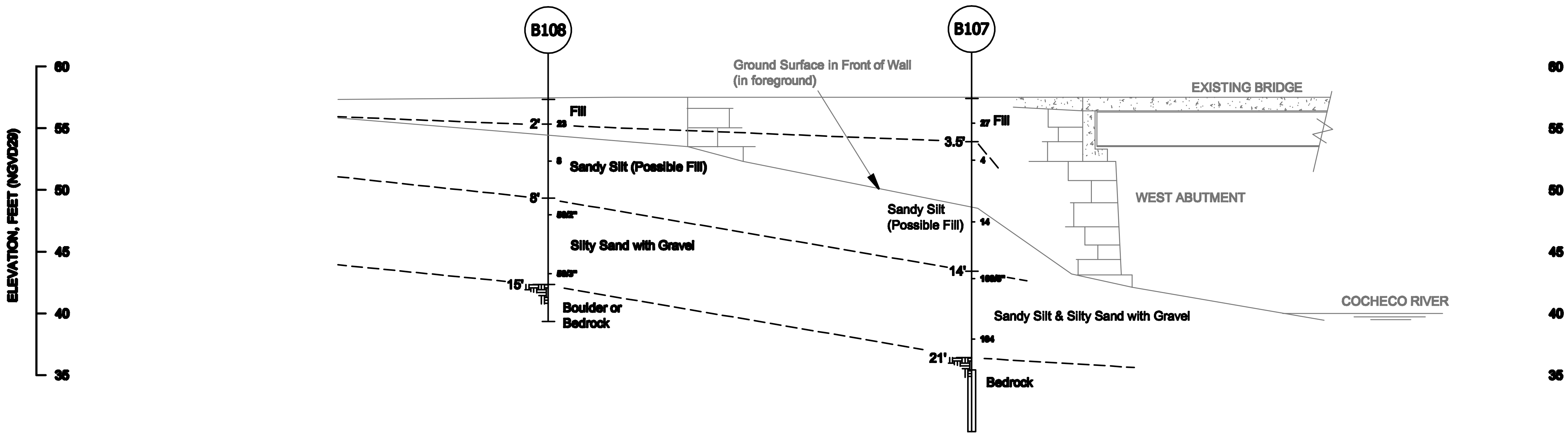


Notes:

1. Boring locations are shown on the Boring Location Plan on Figures 2A and 2B.
2. The subsurface profile was developed based on the base plan provided by the Louis Berger Group, Inc. The confuguration of the existing abutment is approximate.

The Louis Berger Group, Inc. Manchester, New Hampshire	WHITTIER STREET BRIDGE DOVER, NEW HAMPSHIRE	SUBSURFACE PROFILE - WEST ABUTMENT (B101 & B102)
Ward Geotechnical Consulting, PLLC	WGC Project 11210	August 2011 Figure 3A



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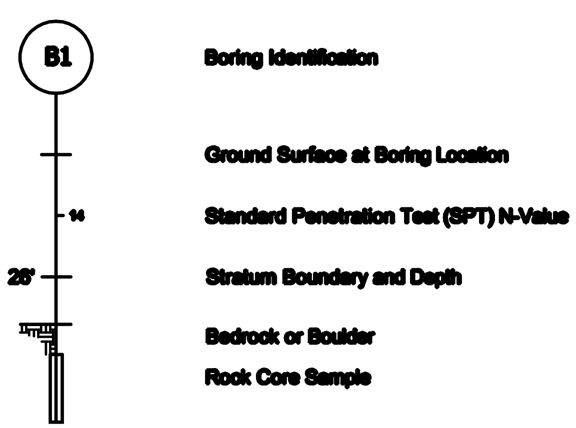
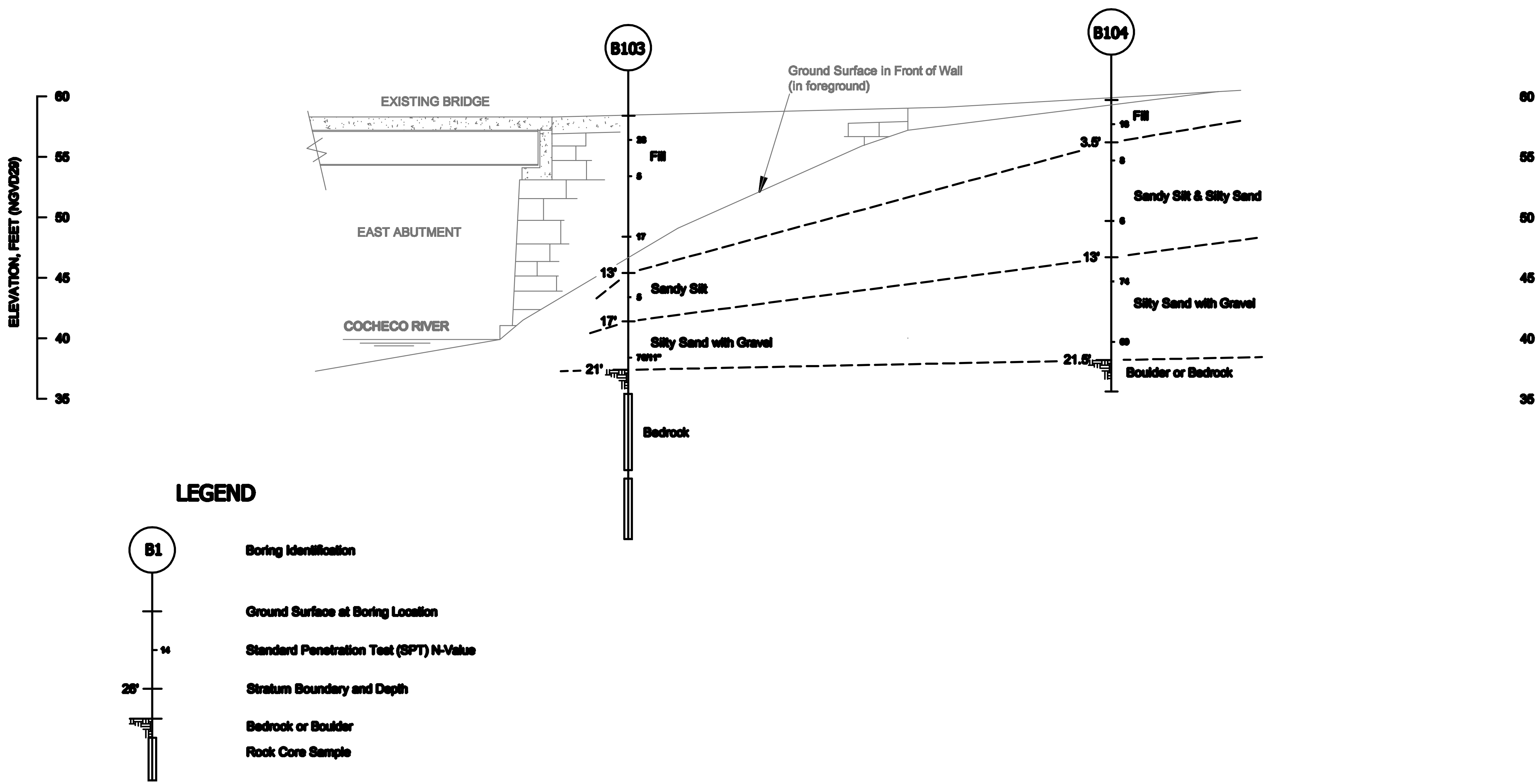


Notes:

1. Boring locations are shown on the Boring Location Plan on Figures 2A and 2B.
2. The subsurface profile was developed based on the base plan provided by the Louis Berger Group, Inc. The confuguration of the existing abutment is approximate.

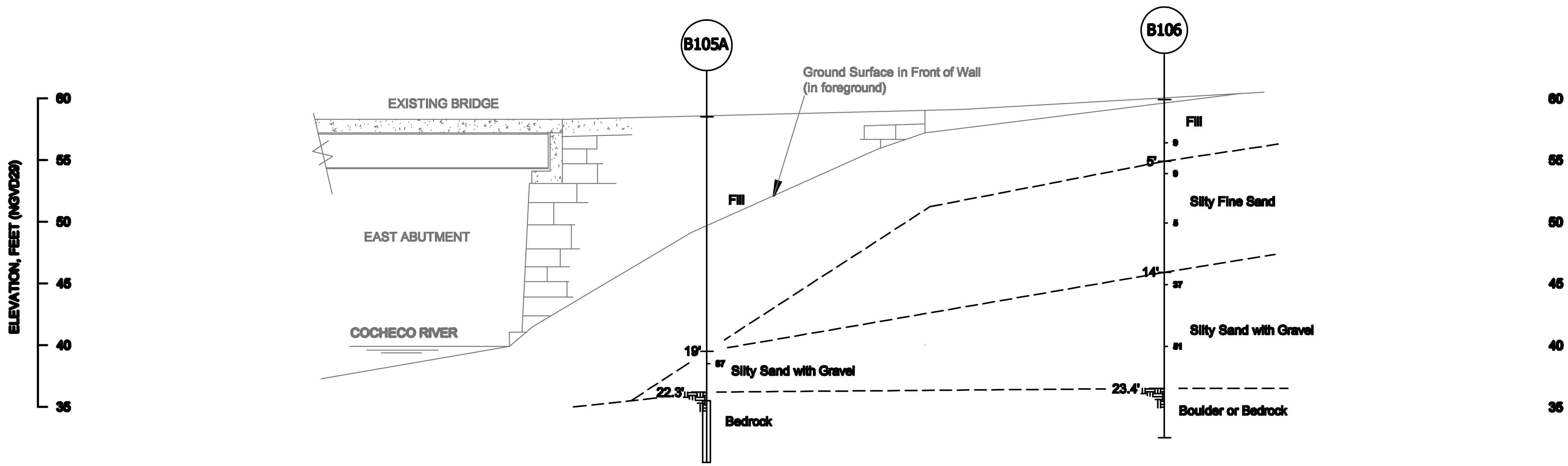
The Louis Berger Group, Inc. Manchester, New Hampshire	WHITTIER STREET BRIDGE DOVER, NEW HAMPSHIRE	SUBSURFACE PROFILE - WEST ABUTMENT (B107 & B108)
Ward Geotechnical Consulting, PLLC	WGC Project 11210	August 2011 Figure 3B



Notes:

1. Boring locations are shown on the Boring Location Plan on Figures 2A and 2B.
2. The subsurface profile was developed based on the base plan provided by the Louis Berger Group, Inc. The confuguration of the existing abutment is approximate.

The Louis Berger Group, Inc. Manchester, New Hampshire	WHITTIER STREET BRIDGE DOVER, NEW HAMPSHIRE	SUBSURFACE PROFILE - EAST ABUTMENT (B103 & B104)
Ward Geotechnical Consulting, PLLC	WGC Project 11210	August 2011 Figure 4A



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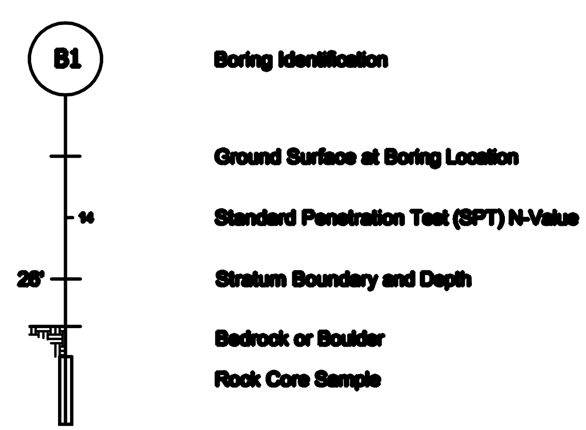


Notes:

1. Boring locations are shown on the Boring Location Plan on Figures 2A and 2B.
2. The subsurface profile was developed based on the base plan provided by the Louis Berger Group, Inc. The confuguration of the existing abutment is approximate.

The Louis Berger Group, Inc. Manchester, New Hampshire	WHITTIER STREET BRIDGE DOVER, NEW HAMPSHIRE	SUBSURFACE PROFILE - EAST ABUTMENT (B105A & B106)
Ward Geotechnical Consulting, PLLC	WGC Project 11210	August 2011 Figure 4B

Appendix A – Boring Logs

Ward Geotechnical Consulting, PLLC	Project:	Whittier Street Bridge	Boring Log
	Location:	Dover, New Hampshire	**B101**
	Client:	The Louis Berger Group, Inc.	
	Project No.:	11210	

Contractor:	New Hampshire Boring, Inc.	Groundwater Depth:	Date:	Page 1 of 2
Logged By:	Craig Ward			
Drilling Dates:	4/27/2011	GS Elevation: 57.4 feet	Boring Location:	
Drill Rig:	Mobile B-47 Truck	Datum: NGVD29	west abutment - eastbound lane	

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	SOIL AND ROCK DESCRIPTIONS	
					4" Case & Wash	7.5" Asphalt Pavement	Fill
	S1	23-17 10-8	24	12		S1: upper 10": **Sand with Gravel (SW)** - fine to medium (some coarse) sand, 20%-30% subangular gravel to 1", bown. lower 2": **Silty Sand (SM)** - fine to medium sand, 15%-25% nonplastic fines, brown.	
5	S2	4-2 3-3	24	10	Little casing driving resistance 4' to 9'. Rolled through gravel or cobble at 8.8'.	S2: **Silty Sand (SM)** - fine to medium sand, 40%-50% slightly plastic fines, occasional subangular gravel to 3/8", redish brown.	~10'
10	S3	6-3 2-10	24	16	Little casing driving resistance 9' to 14'.	S3: **Sandy Silt (ML)** - slightly plastic fines, 10%-20% fine to medium sand, occasional subangular gravel to 1/4", fine roots throughout, moist, redish brown. Rock fragment in tip of spoon.	Sandy Silt ~14'
15	S4	30-50 11-22	24	9	Split-spoon bent. Casing refusal at 16.4'. Rolled ahead and broke through boulder at 17'. Drove casing to 19'. End of casing crimped.	S4: **Silty Sand with Gravel (SM)** - fine to coarse sand, 20%-30% nonplastic fines, 30%-40% angular gravel to 1" (including rock fragments), olive-brown.	Silty Sand with Gravel (Possible Glacial Till)
20	S5	25-27 35-16	24	10	Rolled ahead to 24', then drove casing to 24'.	S5: **Silty Sand with Gravel (SM)** - fine to medium sand, 20%-30% nonplastic fines, 30%-40% subangular gravel to 1" (including black rock fragments), olive & rust.	
24.0	S6	50/0"	0	0		**S6: Spoon Refusal -** no penetration/no recovery	24'

Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel
REC - Recovery length of sample
S - Split Spoon Sample
C - Rock Core Sample
U - Undisturbed Tube Sample

Ward Geotechnical Consulting, PLLC	Project:	Whittier Street Bridge	Boring Log
	Location:	Dover, New Hampshire	**B101**
	Client:	The Louis Berger Group, Inc.	
	Project No.:	11210	

Contractor:	New Hampshire Boring, Inc.	Groundwater Depth:	Date:	Page 2 of 2
Logged By:	Craig Ward			
Drilling Dates:	4/27/2011	GS Elevation: 57.4 feet	Boring Location:	
Drill Rig:	Mobile B-47 Truck	Datum: NGVD29	west abutment - eastbound lane	

DEPTH FT.	SAMPLE TYPE & NO.	SAMPLE BLOWS per 6 IN.	SAMPLE PEN. IN.	SAMPLE REC. IN.	REMARKS	GRAPHIC LOG	SOIL AND ROCK DESCRIPTIONS	
25					Rolled to 25' to core.			
	C1		60	60	Slowly lost water while coring. Coring rates varied from 6.5 to 11.5 min/foot.		C1: **Bedrock** - fine grained gray meta-sedimentary rock, soft to hard, fresh to slightly weathered, steep foliation (60° - 90°), several fine quartz veins and quartz intrusion from 27.8' to 3.1', most joints near horizontal & dipping 10° to 30°, one joint dipping 60° to 70° at 28.6', joint spacings range from 0.5" to 16". RQD = 39"/60" = 65%	**Bedrock**
30							Bottom of Boring at 30'	
35								
40								
45								

Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel
REC - Recovery length of sample
S - Split Spoon Sample
C - Rock Core Sample
U - Undisturbed Tube Sample

Ward Geotechnical Consulting, PLLC

Project:	Whittier Street Bridge		Boring Log
Location:	Dover, New Hampshire		**B102**
Client:	The Louis Berger Group, Inc.		
Project No.:	11210		

Contractor:	New Hampshire Boring, Inc.	Groundwater Depth:	Date:
Logged By:	Craig Ward		
Drilling Dates:	4/27/2011	GS Elevation: 57.2 feet	Boring Location:
Drill Rig:	Mobile B-47 Truck	Datum: NGVD29	west abutment (rear) - eastbound lane

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	SOIL AND ROCK DESCRIPTIONS	Stratum
0					4" Case & Wash	8" Asphalt Pavement	
	S1	10-5 4-3	24	14		S1: upper 3": **Silty Sand with Gravel (SM)** - fine to medium sand, 10%-20% nonplastic fines, 10%-20% subang. gravel to 3/8",black. next 3": **Sand with Silt & Gravel (SW-SM)** - fine to coarse sand, 5%-15% nonplastic fines, 10%-20% subang gravel to 3/8", brown.	Fill ~2'
						lower 8": **Sandy Silt (ML)** - slightly plastic fines, 10%-20% fine to medium sand, occasional subrounded gravel to 3/4", orange-brown.	
5	S2	5-7 9-10	24	16		S2: **Sandy Silt (ML)** - nonplastic fines, 15%-30% fine sand, stratified structure, orange-brown, light brown, & dark brown.	Sandy Silt and Silty Sand
10	S3	6-5 4-50	24	14		S3: **Silty Sand (SM)** - fine sand, 40%-50% slighlty plastic fines. stratified structure, fine sand parting near bottom of sample, occasional subrounded gravel to 3/8", small twig imbedded in middle of sample, orange-brown & brown.	~11'
					Boulder at 11'. Rolled ahead and broke thru at ~12'. Difficult driving casing 12' to 14'. Cobbles or boulders at ~13'.	Rock fragment with silty sand (fine to medium sand with 15%-25% nonplastic fines) in tip of spoon,	Silty Sand with Gravel and Cobbles/Boulders (Possible Glacial Till)
15	S4	50/5.5"	5.5	4	Casing crimped. Rolled ahead thru several cobbles or boulders from 14.5' to 18'.	S4: **Silty Sand with Gravel (SM)** - fine to coarse sand, 20%-30% nonplastic fines, 25%-35% subangular gravel to 3/4", heterogeneouse structure, olive. Black rock fragment in tip of spoon.	~18'
20					Rolled in boulder or bedrock from 18' to 21'.	**Boulder or Bedrock**	Possible Bedrock
						Bottom of boring at 21'	

Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel
REC - Recovery length of sample
S - Split Spoon Sample
C - Rock Core Sample
U - Undisturbed Tube Sample

Ward Geotechnical Consulting, PLLC	Project: Whittier Street Bridge Location: Dover, New Hampshire Client: The Louis Berger Group, Inc. Project No.: 11210	Boring Log **B103**
Contractor: New Hampshire Boring, Inc. Logged By: Craig Ward	Groundwater Depth: Date:	
Drilling Dates: 4/28/2011 Drill Rig: Mobile B-47 Truck	GS Elevation: ~58.4 feet Datum: NGVD29	Boring Location: east abutment - eastbound lane

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	SOIL AND ROCK DESCRIPTIONS	Stratum
					4" Case & Wash	10" Asphalt Pavement	Fill
	S1	15-17 11-7	24	13		S1: upper 2": **Asphalt and Gravel** next 3": **Sand with Gravel (SW)** - fine to medium (some coarse) sand, 15%-25% subangular gravel to 1/2", light brown. lower 8": **Silty Sand with Gravel (SM)** - fine to medium (some coarse) sand, 15%-25% nonplastic fines, 10%-20% subangular gravel to 3/4", brown.	
5	S2	3-2 3-4	24	2	Pushed cobble ahead with spoon. Casing drove easily 4' to 9'. Rolled through gravel or cobbles at ~8.7' to 9'.	S2: **Silty Sand (SM)** - fine to medium sand, 15%-25% nonplastic fines, 10%-20% subangular gravel to 3/4", brown.	
10	S3	10-8 9-11	24	10	Casing drove easily from 9' to 14'. Rolled through gravel or cobbles.	S3: **Silty Sand with Gravel (SM)** - fine to medium sand, 20%-30% nonplastic fines, 20%-30% angular gravel to 3/4", occasional brick fragments, olive-brown.	~13'
15	S4	3-2 2-3	24	0	Pushed cobble ahead with spoon. Casing drove easily to ~17'. Rolled through cobbles or boulders below 18'.	S4: **No Recovery** Redrove with 3" split-spoon: 12" recovery: **Sandy Silt (ML)** - slightly plastic fines, 10%-30% fine to medium sand, occasional gravel to 3/4", 2 small pockets of organic soil, olive-brown.	Sandy Silt 17'
20	S5	25-26 50/5"	17	10	Casing refusal at 19.6'. Rolled ahead. Rolled on dense till or weathered bedrock at 21'. Rolled to 23' to core.	S5: **Silty Sand with Gravel (SM)** - fine to medium (some coarse) sand, 15%-25% nonplastic fines, 25%-35% subangular gravel to 1" (some weathered), heterogeneous structure, olive-brown.	Silty Sand with Gravel (Possible Glacial Till) 21'
							Bedrock

Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel
REC - Recovery length of sample
S - Split Spoon Sample
C - Rock Core Sample
U - Undisturbed Tube Sample

Ward Geotechnical Consulting, PLLC	Project: Whittier Street Bridge Location: Dover, New Hampshire Client: The Louis Berger Group, Inc. Project No.: 11210	Boring Log **B103**
Contractor: New Hampshire Boring, Inc. Logged By: Craig Ward	Groundwater Depth: Date:	
Drilling Dates: 4/28/2011 Drill Rig: Mobile B-47 Truck	GS Elevation: ~58.4 feet Datum: NGVD29	Boring Location: east abutment - eastbound lane

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	GRAPHIC LOG	SOIL AND ROCK DESCRIPTIONS	
25	C1		60	18	Coring rate varied from 2 to 13 min/foot. Only 18" recovery - attempt to retreive failed.		C1: **Bedrock** - fine grained gray meta-sedimentary rock, soft to hard, moderately weathered, steep foliation (~80°), several fine quartz veins, joints dipping 10° to 20° and ~80° (along foliations), joint spacings range from 0.5" to ~4", silty gouge on some fracture faces. RQD ~ 0%	**Bedrock**
	C2		16	12	Coring rates of 1.5 & 5 min/foot. Core barrel jamed at 29.3'. Rolled to 30' to core C3.		C2: **Bedrock** - fine grained gray meta-sedimentary rock, soft to hard, highly weathered, steep foliation (~80°), several fine quartz veins, joints near vertical and near horizontal, joint spacings range from 0.5" to ~3". RQD ~ 0.	
30	C3		60	60	Coring rate varied from 4 to 6 min/foot.		C3: **Bedrock** - fine grained gray meta-sedimentary rock, soft to hard, fresh to slightly weathered, steep foliation (70° to 80°), 3" thick quartz intrusion at 32.7', joints above quartz intrusion dip 70° to 80° with spacings of 0.5" to 5.5", joints below quartz intrusion are near horizontal and near vertical at spacings of 0.5" to 5.5". RQD = 13.5"/60" = 23%	
35							Bottom of boring at 35'.	
40								
45								

Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel
REC - Recovery length of sample
S - Split Spoon Sample
C - Rock Core Sample
U - Undisturbed Tube Sample

Ward Geotechnical Consulting, PLLC

Project:	Whittier Street Bridge		Boring Log
Location:	Dover, New Hampshire		**B104**
Client:	The Louis Berger Group, Inc.		
Project No.:	11210		

Contractor:	New Hampshire Boring, Inc.	Groundwater Depth:	Date:	Page 1 of 1
Logged By:	Craig Ward			
Drilling Dates:	4/28/2011	GS Elevation: 59.7 feet	Boring Location:	
Drill Rig:	Mobile B-47 Truck	Datum: NGVD29	east abutment - eastbound lane (rear)	

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	SOIL AND ROCK DESCRIPTIONS	STRATUM
					4" Case & Wash	9" Asphalt Pavement	Fill
	S1	12-10 8-8	24	15	Rolled through cobble at ~3.5'.	S1: upper 9": **Sand with Gravel (SW)** - fine to coarse sand, 10%-20% subangular gravel to 1/2", orange-brown. lower 6": **Silty Sand (SM)** - fine to medium sand, 20%-30% nonplastic fines, occasional subrounded gravel to 3/4", 3/8" chunk of asphalt, olive-brown.	~3.5'
5	S2	4-3 5-5	24	15		S2: upper 10": **Sandy Silt (ML)** - nonplastic fines, 10%-20% fine sand (occasional medium to coarse sand), vague stratification, occasional fine roots, orange-brown. lower 5": **Silty Fine Sand (SM)** - fine (some medium) sand, 10%-20% nonplastic fines, light brown-orange.	Sandy Silt and Silty Sand
10	S3	4-3 3-4	24	14		S3: **Silty Fine Sand (SM)** - fine sand, 20%-30% nonplastic fines, vague stratification, light brown-orange. Orange-brown silt in tip of spoon.	~13'
15	S4	10-31 43-28	24	6	Rolled through several cobbles from 16' to 19'.	S4: **Silty Sand with Gravel (SM)** - fine to medium (some coarse) sand, 20%-30% nonplastic fines, 30%-40% subangular gravel to 3/4", olive-brown. Rock fragment in tip of spoon.	Silty Sand with Gravel (Possible Glacial Till)
20	S5	41-33 36-44	24	13		S5: **Silty Sand with Gravel (SM)** - similar to S4.	21.5'
					Rolling on boulder or bedrock from 21.5' to 24.1'.		Boulder or Bedrock
						Bottom of Boring at 24.1'	

Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel
REC - Recovery length of sample
S - Split Spoon Sample
C - Rock Core Sample
U - Undisturbed Tube Sample

Ward Geotechnical Consulting, PLLC			
Project:	Whittier Street Bridge	Boring Log	
Location:	Dover, New Hampshire	**B105**	
Client:	The Louis Berger Group, Inc.		
Project No.:	11210		

Contractor:	New Hampshire Boring, Inc.	Groundwater Depth:	Date:	Page 1 of 1
Logged By:	Craig Ward			
Drilling Dates:	4/29/2011	GS Elevation: 58.3 feet	Boring Location:	
Drill Rig:	Mobile B-47 Truck	Datum: NGVD29	east abutment - westbound lane	

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	SOIL AND ROCK DESCRIPTIONS	
					4" Case & Wash	11" Asphalt Pavement	Fill
	S1	16-7 5-3	24	13		S1: upper 7": **Sand with Silt & Gravel (SW-SM)** - fine to coarse sand, 5%-15% nonplastic fines, 25%-35% subrounded gravel to 3/4", brown & black. lower 6": **Sandy Silt with Gravel (ML)** - nonplastic fines, 10%-20% fine sand, light brown-orange.	
5	S2	2-3 6-12	24	10		S2: **Sandy Silt with Gravel (ML)** - nonplastic fines, 10%-30% fine to medium sand, 10%-20% subrounded gravel to 3/4", light brown-olive.	
10	S3	24-18 19-10	24	10	Split-spoon deflected away from river. Spoon bent. Casing refusal at 10'. Rolled thru boulder at 11.5'. Casing refusal at 12.8'. Rolled ahead (lost water) to 13.1'. Seated casing at 13'. Rolled to 14' to core.	S3: **Silty Sand with Gravel (SM)** - fine to medium sand, 20%-30% nonplastic fines, 30%-40% subangular gravel to 1', olive-brown.	
15	C1		36	21	Core barrel dropped suddenly from 15' to 15.4'. Core barrel dropped suddenly again from 16.5' to 17'. Pulled core barrel to check.	C1: **Boulders or Stone Masonry** - gray meta-sedimentary rock similar to local bedrock but with horizontal folliation - probably stone masonry near bottom of wing wall or abutment wall.	
	S4	16-7 19-46	25	5	Took S4 at 17'.	S4: **Silty Sand with Gravel (SM)** - fine (some medium) sand, 20%-30% nonplastic fines, 20%-30% subangular gravel to 3/4", olive-brown & gray. Possibly fill or till that was disturbed by coring.	
20					Tried to roll ahead to 19', but hole collapsed. Coud not advance casing through boulders or masonry (refusal at 13'). Abandoned boring and moved ~2.5' east to B5A.	Bottom of Boring at 19'	

Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel | S - Split Spoon Sample | U - Undisturbed Tube Sample

REC - Recovery length of sample | C - Rock Core Sample

Ward Geotechnical Consulting, PLLC

Project:	Whittier Street Bridge	Boring Log	
Location:	Dover, New Hampshire	**B105A**	
Client:	The Louis Berger Group, Inc.		
Project No.:	11210		

Contractor:	New Hampshire Boring, Inc.	Groundwater Depth:	Date:
Logged By:	Craig Ward		
Drilling Dates:	4/29/2011	GS Elevation: 58.5 feet	Boring Location:
Drill Rig:	Mobile B-47 Truck	Datum: NGVD29	east abutment - westbound lane

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	GRAPHIC LOG	SOIL AND ROCK DESCRIPTIONS	
0					4" Case & Wash		Moved to B5A from B5, which was abandoned due to difficulties in advancing casing through boulders or masonry blocks. B5A drilled to determine conditions below 19', where B5 was abandoned. No samples obtained from B5A above 19'. Refer to log for B5 for descriptions of subsurface condtions above 19'.	
5					Little resistance to driving casing from 4' to 9'.			
10					Little resistance to driving casing from 9' to 14'.			
15					Increased casing resistance at ~17'.			
20	S1	29-31 56-73	24	13	Rolled ahead to 22'. Drove casing to refusal at 22.3'. Rolled in bedrock from 22.3' to 23' to core.		S1: **Silty Sand with Gravel (SM)** - fine to medium (some coarse) sand, 15%-25% nonplastic fines, 25%-35% subangular gravel to 1", olive-brown.	**Silty Sand with Gravel (Glacial Till)** **22.3'**
								Bedrock

Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel
REC - Recovery length of sample
S - Split Spoon Sample
C - Rock Core Sample
U - Undisturbed Tube Sample

Ward Geotechnical Consulting, PLLC	Project: Whittier Street Bridge Location: Dover, New Hampshire Client: The Louis Berger Group, Inc. Project No.: 11210	Boring Log **B105A**
Contractor: New Hampshire Boring, Inc. Logged By: Craig Ward	Groundwater Depth: Date:	Page 2 of 2
Drilling Dates: 4/29/2011 Drill Rig: Mobile B-47 Truck	GS Elevation: 58.5 feet Datum: NGVD29	Boring Location: east abutment - westbound lane

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	GRAPHIC LOG	SOIL AND ROCK DESCRIPTIONS	
25	C1		60	59	Coring rate varied from 4 to 5 min/foot.		C1: **Bedrock** - fine grained gray meta-sedimentary rock, soft to hard, fresh to slightly weathered, steep foliation (60° to 80°), joints dipping 0° to 20° and ~70° (along foliation) at spacings ranging from 1.5" to 9". RQD = 45"/60" = 75%	**Bedrock**
							Bottom of Boring at 28'	
30								
35								
40								
45								

Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel
REC - Recovery length of sample
S - Split Spoon Sample
C - Rock Core Sample
U - Undisturbed Tube Sample

Ward Geotechnical Consulting, PLLC			
Project:	Whittier Street Bridge	Boring Log	
Location:	Dover, New Hampshire	**B106**	
Client:	The Louis Berger Group, Inc.		
Project No.:	11210		

Contractor:	New Hampshire Boring, Inc.	Groundwater Depth:	Date:
Logged By:	Craig Ward		
Drilling Dates:	4/29/2011	GS Elevation: 59.9 feet	Boring Location:
Drill Rig:	Mobile B-47 Truck	Datum: NGVD29	east abutment - westbound lane (rear)

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	SOIL AND ROCK DESCRIPTIONS	STRATUM
0					4" Case & Wash	9" Asphalt Pavement	Fill
	S1	6-4 5-6	24	22		S1: **Silty Fine Sand (SM)** - fine sand, 10%-30% (variable) nonplastic fines, occasional angular gravel to 3/4", light brown-orange.	~5'
5	S2	5-4 5-5	24	20		S2: **Silty Fine Sand (SM)** - fine sand, 10%-30% (increasing with depth) nonplastic fines, light brown-orange.	Silty Fine Sand
10	S3	4-3 2-2	24	21		S3: **Silty Fine Sand (SM)** - similar to S2.	~14'
15	S4	26-22 15-21	24	7		S4: **Silty Sand with Gravel (SM)** - fine to medium sand, 15%-25% nonplastic fines, 25%-35% subangular gravel to 3/4", olive-brown.	Silty Sand with Gravel (Possible Glacial Till)
					Rolled through boulder from 16.8' to ~19'.		
20	S5	24-25 26-35	24	17		S5: **Silty Sand with Gravel (SM)** - fine to medium (some coarse) sand, 10%-20% nonplastic fines, 30%-40% subangular gravel to 1", olive-brown.	23.4'
					Rolling on boulder or bedrock from 23.4' to 27.4'.		Bedrock or Boulder
						Bottom of Boring at 27.4'	

Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel
REC - Recovery length of sample
S - Split Spoon Sample
C - Rock Core Sample
U - Undisturbed Tube Sample

Ward Geotechnical Consulting, PLLC

Project:	Whittier Street Bridge		Boring Log
Location:	Dover, New Hampshire		**B107**
Client:	The Louis Berger Group, Inc.		
Project No.:	11210		

Contractor:	New Hampshire Boring, Inc.	Groundwater Depth:	Date:
Logged By:	Craig Ward		
Drilling Dates:	5/2/2011	GS Elevation: 57.4 feet	Boring Location:
Drill Rig:	Mobile B-47 Truck	Datum: NGVD29	west abutment - westbound lane

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	SOIL AND ROCK DESCRIPTIONS	STRATUM
					4" Case & Wash	9" Asphalt Pavement	Fill
	S1	10-12 15-13	24	10		S1: **Sand with Gravel (SW)** - fine to medium (some coarse) sand, 25%-35% subrounded gravel to 3/4", orange-brown.	~3.5'
5	S2	4-2 2-12	24	6		S2: **Sandy Silt (ML)** - nonplastic fines, 10%-20% fine to medium sand, occasional rounded gravel to 1/2", light brown-orange. Possible Fill.	Sandy Silt - Possible Fill
					Rolled through boulder at ~8.5'.		
10	S3	10-4 10-6	24	14		S3: **Sandy Silt (ML)** - nonplastic fines, 5%-10% subrounded gravel to 3/4", olive-brown. Possible Fill.	~14'
15 15.2	S4	26-40 60/2"	14	6	Casing refusal at 15.6'. Rolled ahead, lost water, and broke thru boulder at 16.2'. Drove casing to refusal at 18.7'. Rolled ahead and lost water at 19'. Took S5.	S4: **Sandy Silt (ML) & Rock Fragments** - slightly plastic fines, 10%-20% fine sand, olive-brown. About 60% of sample consists of angular rock fragments.	Sandy Silt and Silty Sand with Boulders/Cobbles (Possble Glacial Till)
20	S5	29-75	12	3	Rolled ahead to 21.4' and drove casing to 20.4'. Rolled on bedrock or boulder from 21.2' to 22'. Started core at 22'.	S5: **Rock Fragments** - angular rock fragments with a small amount of silty sand. Possible till with boulders or weathered bedrock.	~21'
	C1		38	38	Core barrel jammed at 25.2'.	C1: **Bedrock** - fine grained gray meta-sedimentary rock, steep foliation (70° to 90°), fresh to slightly weathered, joints dipping ~10°, 35° to 45°, and 70° to 90° (along foliation) at spacings of 1.5" to 9". RQD = 22.5"/38" = 59%	Bedrock

Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel
REC - Recovery length of sample
S - Split Spoon Sample
C - Rock Core Sample
U - Undisturbed Tube Sample

Ward Geotechnical Consulting, PLLC			
Project:	Whittier Street Bridge	Boring Log	
Location:	Dover, New Hampshire	**B107**	
Client:	The Louis Berger Group, Inc.		
Project No.:	11210		

Contractor:	New Hampshire Boring, Inc.	Groundwater Depth:	Date:	
Logged By:	Craig Ward			
Drilling Dates:	5/2/2011	GS Elevation: 57.4 feet	Boring Location:	
Drill Rig:	Mobile B-47 Truck	Datum: NGVD29	west abutment - westbound lane	

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	GRAPHIC LOG	SOIL AND ROCK DESCRIPTIONS	
25 / 25.2	C2		22	21	Coring rate varied from 5.5 to 7.5 min/foot. Coring rate of 7.5 min/foot.		C2: **Bedrock** - fine grained gray meta-sedimentary rock, fresh to slightly weathered, ~45° foliation, quartz intrusion from 25.8' to 26.1', joints near horizontal and dipping ~45° (along foliation) at spacings from 1.5" to 6.5". RQD = 11.5"/22" = 52%	**Bedrock**
							Bottom of Boring at 27'.	
30								
35								
40								
45								

Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel
REC - Recovery length of sample
S - Split Spoon Sample
C - Rock Core Sample
U - Undisturbed Tube Sample

Ward Geotechnical Consulting, PLLC			
Project:	Whittier Street Bridge	Boring Log	
Location:	Dover, New Hampshire	**B108**	
Client:	The Louis Berger Group, Inc.		
Project No.:	11210		

Contractor:	New Hampshire Boring, Inc.	Groundwater Depth:	Date:
Logged By:	Craig Ward		
Drilling Dates:	5/2/2011	GS Elevation: 57.3 feet	Boring Location:
Drill Rig:	Mobile B-47 Truck	Datum: NGVD29	west abutment - westbound lane (rear)

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	SOIL AND ROCK DESCRIPTIONS	Strata
0					4" Case & Wash	10" Asphalt Pavement	
	S1	18-13 10-6	24	3		S1: **Sand with Silt & Gravel (SP-SM)** - fine to medium sand, 5%-15% nonplastic fines, 20%-30% gravel, brown. Rock fragment in tip of spoon. Due to poor recovery, overdrove 3" spoon: 20" recovery: upper 12": **Sand with Gravel (SW)** - fine to coarse sand, 25%-35% subrounded & subangular gravel to 2", 5%-10% fines, brown.	**Fill** **~2`**
						lower 8": **Sandy Silt (ML)** - nonplastic fines, 10%-20% fine to medium sand, occasional fine roots, olive-brown. Possible Fill.	
5	S2	4-3 5-13	24	13		S2: **Sandy Silt (ML)** - similar to lower 8" of S1 overdrive. Possible Fill.	**Sandy Silt - Possible Fill** **~8'**
					Increased casing resistance below 8'. Lost water at 9'.		
10 (9.7)	S3	22-50/2"	8	0		S3: **No Recovery** - probably pushed boulder with spoon.	
					Rolled ahead and drove casing to 14'.		
15 (14.3)	S4	50/3"	3	1	Rolled ahead. Cuttings in wash appear to be weathered rock.	S4: **Silty Sand with Gravel (SM)** - fine to coarse sand, 10%-20% nonplastic fines, 40%-50% angular gravel and rock fragments, olive-brown.	**Silty Sand with Gravel & Cobbles/ Boulders (Possibly Glacial Till)** **~15'**
					Roller bit cut rapidly from 15' to 16', then slowed. Lost water at 16.4'. Rolled to 18' in boulder or bedrock.		**Boulder or Bedrock**
						Bottom of Boring at 18'	
20							

Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel
REC - Recovery length of sample
S - Split Spoon Sample
C - Rock Core Sample
U - Undisturbed Tube Sample

Ward Geotechnical Consulting, PLLC

Project:	Whittier Street Bridge	Boring Log	
Location:	Dover, New Hampshire	**B109**	
Client:	The Louis Berger Group, Inc.		
Project No.:	11210		

Contractor:	New Hampshire Boring, Inc.	Groundwater Depth:	Date:	Page 1 of 1
Logged By:	Craig Ward			
Drilling Dates:	5/3/2011	GS Elevation: 47.5 feet	Boring Location:	
Drill Rig:	Remote ATV Rig	Datum: NGVD29	near toe of embankment at northwest quadrant	

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	SOIL AND ROCK DESCRIPTIONS	
0–1.8	S1	2-4 12-100/4"	22	16	4" Case & Wash Rolled in bedrock to 3' to core.	S1: upper 2": **Forest Mat** next 9": **Sandy Silt (ML)** - nonplastic fines, 20%-30% fine sand, roots, brown. lower 5": **Sand with Gravel (SW)** - fine to medium (some coarse) sand, 30%-40% subrounded gravel and angular rock fragments to 3/4", dry, light brown-gray.	**Fill** ~2'
3–8	C1		60	55	Coring rate varied from 3 to 4 min/foot. Core barrel dropped ~1" at 4.9'.	C1: **Bedrock** - fine grained gray meta-sedimentary rock, fresh to slightly weathered, vague foliation dipping ~45° to 80° (variable), 1/2" thick quartz intrusion at 6.5', occasional quartz veins, joints dipping 0° to 15°, ~30°, 60° to 70° at spacings ranging from 1.5" to 8". RQD = 44"/60" = 73%	**Bedrock**
						Bottom of Boring at 8'	

Depth scale: 5, 10, 15, 20

Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel
REC - Recovery length of sample
S - Split Spoon Sample
C - Rock Core Sample
U - Undisturbed Tube Sample

Ward Geotechnical Consulting, PLLC

Project:	Whittier Street Bridge		Boring Log
Location:	Dover, New Hampshire		**B110**
Client:	The Louis Berger Group, Inc.		
Project No.:	11210		

Contractor:	New Hampshire Boring, Inc.	Groundwater Depth:	Date:	Page 1 of 1
Logged By:	Craig Ward			
Drilling Dates:	5/3/2011	GS Elevation: 59.2 feet	Boring Location:	
Drill Rig:	Remote ATV Rig	Datum: NGVD29	~247' west of west abutment - westbound lane	

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	SOIL AND ROCK DESCRIPTIONS	
					4.25" Hollow Stem Augers	5" Asphalt Pavement	Fill: Sand with Gravel (base)
	S1	2-5 6-2	24	20	S1 is 3" split-spoon driven with 300# hammer, 24" drop,	S1: upper 6": **Sand with Gravel (SW)** - fine to coarse sand, 20%-30% subrounded gravel to 3/4", dark brown. next 6": **Sand with Gravel (SW)** - fine to medium (some coarse) sand, 15%-25% subangular gravel to 1/2", light brown.	~1.5'
						lower 8": **Sandy Silt (ML)** - nonplastic fines, 10%-20% fine sand, occasional subangular gravel to 1/2", moist, orange-brown.	Sandy Silt and Silty Sand
	S2	3-4 3-13	24	20		S2: **Sandy Silt (ML)** - nonplastic fines, 10%-30% fine sand (increasing with depth), light brown-orange.	
5	S3	23-90 42-39	24	18		S3: upper 7": **Sandy Silt (ML)** - nonplastic fines, 15%-25% fine to medium sand, light brown-olive. next 5": **Rock Fragments** lower 6": **Silty Fine Sand (SM)** - fine sand, 10%-20% nonplastic fines, light brown-tan with rust streaks. Bottom 3" is wet. Weathered rock fragment in tip of spoon.	
						Bottom of Boring at 6.5'	
10							
15							
20							

Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel
REC - Recovery length of sample
S - Split Spoon Sample
C - Rock Core Sample
U - Undisturbed Tube Sample

Ward Geotechnical Consulting, PLLC			
Project:	Whittier Street Bridge		Boring Log
Location:	Dover, New Hampshire		**B111**
Client:	The Louis Berger Group, Inc.		
Project No.:	11210		

Contractor:	New Hampshire Boring, Inc.	Groundwater Depth:	Date:
Logged By:	Craig Ward		Page 1 of 1
Drilling Dates:	5/3/2011	GS Elevation: 68.9 feet	Boring Location:
Drill Rig:	Remote ATV Rig	Datum: NGVD29	~500' west of west abutment - westbound lane

DEPTH FT.	SAMPLE TYPE & NO.	SAMPLE BLOWS per 6 IN.	SAMPLE PEN. IN.	SAMPLE REC. IN.	REMARKS	GRAPHIC LOG	SOIL AND ROCK DESCRIPTIONS	
					Hollow Stem Augers		4.5" Asphalt Pavement	**Fill: Sand with Gravel (base)**
	S1	6-14 10-6	24	17	S1 is 3" split-spoon driven with 300# hammer, 24" drop,		S1: **Sand with Gravel (SW)** - fine to medium (some coarse) sand, <10% fines, 30%-40% subrounded gravel to 2", lt. brown-orange.	**~2.5'**
	S2	4-8 12-16	24	24			S2: **Sandy Silt (ML)** - slightly plastic fines, laminated structure, 10%-20% fine sand, olive-brown, orange, & gray. Fine sand lens (~1/4" thick) near bottom of sample.	**Silt**
5	S3	5-10 13-12	24	24			S3: **Clayey Silt (ML)** - varved clayey silt and silty clay (thin laminations) with a few fine sand partings, slightly to medium plastic, 5%-15% fine sand, olive-brown, orange, and gray.	
							Bottom of Boring at 6.5'	
10								
15								
20								

Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel
REC - Recovery length of sample
S - Split Spoon Sample
C - Rock Core Sample
U - Undisturbed Tube Sample

Ward Geotechnical Consulting, PLLC			Boring Log
	Project:	Whittier Street Bridge	**B112**
	Location:	Dover, New Hampshire	
	Client:	The Louis Berger Group, Inc.	
	Project No.:	11210	

Contractor:	New Hampshire Boring, Inc.	Groundwater Depth:	Date:	Page 1 of 1
Logged By:	Craig Ward			
Drilling Dates:	5/3/2011	GS Elevation: 71.3 feet	Boring Location:	
Drill Rig:	Remote ATV Rig	Datum: NGVD29	~190' east of eat abutment - westbound lane	

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	GRAPHIC LOG	SOIL AND ROCK DESCRIPTIONS	
					Hollow Stem Augers		5" Asphalt Pavement	
	S1	9-11 15-17	24	18	S1 is 3" split-spoon driven with 300# hammer, 24" drop,		S1: upper 6": **Sand with Gravel (SW)** - fine to medium (some coarse) sand, 35%-45% subagnular gravel to 2", 5%-10% fines, black-dark gray. Lower 12": **Sand with Gravel (SW)** - fine to coarse sand, 25%-35% subrounded gravel to 2", light brown-orange.	**Fill: Sand with Gravel (Base)** **~2.5'**
	S2	14-22 16-12	24	9			S2: **Sandy Silt (ML)** - nonplastic fines, 20%-40% fine to medium sand, occasional subangular gravel to 3/8", olive-brown.	**Sandy Silt and Silty Sand**
5	S3	6-14 17-14	24	14			S2: upper 3": **Sandy Silt (ML)** - similar to S2. lower 18": **Stratified Silty Fine Sand (SM) and Varved Silt (ML) & Clay (CL)** - mostly fine sand with 10%-20% nonplastic fines and occasional silt lenses(<1/16" thick), four 1" to 2" layers of varved silt and clay (finely laminated), light brown-orange & olive brown with rust streaks.	
							Bottom of Boring at 6.5'	
10								
15								
20								

Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel — S - Split Spoon Sample — U - Undisturbed Tube Sample

REC - Recovery length of sample — C - Rock Core Sample

Ward Geotechnical Consulting, PLLC			
Project:	Whittier Street Bridge	Boring Log	
Location:	Dover, New Hampshire	**B113**	
Client:	The Louis Berger Group, Inc.		
Project No.:	11210		

Contractor:	New Hampshire Boring, Inc.	Groundwater Depth:	Date:
Logged By:	Craig Ward		Page 1 of 1
Drilling Dates:	5//3/11	GS Elevation: 90.2 feet	Boring Location:
Drill Rig:	Remote ATV Rig	Datum: NGVD29	~400' east of east abutment - westbound lane

DEPTH FT.	SAMPLE TYPE & NO.	BLOWS per 6 IN.	PEN. IN.	REC. IN.	REMARKS	GRAPHIC LOG	SOIL AND ROCK DESCRIPTIONS	
					Hollow Stem Augers		5" Asphalt Pavement	Fill: Sand with Gravel (base) over Silty Sand with Gravel
1.5	S1	2-65	12	6	S1 is 3" split-spoon driven with 300# hammer, 24" drop,		S1: **Sand with Gravel (SW)** - fine to medium (some coarse) sand, 20%-30% subrounded gravel to 1", dark brown.	
	S2	28-34 23-22	24	20			S2: upper 12": **Sand with Gravel (SW)** - fine to coarse sand, 25%-35% subangular gravel to 2", light brown-orange. lower 8": **Silty Sand with Gravel (SM)** - fine to medium (some coarse) sand, 20%-30% nonplastic fines, 10%-20% subangular gravel to 3/8", heterogeneous, olive & gray.	~4'
	S3	13-50/3"	9	9			S3: **Silty Sand (SM)** - fine & fine to medium sand, 15%-25% non-plastic fines, vaguely stratified, light brown-orange. Bottom 3" wet.	Silty Sand
4.8 5 5.0	S4	50/0.5"	0.5	0			S4: **No Recovery**	
							Bottom of Boring at 5'	
10								
15								
20								

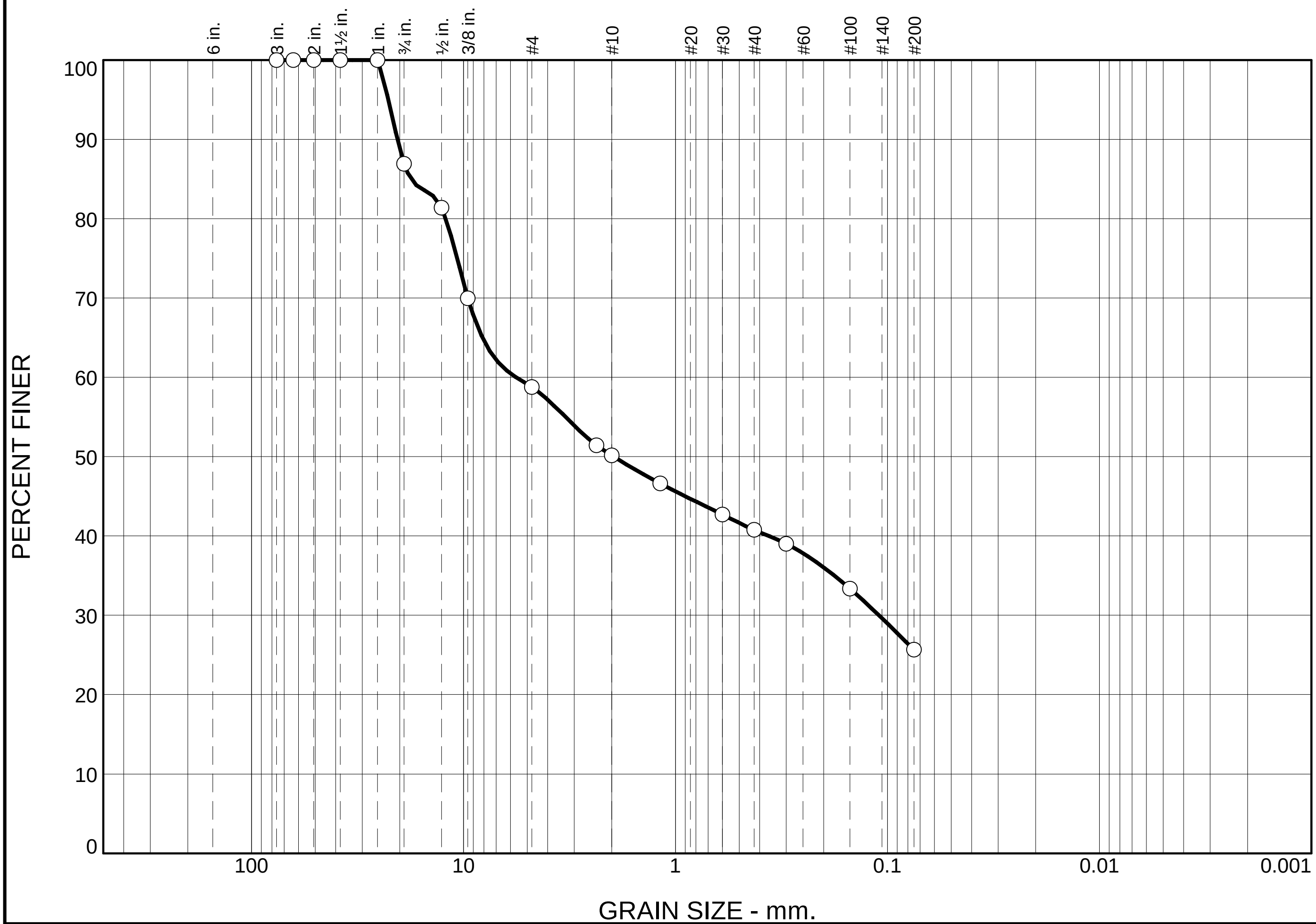
Notes:

Abbreviations:

PEN - Penetration length of sampler or core barrel
REC - Recovery length of sample
S - Split Spoon Sample
C - Rock Core Sample
U - Undisturbed Tube Sample

Appendix B – Laboratory Grain Size Analyses

Particle Size Distribution Report



% +3"	% Gravel			% Sand			% Fines
	Coarse	Medium	Fine	Coarse	Medium	Fine	
0.0	0.0	30.0	19.9	7.4	4.9	12.1	25.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3"	100.0		
2.5"	100.0		
2"	100.0		
1.5"	100.0		
1"	100.0		
0.75"	86.9		
0.5"	81.4		
0.375"	70.0		
#4	58.8		
#8	51.4		
#10	50.1		
#16	46.6		
#30	42.7		
#40	40.8		
#50	39.0		
#100	33.4		
#200	25.7		

* (no specification provided)

Material Description

coarse to fine Sand, some Silt, and med to fine Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 17.6772 D_{60}= 5.6521 D_{50}= 1.9590
D_{30}= 0.1096 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

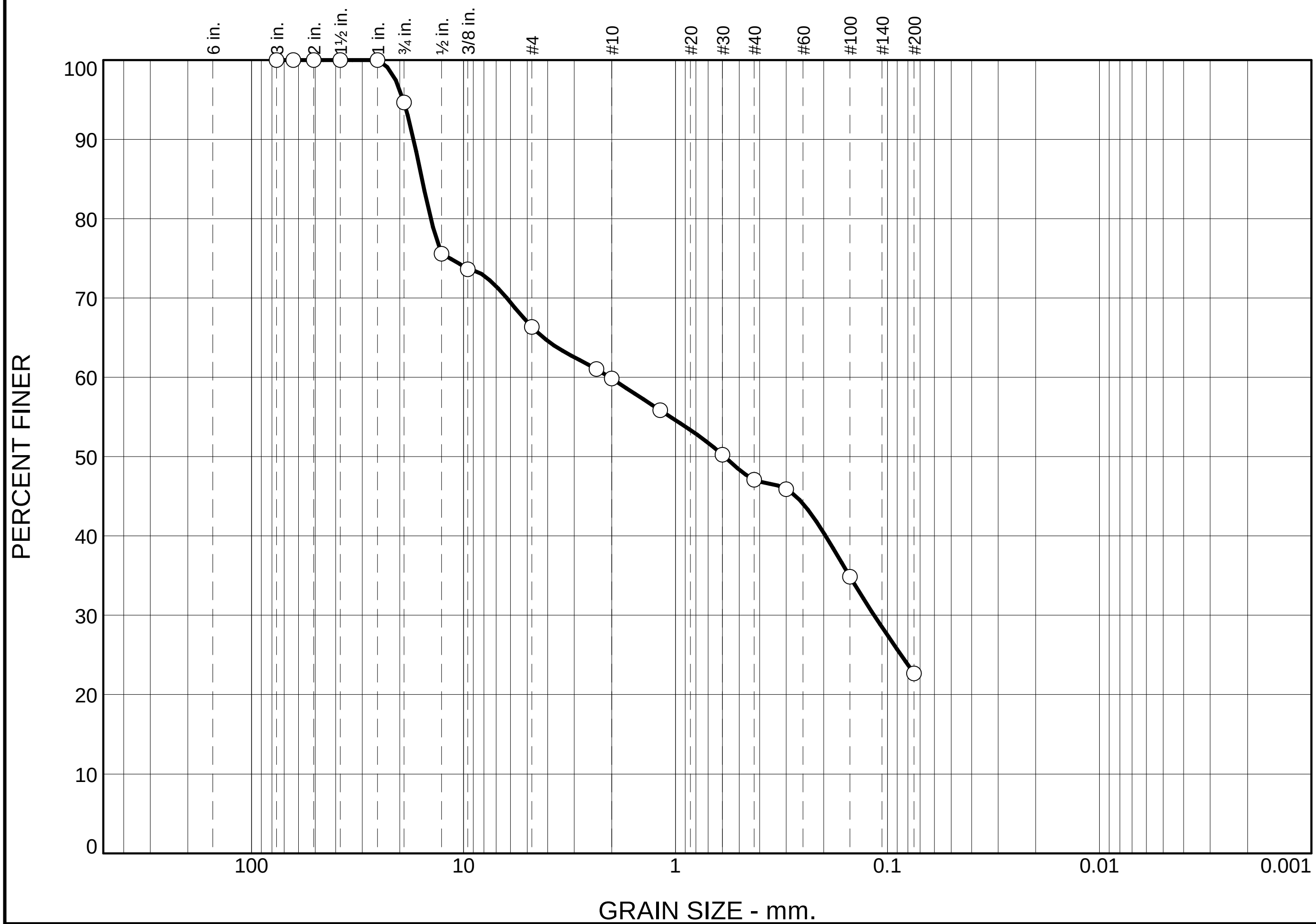
Sample No.: L-311-11 **Source of Sample:** Whittier Street **Date:** 5/31/11
Location: B-101 / S-4 **Elev./Depth:** 14-16'

GEOTECHNICAL SERVICES, INC.

Goffstown, New Hampshire

Client: Ward Geotechnical Consulting PLLC
Project: Whittier Street
Dover, New Hampshire
Project No: 205365 **Plate**

Particle Size Distribution Report



% +3"	% Gravel			% Sand			% Fines
	Coarse	Medium	Fine	Coarse	Medium	Fine	
0.0	0.0	26.4	13.8	9.6	6.1	21.4	22.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3"	100.0		
2.5"	100.0		
2"	100.0		
1.5"	100.0		
1"	100.0		
0.75"	94.7		
0.5"	75.6		
0.375"	73.6		
#4	66.3		
#8	61.0		
#10	59.8		
#16	55.8		
#30	50.2		
#40	47.1		
#50	45.9		
#100	34.8		
#200	22.7		

* (no specification provided)

Material Description

coarse to fine Sand, some Silt, and med to fine Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 15.6864 D_{60}= 2.0416 D_{50}= 0.5862
D_{30}= 0.1154 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= AASHTO=

Remarks

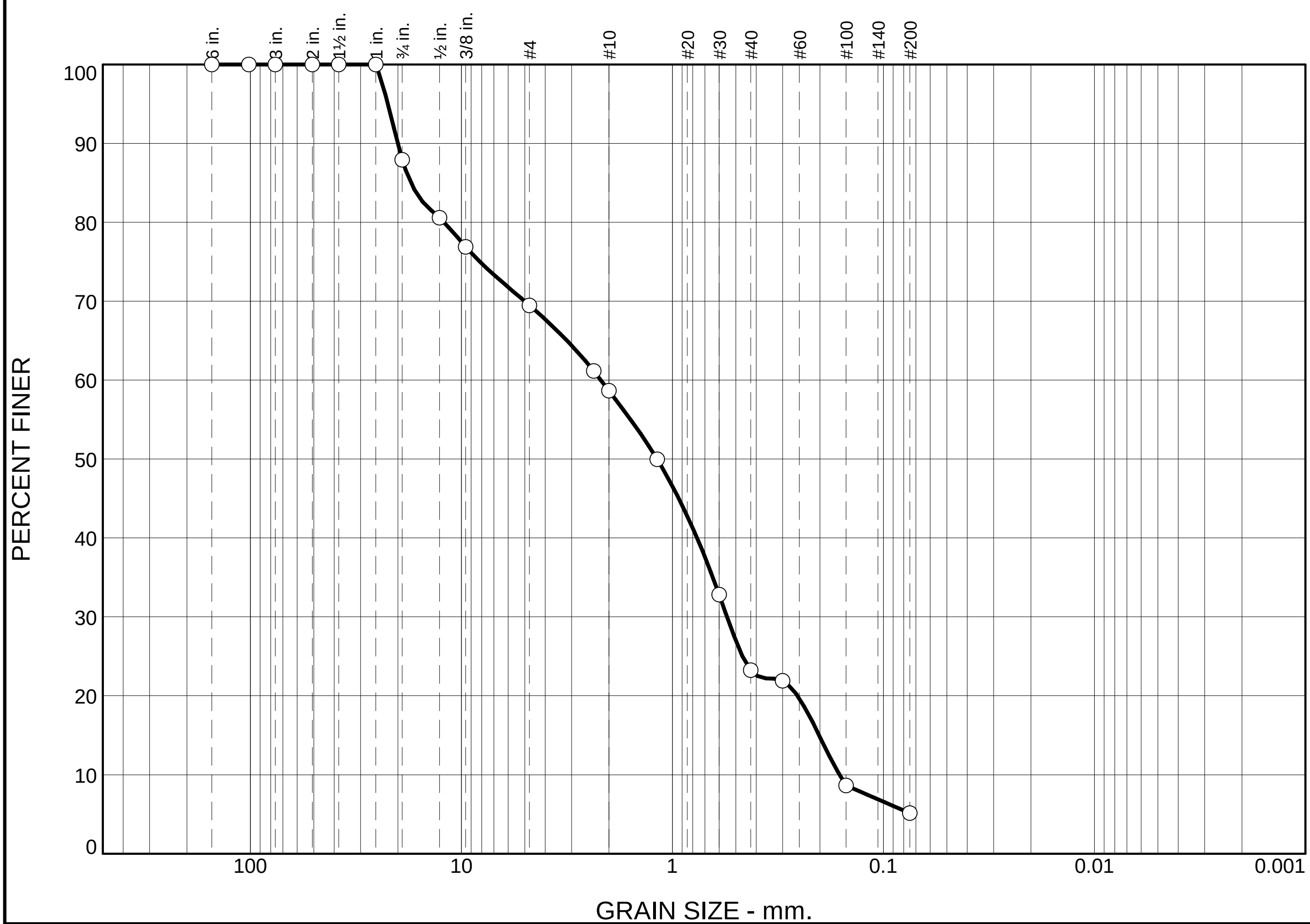
Sample No.: L-310-11 **Source of Sample:** Whittier Street **Date:** 5/31/11
Location: B-105A / S-1 **Elev./Depth:** 19-21'

GEOTECHNICAL SERVICES, INC.

Goffstown, New Hampshire

Client: Ward Geotechnical Consulting PLLC
Project: Whittier Street
Dover, New Hampshire
Project No: 205365 **Plate**

Particle Size Distribution Report



% +3"	% Gravel			% Sand			% Fines
	Coarse	Medium	Fine	Coarse	Medium	Fine	
0.0	0.0	23.1	18.2	25.9	13.2	14.5	5.1

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
6"	100.0		
4"	100.0		
3"	100.0		
2"	100.0		
1.5"	100.0		
1"	100.0		
.75"	87.9		
.5"	80.6		
.375"	76.9		
#4	69.5		
#8	61.2		
#10	58.7		
#16	50.0		
#30	32.8		
#40	23.3		
#50	21.9		
#100	8.6		
#200	5.1		

* (no specification provided)

Material Description

coarse to fine SAND, and med to fine Gravel, trace Silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 17.3415 D_{60}= 2.1833 D_{50}= 1.1821
D_{30}= 0.5507 D_{15}= 0.2014 D_{10}= 0.1614
C_u= 13.52 C_c= 0.86

Classification

USCS= AASHTO=

Remarks

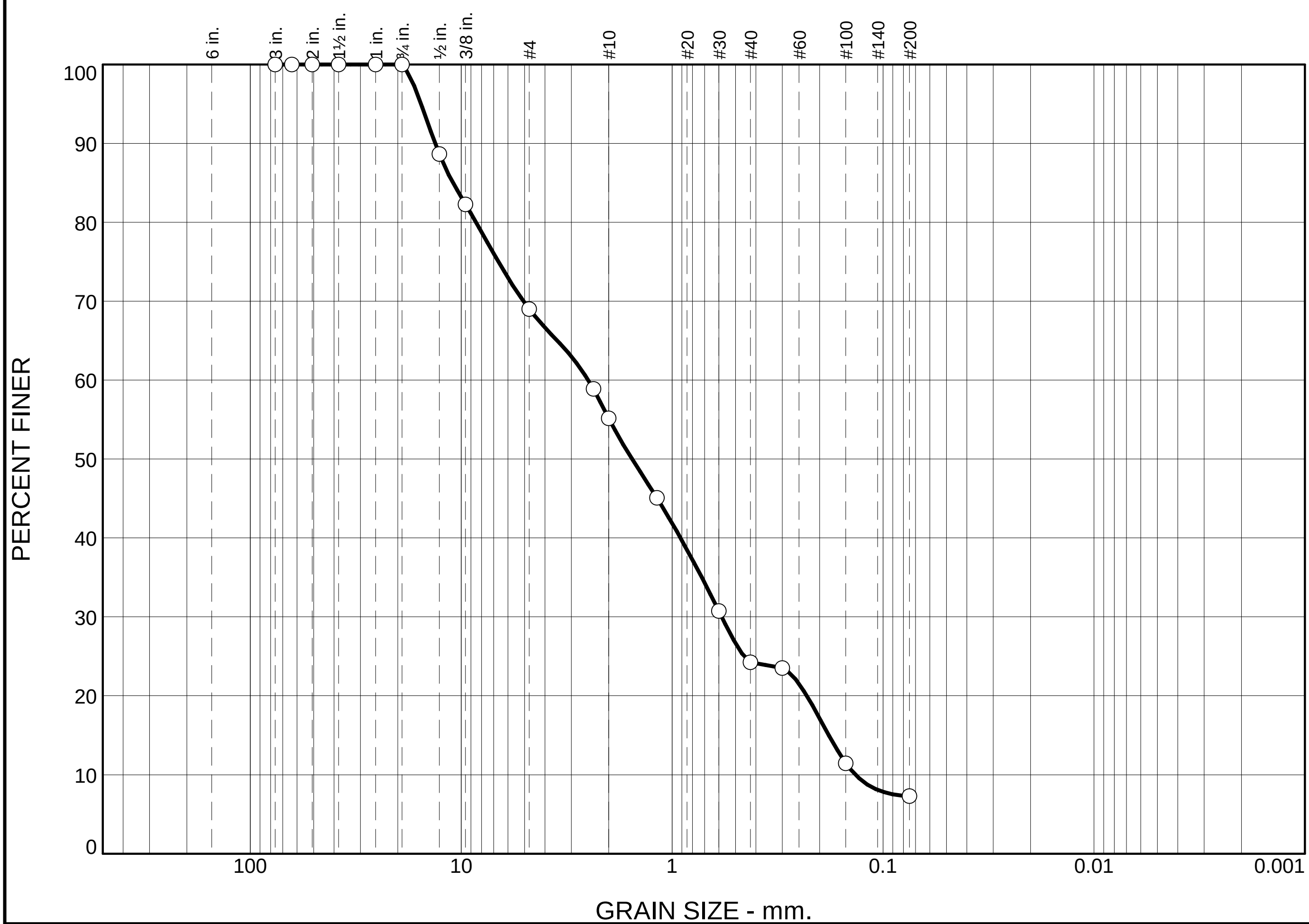
Sample No.: L-309-11 **Source of Sample:** Whittier Street **Date:** 6/7/11
Location: B-112 / S-1B **Elev./Depth:** 0-2'

GEOTECHNICAL SERVICES, INC.

Goffstown, New Hampshire

Client: Ward Geotechnical Consulting PLLC
Project: Whittier Street
Dover, New Hampshire
Project No: 205365 **Plate**

Particle Size Distribution Report



% +3"	% Gravel			% Sand			% Fines
	Coarse	Medium	Fine	Coarse	Medium	Fine	
0.0	0.0	17.7	27.1	24.5	9.2	14.2	7.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3"	100.0		
2.5"	100.0		
2"	100.0		
1.5"	100.0		
1"	100.0		
0.75"	100.0		
0.5"	88.7		
0.375"	82.3		
#4	69.0		
#8	58.9		
#10	55.2		
#16	45.1		
#30	30.7		
#40	24.2		
#50	23.5		
#100	11.5		
#200	7.3		

* (no specification provided)

Material Description

coarse to fine Sand, trace Silt, and med to fine Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 10.9236 D_{60}= 2.4979 D_{50}= 1.5461
D_{30}= 0.5817 D_{15}= 0.1806 D_{10}= 0.1350
C_u= 18.51 C_c= 1.00

Classification

USCS= AASHTO=

Remarks

Sample No.: L-308-11 **Source of Sample:** Whittier Street **Date:** 5/31/11
Location: B-110 / S-1A **Elev./Depth:** 0.5-2.5'

GEOTECHNICAL SERVICES, INC.

Goffstown, New Hampshire

Client: Ward Geotechnical Consulting PLLC
Project: Whittier Street
Dover, New Hampshire
Project No: 205365 **Plate**