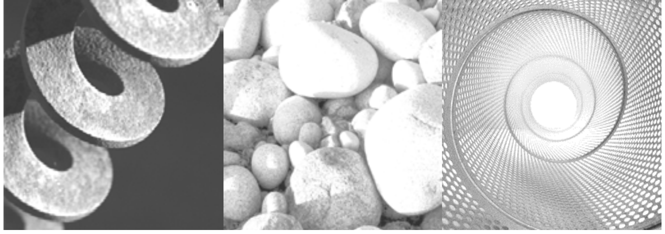

GEI Consultants

Consulting
Engineers and
Scientists

**Geotechnical Report**

# Brandy Brow Road over East Meadow River

Haverhill, Massachusetts

**Submitted to:**
Greenman-Pedersen, Inc.
181 Ballardvale Street, Suite 202
Wilmington, MA 01887


**Submitted by:**
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June 17, 2024
Project 2400197

COMMONWEALTH OF MASSACHUSETTS
LAUREEN M. BEINTUM
CIVIL
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REGISTERED
PROFESSIONAL ENGINEER


Laureen M. Beintum, P.E.
Senior Project Manager

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# Executive Summary

This report presents the results of the subsurface explorations and our geotechnical recommendations for the proposed culvert replacement at Brandy Brow Road over East Meadow in Haverhill, Massachusetts. We understand that this project is being funded by the Culvert Replacement Municipal Assistance Grant Program, and that design and construction must be completed by June 30, 2024. The existing culvert is a single span approximately 15 feet wide (i.e., parallel to Brandy Brow Road) and 32 feet long. The superstructure consists of a cast-in-place concrete deck supported by concrete-encased steel beams. The abutments are constructed from dry-laid rubble masonry. The north wingwalls are mortared cut granite blocks, and the south wingwalls are reinforced concrete. No record plans of the culvert have been located.

Currently, a precast 3-sided frame replacement alternative with a 34-foot clear span is being considered, with cast-in-place concrete retaining walls. We understand that if the hydraulic study requires reducing the span length to 20 feet or less, a box culvert alternative may also be considered. In January 2024, Northern Drill Service, Inc of Northborough, Massachusetts drilled one boring (B-1) east of the existing culvert. The drillers performed Standard Penetration Testing. The boring encountered a soil profile consisting of a layer of loose to very dense sand and gravel. The boring was terminated at a depth of 51 feet.

Groundwater was measured in the completed boring at depth of about 8.6 feet and is likely controlled by the water level in East Meadow River.

We recommend that the supporting culvert and supporting wingwalls bear on the sand and gravel at the project site. Recommended soil properties and factored bearing resistances are provided for design.

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

# 1. Introduction

## 1.1 Purpose

This report presents the results of the subsurface explorations, and our geotechnical recommendations for the replacement of the culvert carrying East Meadow River under Brandy Brow Road in Haverhill, Massachusetts.

## 1.2 Scope

We performed the following scope of work:

- Reviewed available published geologic data.
- Provided full-time observation of the boring.
- Collected samples from the stream to perform laboratory testing to support the hydraulic study for the project.
- Evaluated the soil conditions and developed geotechnical design and construction recommendations.
- Prepared this report.

## 1.3 Authorization

Greenman-Pedersen, Inc. (GPI) authorized our work for this project in a Subconsultant Agreement between GPI and GEI dated January 3, 2024.

## 1.4 Project Personnel

The following personnel at GEI were involved with the field investigations, evaluations, recommendations, and preparation of this report:

| | |
|---|---|
| Laureen M. Beintum, P.E. | Senior Project Manager |
| Shradha Poudyal | Geotechnical Project Professional |
| Annika Han | Geotechnical Project Professional |
| Richard F. Tobin, P.E. | In-house Reviewer |

# 2. Site and Project Description

## 2.1 Site and Project Description

The project is located on Brandy Brow Road over East Meadow River in Haverhill, Massachusetts (Fig. 1). The existing culvert is a single span approximately 15 feet wide (i.e., parallel to Brandy Brow Road) and 32 feet long. The superstructure consists of a cast-in-place concrete deck supported by concrete-encased steel beams. The abutments are constructed from dry-laid rubble masonry. The north wingwalls are mortared cut granite blocks, and the south wingwalls are reinforced concrete. No record plans of the culvert have been located.

The culvert has been closed to traffic for approximately 20 years, with barricades and locked gates erected to prevent vehicular access. The barricades/gates are located near 378 and 284 Brandy Brow Road residences. The culvert was last inspected in 2010 and classified as in an advanced state of disrepair. Numerous parts of the substructure abutments and wingwalls were reported as having significant differential settlements and large cracks. Various substructure component conditions (including the bridge seats) were reported as severe, critical, or unstable, with a danger of localized collapse.

We understand that this project is being funded by the Culvert Replacement Municipal Assistance Grant Program, and that design and construction must be completed by June 30, 2024. Currently, a precast 3-sided frame replacement alternative with a 34-foot clear span is being considered. We understand that if the hydraulic study requires reducing the span length to 20 feet or less, a box culvert alternative may also be considered. The bottom of the culvert is anticipated to be approximately at El. 56.

## 2.2 Project Design Basis

Our recommendations conform to the AASHTO LRFD Bridge Design Specifications, 9th Edition (2020), and AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition (2011), with interim revisions through 2022. Our recommendations are also based on the Massachusetts Department of Transportation (MassDOT) LRFD Bridge Manual, 2020 Edition.

## 2.3 Elevation Datum

Elevations in this report are in feet and are referenced to the 1988 North American Vertical Datum (NAVD 1988).

# 3. Subsurface Conditions

## 3.1 Surficial Geology

The U.S. Geological Survey (USGS) Scientific Investigation Map 3402, Quadrangle 122 "Surficial Materials Map of the Haverhill Quadrangle, Massachusetts," compiled by Byron D. Stone and Mary L. DiGiacomo-Cohen, 2018, identifies the native soils underlying the project site as "coarse deposits", consisting of gravel deposits, sand and gravel deposits, and sand deposits. A number of gravel pits are called out on the map to the north, west, east, and south. The USGS bedrock geology map of Massachusetts ("Bedrock Geologic Map of Massachusetts," authored by Zen et. Al., 1983) identifies the bedrock in the area as being of the Berwick Formation (sandstone, siltstone, and muscovite schist) and Chelmsford Granite.

The USGS maps are shown in Appendix C.

## 3.2 Subsurface Exploration Program

In January 2024, Northern Drill Service, Inc of Northborough, Massachusetts drilled one boring (B-1) east of the existing culvert, as shown on Fig. 2. The boring was advanced to a depth of 51 feet, using driven flush joint casing and rotary drilling techniques. Standard Penetration Testing (SPT) with split-spoon sampling was performed using an automatic hammer. A GEI engineer observed and logged the drilling. After the boring was completed, the hole was backfilled with drill cuttings and bentonite. The road surface was repaired with a cold asphalt patch. The boring log is provided in Appendix A.

A GEI representative collected two bulk sediment samples in East Meadow River at the approximate locations shown in Fig. 2 – one sample approximately 10 feet upstream of the culvert and one sample approximately 10 feet downstream of the culvert. The samples were composed of mainly sand and gravel.

## 3.3 Sample Review

Soil samples from the boring were examined at GEI's office. The results of this examination were incorporated in the boring log in Appendix A.

## 3.4 Laboratory Testing

We performed grain size analyses (ASTM D6913) on the two sediment samples obtained from the East Meadow River (about 10 feet upstream of the culvert and one about 10 feet downstream of the culvert) and provided the results to GPI in a February 12, 2024 letter.

The letter, which includes the grain size distribution curves for these samples, is provided in Appendix B.

## 3.5 Subsurface Conditions

The strata encountered in the one boring performed for this project are described below in order of increasing depth. The descriptions are based on observations of soil samples collected from boring B-1. Subsurface conditions elsewhere in the project vicinity may differ from those indicated below.

SPTs were performed using an automatic hammer. The soil description below refers to $N_{60}$, which is the measured Standard Penetration Resistance (N-value) corrected to represent a hammer system that is 60 percent efficient, assuming an automatic hammer efficiency of 80 percent. Uncorrected N-values are reported on the boring logs in Appendix A.

Asphalt: Four inches of asphalt (pavement) was encountered from the ground surface.

Sand and Gravel: Fine to coarse sand and gravel with varying amounts of inorganic silt was encountered below the thin layer of asphalt from about El. 70 to El. 19. The $N_{60}$ values ranged between 5 and 55 blows per foot, indicating loose to dense soil.

## 3.6 Groundwater Levels

Groundwater was measured at a depth of 8.6 feet (~El. 62) below the ground surface at the end of drilling in B-1. The water level may not have reached equilibrium in the boring at the time it was measured; therefore, this measurement may not accurately reflect the true groundwater level. Significantly different groundwater levels may occur at other times and locations.

# 4. Design Recommendations

## 4.1 Soil Properties

Recommended soil properties for design are presented in Table 1. We selected these values based on published correlations to SPT N-values (Appendix C), our review of the soil descriptions, and our engineering judgment.

## 4.2 Foundation Design

In our opinion, the Sand and Gravel underlying the site is a suitable bearing layer for the replacement culvert and wingwalls.

Fig. 3 presents factored bearing resistances versus effective footing width for the design of the culvert and retaining wall foundations. These recommendations are based on a design embedment of four feet, to account for frost protection. We have provided service limit bearing resistance curves associated with total settlement of 1-inch and 1½-inches. We anticipate most of this settlement will occur during construction.

Our calculations are provided in Appendix C.

Design of reinforced concrete box culverts should be performed in accordance with Chapter 3 of the MassDOT LRFD Bridge Design Manual (Bridge Design Manual) and Article 12.11 of the AASHTO LRFD Bridge Design Specifications (AASHTO). In addition, the scour design requirements in Section 3.2.10 (MassDOT) and Section 12.6.5 (AASHTO), should be considered during the design process, in conjunction with the current estimated scour depths from the hydraulic engineer.

Any fill or unsuitable soils, such as organics, within the zone of influence of any of the footings should be removed per the recommendations in Section 5.1.

For analysis of sliding, we recommend using a nominal (ultimate) friction factor of 0.67 to represent cast-in-place concrete on the Sand and Gravel layer, and 0.54 to represent pre-cast concrete on Sand and Gravel. Applicable resistance factors for sliding are provided in Table 2 for the Strength, Service, and Extreme Limit Load States for both cast-in-place concrete and precast concrete.

## 4.3 Seismic Design Information

The soil conditions indicate that the site classifies as Site Class D. The resulting site coefficients for peak ground acceleration [$F_{PGA}$], short-period range [$F_A$], and long-period range [$F_v$] are 1.6, 1.6, and 2.4, respectively.

Based on the maps in the AASHTO "Guide Specifications for LRFD Seismic Bridge Design," we recommend the following seismic parameters:

- Horizontal Peak Ground Coefficient (PGA) = 0.119
- Horizontal Response Spectral Coefficient (period = 0.2 sec) ($S_s$) = 0.197
- Horizontal Response Spectral Coefficient (period = 1.0 sec) ($S_1$) = 0.038

Application of the above site coefficients results in the following recommended coefficients for development of design response spectra:

- Response Spectral Acceleration, $A_s$ = 0.190
- Design Spectral Acceleration Coefficient at 0.2 second period, $S_{DS}$ = 0.315
- Design Spectral Acceleration Coefficient at 1.0 second period, $S_{D1}$ = 0.091

This site falls into Seismic Design Category (SDC) A, based on the 1-second-period design spectral acceleration being less than 0.15. In the updated (2020) MassDOT LRFD Bridge Manual, design procedures for bridges in SDC A are in Section 3.4.4.

We did not check liquefaction because the AASHTO Seismic Guide Specifications (Section 6.8) state that liquefaction potential need not be evaluated for sites in SDC A.

# 5. Construction Recommendations

## 5.1 Excavation and Dewatering

All excavations should be made in accordance with OSHA standards. Excavation should be performed so as not to undermine or impact any existing structures or utilities. Any necessary excavation support system should be designed by a Massachusetts-registered professional engineer experienced in excavation support design. The engineer should be engaged by the contractor and submit the designs for review before installation.

Groundwater will likely be encountered during foundation excavations. The contractor should be prepared to manage and control groundwater during excavation and to control surface water from entering excavations to provide a dry and stable subgrade. The contractor should be responsible for selecting the dewatering methods based on their proposed methods and equipment used for excavation. For excavations in soil, groundwater levels should be maintained at least two feet below excavation subgrade levels at all times, or deeper if necessary, to maintain stable conditions.

The dewatering plan and systems should be designed by an experienced professional engineer registered in Massachusetts and retained by the contractor. The contractor should submit a dewatering plan for review prior to the start of excavation. Dewatering efforts must satisfy requirements of local, state, and federal environmental and conservation authorities.

## 5.2 Preparation of Subgrade

Prior to foundation construction on soil, the foundation subgrade should be compacted with at least 4 passes of a smooth-wheel vibratory compactor weighing at least 10,000 lbs. In confined areas, compact with a vibratory plate compactor that weighs at least 200 lbs. and imparts an impact load of at least 2.5 tons.

At foundation locations, unsuitable soils and soils that do not become firm under proof compaction should be removed and replaced with compacted Gravel Borrow for Bridge Foundations or Crushed Stone for Bridge Foundations to create a competent bearing surface. If Crushed Stone is used, a geotextile should be placed between the Crushed Stone and the soil below and to the sides of the Crushed Stone.

Where footing subgrades are at or near the groundwater level, static compaction may be recommended by the Field Engineer in lieu of vibratory compaction.

Concrete for footings on soil may be placed directly on the soil subgrade. Bearing surfaces should be free of standing water, frost, and loose soil before placement of reinforcing steel and concrete. Areas of the subgrade disturbed by traffic, frost, or surface water should be re-compacted. We recommend that a qualified Geotechnical Engineer evaluate the soil subgrades of shallow foundations prior to placement of footings and fill.

## 5.3 Backfilling

Gravel Borrow backfill (MassDOT Standard Specification No. M1.03 Type b) should be placed behind the abutments and wingwalls, extending at least one foot beyond the back of the structure, as shown in Drawing No. 3.6.13 of the MassDOT "*LRFD Bridge Design Manual.*" Other backfill (e.g., to fill excavations associated with construction or removal of existing foundations or utilities) should consist of Ordinary Borrow (MassDOT Standard Specification No. M1.01.0) or Gravel Borrow.

Fill for the roadway, backfill behind the culvert, backfill of excavations, and crushed stone for scour protection should be placed and compacted in accordance with MassDOT Standard Specifications for Highway and Bridges Section 150. However, we recommend that compaction in areas too small for a smooth wheel vibratory compactor, or within 5 feet of walls, be performed using a vibratory walk-behind roller or plate compactor (weighing at least 200 lbs. imparting an impact load of at least 2.5 tons), with soil placed in maximum 6-inch-loose lifts.

## 5.4 Re-Use of Existing Materials

Based on the soil descriptions on the boring logs, some of the existing on-site granular soils to be excavated for replacement of the superstructure may meet the requirements for Ordinary Borrow. Suitability for reuse can be confirmed by testing samples to evaluate if the soil in question meets the MassDOT requirements for Ordinary Borrow. The Contractor should be aware that materials that are not free draining may be difficult to compact in wet weather.

## 5.5 Freezing Conditions

If construction is performed during freezing weather, special precautions will be required to prevent the soil subgrades from freezing. Freezing of the soil beneath foundations and pavements during construction may result in heave and subsequent settlement of the structure.

All soil subgrades should be free of frost before foundation construction. Frost-susceptible soils that have frozen should be removed and replaced with compacted Gravel Borrow. The foundation and the soil adjacent to the foundation should be insulated until they are backfilled.

Soil placed as fill should be free of frost, as should the ground on which it is placed.

If shallow foundations are constructed during the winter, precautions should be taken to prevent freezing of the underlying soil while constructing the shallow foundations.

# 6. 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 evaluate 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 boring. The nature and extent of variations across the site 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.

# Tables

**Table 1. Recommended Soil Properties**
Brandy Brow Road over East Meadow River
Haverhill, MA

| Layer/Soil Type | Unit Weight, $\gamma$ (pcf) | Friction Angle, $\phi$ (deg) | Undrained Shear Strength (psf) | Earth Pressure Coefficients[(1)] |
|---|---|---|---|---|
| Sand and Gravel | 125/130[(2)] | 34 | - | $K_a$=0.26<br>$K_0$=0.44<br>$K_p$=6.2 |

**Notes:**

1. Notes for use of Earth Pressure Coefficients:
   - Recommended earth pressure coefficients are associated with horizontal backfill in front and behind the structure with vertical back faces and are in accordance with the recommendations of Section 3.1.6 of the MassDOT LRFD Bridge Manual, which includes guidelines on how to use the earth pressure coefficients in design.
   - For sloping wall face, calculate using the Coulomb method and actual wall slope angle, with the interface friction angle assumed to be half the angle of internal friction of the soil (see AASHTO Figs. 3.11.5.3-1 and C3.11.5.3-1).
   - Where the back of the active wedge crosses multiple soil types, use the higher Ka value.
   - Above Ka values are not applicable for soldier pile and lagging Support of Excavation systems; for soldier pile and lagging systems, increase Ka values by 10 percent.
   - Passive earth pressures are not intended for use where there is a potential for future excavation in front of the structure.

2 The first value represents a moist condition above the water table; the second value represents a saturated condition below the water table.

3 Refer to Fig. 3.11.5.6-5 and Fig. 3.11.5.6-7 in Article 3.11.5.6 of the AASHTO LRFD Bridge Design Specifications for simplified earth pressure distributions for cohesive soils.

4 Seismic earth pressure coefficients are not included because the bridge is classified as Seismic Design Category A, and seismic soil forces need not be included in design of the substructures, except for semi-integral abutments and MSE walls supporting stub abutments (per the MassDOT LRFD Bridge Manual, Paragraphs 3.4.4.3, 3.4.6, and 3.4.9).

**Table 2. Resistance Factors**
**Brandy Brow Road over East Meadow River**
**Haverhill, MA**

| Load Case | Strength Limit State(2) | Service Limit State(3) | Extreme Limit State(4) |
|---|---|---|---|
| *Culvert Footings* | | | |
| Bearing resistance of shallow foundations on rock or soil | 0.45 | 1.0 | 1.0 |
| Sliding (Cast-in-Place/Pre-cast foundation) | 0.8/0.9 | 1.0 | 1.0 |
| Global Stability(5) | 0.75/0.65(6) | NA | NA |
| *Cast-in-Place Retaining Walls* | | | |
| Bearing resistance | 0.55 | 1.0 | 0.8 |
| Sliding | 1.0 | 1.0 | 1.0 |
| Global Stability(5) | 0.75/0.65(6) | NA | NA |

**General Notes:**

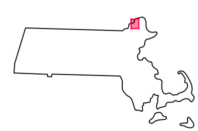
1. Resistance factors above were obtained from the 2020 AASHTO LRFD Bridge Design Specifications (AASHTO).
2. The strength limit state resistance factors for bearing and sliding of shallow foundations were obtained from AASHTO Table 10.5.5.2.2-1 and Table 11.5.7-1.
3. Both AASHTO Sections 10.5.5.1 and 11.5.7 indicate that a resistance factor of 1.0 should be used for bearing resistance and sliding at the service limit state.
4. AASHTO Sections 10.5.5.3 and 11.5.8 provide resistance factors for the Extreme Limit State.
5. Per AASHTO Articles 10.5.5.2.1 and 11.6.3.7, global (overall) stability analysis is required using Strength I load combination with a Load Factor of 1.0 on vertical earth loading and Load Factors from Table 3.4.1-1 for other loads. Global stability analysis is not required for the Extreme Event Limit State, because seismic analysis of abutments and walls is not necessary, except for semi-integral abutments and MSE walls supporting stub abutments (per Paragraphs 3.4.4.3, 3.4.6, and 3.4.9 of the MassDOT LRFD Bridge Manual).
6. The resistance factor for global stability was obtained from AASHTO Section 11.6.3.7. Resistance factor = 0.65 at abutments and retaining walls where soil stratigraphy and soil properties are not well defined; resistance factor = 0.75 for abutments and retaining walls where stratigraphy and properties are well defined.

# Figures


0 1000 2000 4000 6000

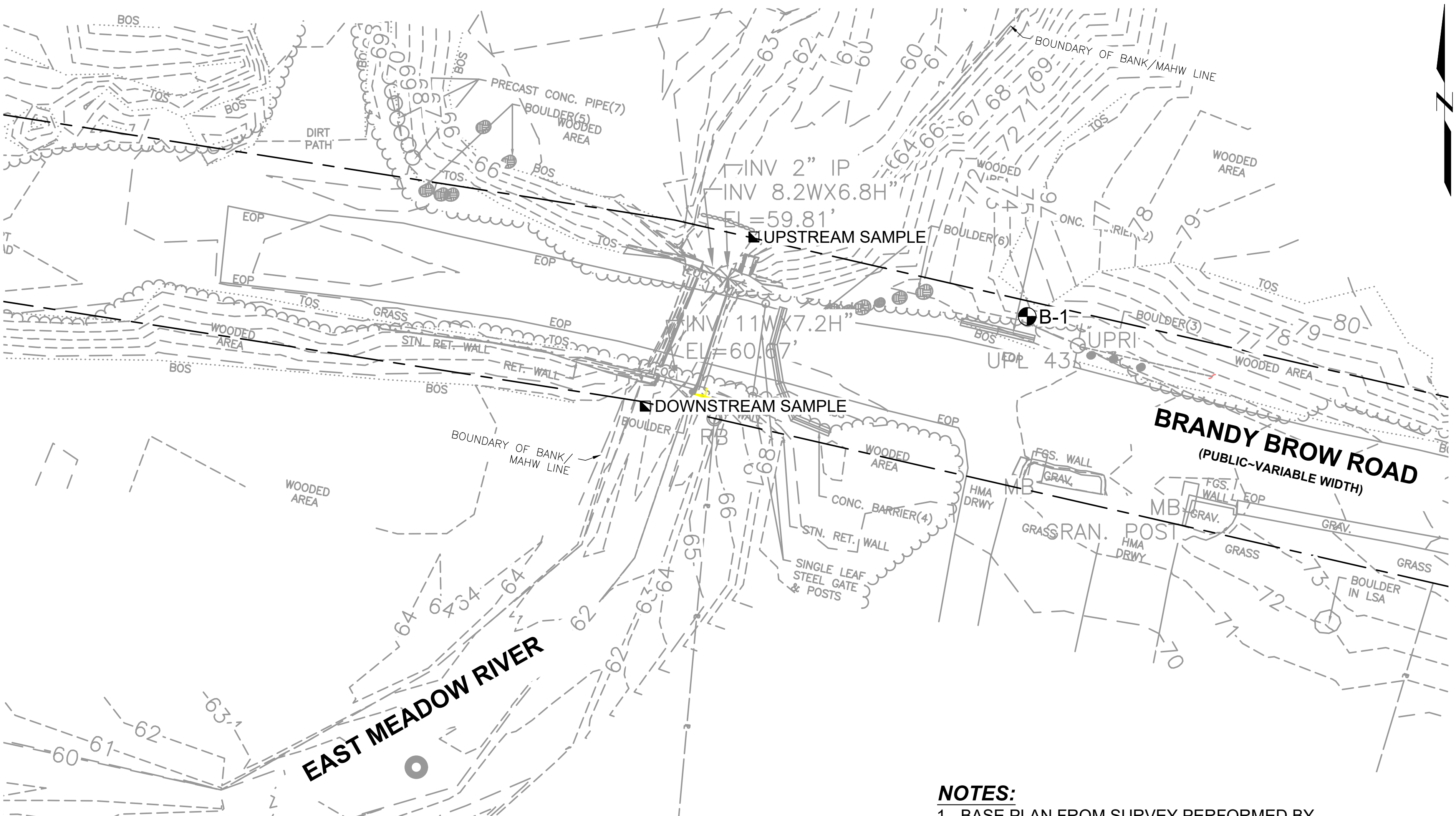
SCALE, FEET

This Image is from U.S.G.S. Topographic 7.5 Minute Series,
Haverhill, MA Quadrangle, 2021.
Datum is North Americal Vertical Datum of 1988 (NAVD88).
Contour Interval is 10 feet.




| | | |
|---|---|---|
| Brandy Brow Road over East Meadow River<br>Haverhill, Massachusetts | GEI Consultants | SITE LOCATION MAP |
| Greenman-Pedersen, Inc.<br>Wilmington, Massachusetts | Project 2400197 | June 2024 Fig. 1 |

B:\Working\GREENMAN PEDERSEN\2400197 Brandy Brow Rd over E Meadow R_Haverhill\00_CAD\Figures\Gt\2400197-01




**LEGEND:**

B BORING, GEI 2024

SEDIMENT SAMPLE, GEI 2024

0 40 80

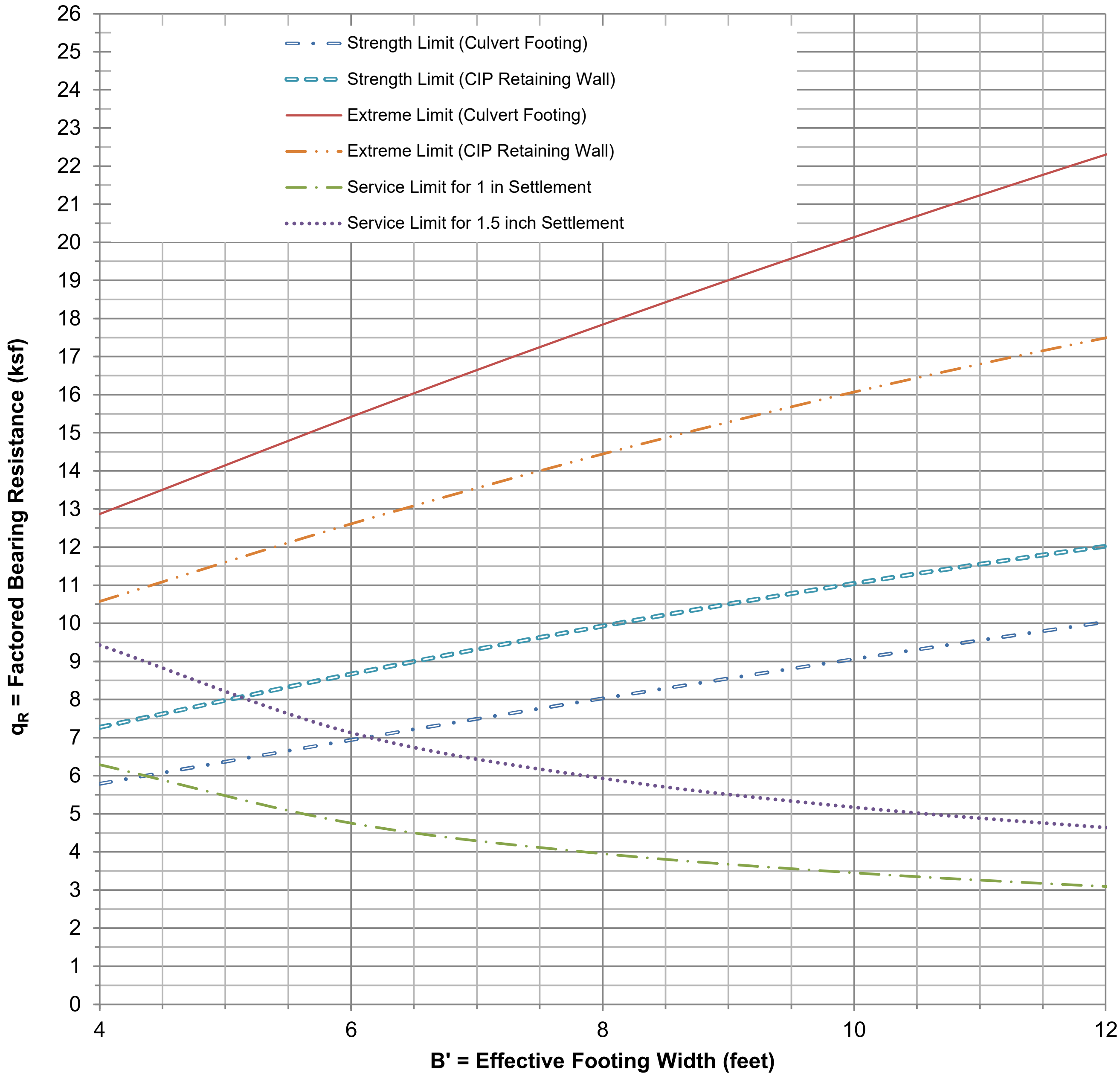
SCALE, FEET

**NOTES:**

1. BASE PLAN FROM SURVEY PERFORMED BY GREENMAN-PEDERSEN, INC. IN JANUARY 2024.
2. VERTICAL DATUM REFERENCED TO NATIONAL VERTICAL DATUM OF 1988 (NAVD 88) IN FEET.

| | | |
|---|---|---|
| Brandy Brow Road over East Meadow River<br>Haverhill, Massachusetts | GEI Consultants | EXPLORATION LOCATION PLAN |
| Greenman-Pedersen, Inc.<br>Wilmington, Massachusetts | Project 2400197 | June 2024 Fig. 2 |

CLINGEN, PAUL B:\Working\GREENMAN PEDERSEN\2400197 Brandy Brow Rd over E Meadow R_Haverhill\00_CAD\Figures\Gt\2400197-03.dwg - 4/3/2024




Notes:

1. The above resistances are based on methods provided in the 2020 AASHTO LRFD Bridge Specifications.
2. The resistances above are based on 4 feet of embedment and level ground (i.e., no sloping ground) below the existing brook bottom.
3. Groundwater was assumed to be at ground surface, based on nearby borings.
4. The Strength Limit values above are based on a Resistance Factor of 0.45 for abutments on shallow foundation and 0.55 for gravity and semi-gravity retaining walls. The Extreme Limit values above are based on Resistance Factors of 1.0 for abutments on shallow foundation and 0.8 for gravity and semi-gravity retaining walls. The Service Limit values are based on a Resistance Factor of 1.0.
5. The Service Limit resistance curve can be referenced to evaluate settlement associated with a net increase in loading.

| | | |
|---|---|---|
| Brandy Brow Road over East Meadow River<br>Haverhill, MA | GEI Consultants | FACTORED BEARING RESISTANCE<br>VS. EFFECTIVE FOOTING WIDTH |
| Greenman-Pedersen, Inc.<br>Wilmington, Massachusetts | Project No. 2400197 | June 2024 Fig. 3 |

# Appendix A

## GEI Boring Log

# BORING B-1



**BORING INFORMATION**

**LOCATION:** See Site Plan
**GROUND SURFACE EL. (ft):** ~70
**DATE START/END:** 1/23/2024 - 1/23/2024
**VERTICAL DATUM:** Project Datum
**DRILLING COMPANY:** Northern Drill Service, Inc.
**TOTAL DEPTH (ft):** 51.0
**DRILLER NAME:** T. Tucker
**LOGGED BY:** A. Han
**RIG TYPE:** Mobile B-57

**DRILLING INFORMATION**

**HAMMER TYPE:** Automatic
**CASING I.D./O.D.:** 4 inch/ 4.25 inch
**CORE BARREL TYPE:**
**AUGER I.D./O.D.:** NA / NA
**DRILL ROD O.D.:** NM
**CORE BARREL I.D./O.D.** NA / NA
**DRILLING METHOD:** Drive and Wash
**WATER LEVEL DEPTHS (ft):** ▼ 8.6 1/23/2024 11:45 am

**ABBREVIATIONS:**
Pen. = Penetration Length
Rec. = Recovery Length
RQD = Rock Quality Designation = Length of Sound Cores>4 in / Pen.,%
WOR = Weight of Rods
WOH = Weight of Hammer
S = Split Spoon Sample
C = Core Sample
U = Undisturbed Sample
SC = Sonic Core
DP = Direct Push Sample
HSA = Hollow-Stem Auger
Qp = Pocket Penetrometer Strength
Sv = Pocket Torvane Shear Strength
LL = Liquid Limit
PI = Plasticity Index
PID = Photoionization Detector
I.D./O.D. = Inside Diameter/Outside Diameter
NA, NM = Not Applicable, Not Measured
Blows per 6 in.: 140-lb hammer falling 30 inches to drive a 2-inch-O.D. split spoon sampler.

| Elev. (ft) | Depth (ft) | Sample No. | Sample Depth (ft) | Pen./ Rec. (in) | Blows per 6 in. or RQD | Drilling Remarks/ Field Test Data | Layer Name | Soil and Rock Description |
|---|---|---|---|---|---|---|---|---|
| | | S1 | 0 to 2 | 24/20 | 60-73-26-27 | | GRAVEL | S1 (0"-4"): Asphalt. S1 (4"-20"): Moist, very dense, brown, FINE TO COARSE GRAVEL, some fine to coarse sand, trace inorganic silt. |
| | | S2 | 2 to 4 | 24/12 | 20-19-14-11 | | SAND | S2: Moist, dense, brown, FINE TO COARSE SAND, some fine to coarse gravel, trace inorganic silt. |
| | 5 | S3 | 4 to 6 | 24/10 | 8-7-6-4 | Drove casing and rollerbit to 4'. | SAND | S3: Moist, medium dense, orange, FINE TO COARSE SAND, some fine to coarse gravel, trace inorganic silt. |
| | | S4 | 6 to 8 | 24/11 | 4-3-2-5 | | SAND | S4: Moist, loose, orange, FINE TO COARSE SAND, some fine to coarse gravel, trace inorganic silt. |
| | | S5 | 8 to 10 | 24/9 | 6-8-10-8 | | GRAVEL | S5: Wet, medium dense, orange, FINE TO COARSE GRAVEL, some fine to coarse sand. |
| | 10 | S6 | 10 to 12 | 24/9 | 6-9-12-12 | Drove casing to 10'. | GRAVEL | S6: Wet, medium dense, brown, FINE TO COARSE GRAVEL, some fine to coarse sand, trace inorganic silt. |
| | | S7 | 12 to 14 | 24/13 | 16-7-9-9 | | GRAVEL | S7: Moist, medium dense, brown, FINE TO COARSE GRAVEL, some fine to coarse sand, trace inorganic silt. |
| | 15 | S8 | 14 to 16 | 24/9 | 7-12-9-11 | Drove casing and rollerbit to 14'. | GRAVEL | S8: Wet, medium dense, brown, FINE TO COARSE GRAVEL, some fine to coarse sand. |
| | 20 | S9 | 19 to 21 | 24/4 | 4-2-2-2 | Drove casing and rollerbit to 19'. | SAND | S9: Wet, loose, grey, FINE TO COARSE SAND. |

**NOTES:**
Driller notes that he maintained a head on the casing as rods were pulled during sampling.

**PROJECT NAME:** Brandy Brow Road Over East Meadow River
**CITY/STATE:** Haverhill, Massachusetts
**GEI PROJECT NUMBER:** 2400197

GEI Consultants

| Elev. (ft) | Depth (ft) | Sample Information | | | | Drilling Remarks/ Field Test Data | Layer Name | Soil and Rock Description |
|---|---|---|---|---|---|---|---|---|
| | | Sample No. | Depth (ft) | Pen./ Rec. (in) | Blows per 6 in. or RQD | | | |
| | 25 | S10 | 24 to 26 | 24/9 | 2-5-4-5 | Drove casing and rollerbit to 24'. | SAND | S10: Similar to S9. |
| | 30 | S11 | 29 to 31 | 24/0 | 9-6-7-8 | Drove casing and rollerbit to 29'. Redrove S11 with 3" spoon. | GRAVEL | S11 (Redrive): Wet, medium dense, grey, FINE GRAVEL, some fine to coarse sand. |
| | 35 | S12 | 34 to 36 | 24/2 | 6-10-11-9 | Drove casing and rollerbit to 34'. | GRAVEL | S12: Similar to S9. |
| | 40 | S13 | 39 to 41 | 24/10 | 6-7-8-8 | Drove casing and rollerbit to 39'. | GRAVEL | S13: Wet, medium dense, grey, FINE TO COARSE GRAVEL, trace fine to coarse sand. |
| | 45 | S14 | 44 to 46 | 24/14 | 4-3-5-7 | Drove casing and rollerbit to 44' | GRAVEL | S14: Wet, loose, grey, FINE GRAVEL, some fine to coarse sand. |
| | 50 | S15 | 49 to 51 | 24/15 | WOH-2-3-4 | Drove casing and rollerbit to 49'. | GRAVEL | S15: Similar to S14. |
| | 55 | | | | | Driller terminated hole due to time constraints. | | Bottom of borehole at 51.0'. Backfilled with soil cuttings and bentonite. Patched with asphalt. |

**NOTES:**
Driller notes that he maintained a head on the casing as rods were pulled during sampling.

# Appendix B

## Sediment Sample Letter

GEI Consultants

Consulting Engineers and Scientists

February 12, 2024
Project 2400197

Ms. Kim Armstrong, P.E.
Greenman-Pedersen, Inc.
181 Ballardvale Street, Suite 202
Wilmington, MA 01887

Dear Ms. Armstrong:

Re: **Sediment Sample Test Results**
**Culvert Replacement – Brandy Brow Road over East Meadow River**
**Haverhill, Massachusetts**

This letter presents the results of our laboratory testing on the two sediment samples collected on January 23, 2024, from the riverbed upstream and downstream of the culvert located near 284 Brandy Brow Road in Haverhill, MA. The approximate locations of the samples are shown on the attached Figure.

Samples were tested by GEI in our Woburn geotechnical laboratory for particle size analyses in accordance with ASTM D6913. The test results are attached. We understand that these test results will be used in the preparation of the Hydraulic Study Report for this project.

Please call Laureen at 781-721-4022 or Rich at 781-721-4084 if you have any questions.

Sincerely,

GEI CONSULTANTS, INC.

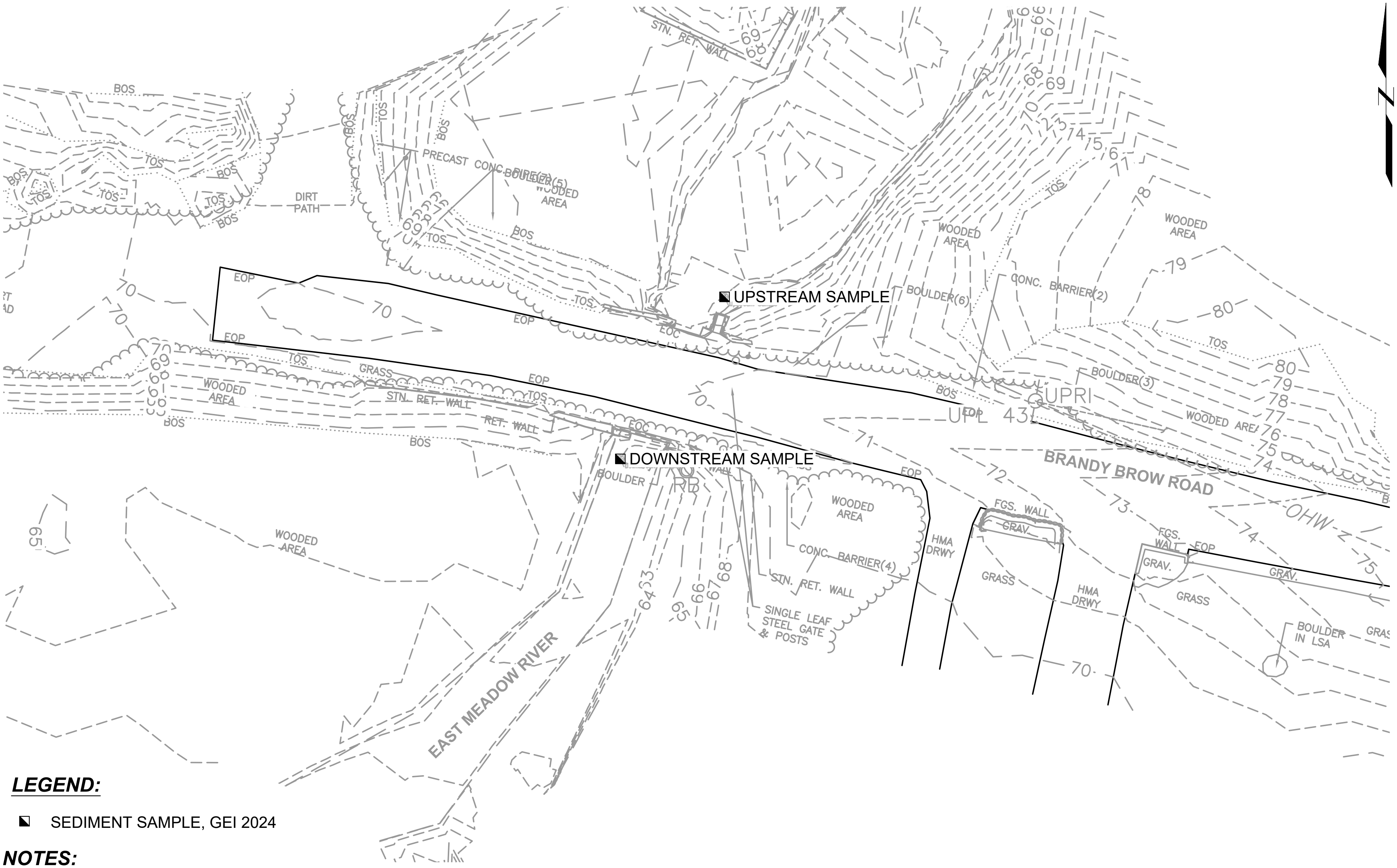
| | |
|---|---|
| Laureen M. Beintum, P.E. | Rich F. Tobin, P.E. |
| Senior Project Manager | Senior Project Manager |

SP:LMB/RFT:jam
c: Ryan Melchionno, Greenman-Pedersen, Inc.

Attachments: Fig.1 – Sediment Sample Location Plan
Particle Size Distribution Reports (4 pages)

B:\Working\GREENMAN PEDERSEN\2400197 Brandy Brow Rd over E Meadow R_Haverhill\10_Sediment Transmittal\Brandy Brow Rd over E Meadow R_Haverhill Sediment Sample Test Results.docx

# Attachments




**LEGEND:**

SEDIMENT SAMPLE, GEI 2024

**NOTES:**

1. BASE PLAN PROVIDED BY GREENMAN-PEDERSEN, INC. FEB. 8, 2024.

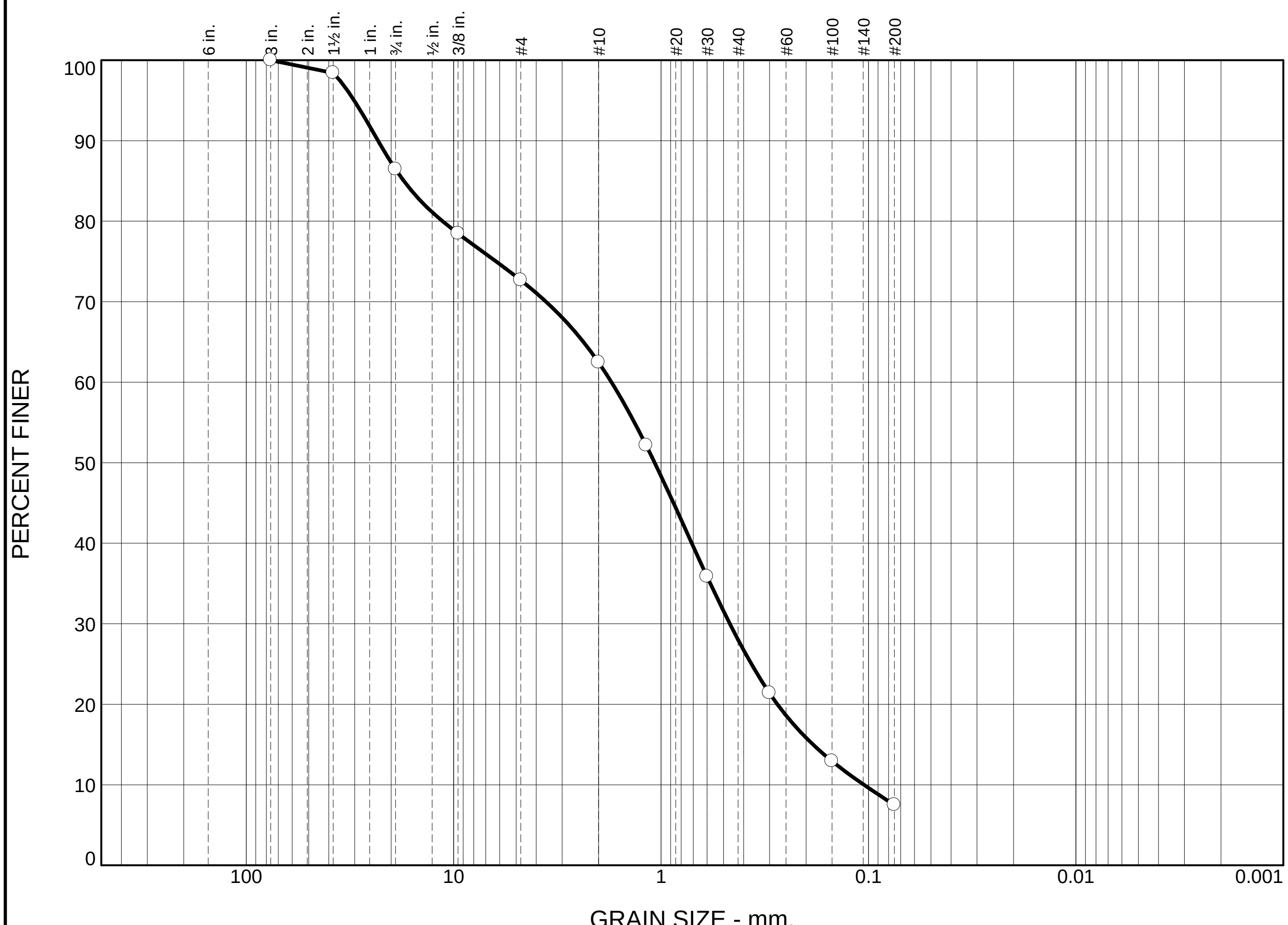
0 40 80

SCALE, FEET

| | | |
|---|---|---|
| Culvert Replacement at Brandy Brow Road<br>over East Meadow River<br>Haverhill, Massachusetts | GEI Consultants | SEDIMENT SAMPLE LOCATION PLAN |
| Greenman-Pedersen, Inc.<br>Wilmington, Massachusetts | Project 2400197 | February 2024 Fig. 1 |

CLINGEN, PAUL B:\Working\GREENMAN PEDERSEN\2400197 Brandy Brow Rd over E Meadow R_Haverhill\00_CAD\Figures\Gt\2400197-02.dwg - 2/12/2024

# Particle Size Distribution Report




| | % Boulders | % +3" | % Gravel | | | % Sand | | % Fines |
|---|---|---|---|---|---|---|---|---|
| | | | Coarse | Medium | Fine | Coarse | Fine | |
| ○ | 0.0 | 0.0 | 8.2 | 13.3 | 16.0 | 34.5 | 20.5 | 7.5 |
| | | | | | | | | |
| | | | | | | | | |

| | LL | PL | $D_{85}$ | $D_{60}$ | $D_{50}$ | $D_{30}$ | $D_{15}$ | $D_{10}$ | $C_c$ | $C_u$ |
|---|---|---|---|---|---|---|---|---|---|---|
| ○ | | | 17.3833 | 1.7337 | 1.0734 | 0.4661 | 0.1852 | 0.1052 | 1.19 | 16.48 |
| | | | | | | | | | | |
| | | | | | | | | | | |

| Material Description | USCS | AASHTO |
|---|---|---|
| ○ FINE TO COARSE SAND, some fine to coarse gravel, trace inorganic silt. | SW-SM | |

**Project No.** 2400197 **Client:** Greenman Pedersen

**Project:** Culvert Replacement - Brandy Brow Road over East Meadow River

○ **Source of Sample:** Upstream Sample **Depth:** - **Sample Number:** -

**Remarks:**

○As Received WC=17.5%
Fines classified visually.

GEI Consultants, Inc.
400 Unicorn Park Drive
Woburn, MA 01801

**Figure**

**Tested By:** M. Alstede 2/1/24 **Checked By:** W. Lukas 2/1/24

## GRAIN SIZE DISTRIBUTION TEST DATA

**2/2/2024**

**Client:** Greenman Pedersen
**Project:** Culvert Replacement - Brandy Brow Road over East Meadow River
**Project Number:** 2400197
**Location:** Upstream Sample
**Depth:** - **Sample Number:** -
**Material Description:** FINE TO COARSE SAND, some fine to coarse gravel, trace inorganic silt.
**USCS Classification:** SW-SM
**Testing Remarks:** As Received WC=17.5%
Fines classified visually.
**Tested by:** M. Alstede 2/1/24 **Checked by:** W. Lukas 2/1/24

### Sieve Test Data

| Dry Sample and Tare (grams) | Tare (grams) | Cumulative Pan Tare Weight (grams) | Sieve Opening Size | Cumulative Weight Retained (grams) | Percent Finer |
|---|---|---|---|---|---|
| 4379.38 | 0.00 | 412.08 | 3.0 | 412.08 | 100.0 |
| | | | 1.5 | 481.52 | 98.4 |
| | | | 3/4 | 1005.80 | 86.4 |
| | | | 3/8 | 1354.63 | 78.5 |
| 365.72 | 0.00 | 198.46 | #4 | 225.50 | 72.7 |
| | | | #10 | 273.14 | 62.5 |
| | | | #16 | 321.06 | 52.2 |
| | | | #30 | 397.12 | 35.8 |
| | | | #50 | 464.55 | 21.4 |
| | | | #100 | 503.85 | 12.9 |
| | | | #200 | 529.26 | 7.5 |

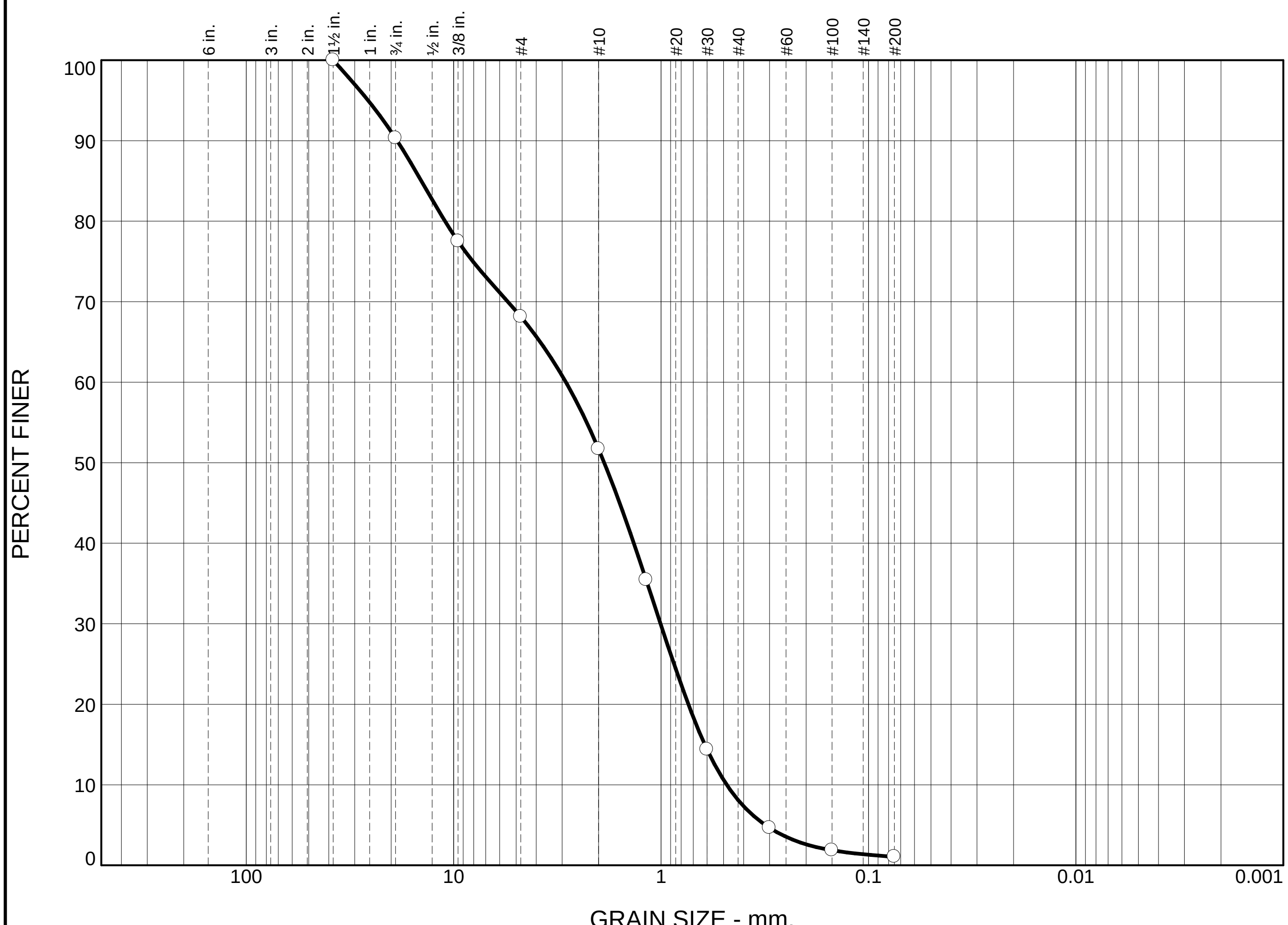
### Fractional Components

| Boulders | Cobbles | Gravel | | | | Sand | | | Fines |
|---|---|---|---|---|---|---|---|---|---|
| | | Coarse | Medium | Fine | Total | Coarse | Fine | Total | |
| 0.0 | 0.0 | 8.2 | 13.3 | 16.0 | 37.5 | 34.5 | 20.5 | 55.0 | 7.5 |

| $D_{10}$ | $D_{15}$ | $D_{20}$ | $D_{30}$ | $D_{50}$ | $D_{60}$ | $D_{80}$ | $D_{85}$ | $D_{90}$ | $D_{95}$ |
|---|---|---|---|---|---|---|---|---|---|
| 0.1052 | 0.1852 | 0.2750 | 0.4661 | 1.0734 | 1.7337 | 11.3195 | 17.3833 | 23.1520 | 30.1760 |

| Fineness Modulus | $C_u$ | $C_c$ |
|---|---|---|
| 3.77 | 16.48 | 1.19 |

# Particle Size Distribution Report




| | % Boulders | % +3" | % Gravel Coarse | % Gravel Medium | % Gravel Fine | % Sand Coarse | % Sand Fine | % Fines |
|---|---|---|---|---|---|---|---|---|
| ○ | 0.0 | 0.0 | 5.2 | 17.3 | 25.8 | 43.6 | 7.0 | 1.1 |
| | | | | | | | | |
| | | | | | | | | |

| | LL | PL | $D_{85}$ | $D_{60}$ | $D_{50}$ | $D_{30}$ | $D_{15}$ | $D_{10}$ | $C_c$ | $C_u$ |
|---|---|---|---|---|---|---|---|---|---|---|
| ○ | | | 14.3141 | 2.8758 | 1.8788 | 1.0057 | 0.6158 | 0.4825 | 0.73 | 5.96 |
| | | | | | | | | | | |
| | | | | | | | | | | |

| Material Description | USCS | AASHTO |
|---|---|---|
| ○ FINE TO COARSE SAND AND FINE TO COARSE GRAVEL, trace inorganic silt. | SP | |

**Project No.** 2400197 **Client:** Greenman Pedersen

**Project:** Culvert Replacement - Brandy Brow Road over East Meadow River

○ **Source of Sample:** Downstream Sample **Depth:** - **Sample Number:** -

**Remarks:**

○As Received WC=12.1%

Fines classified visually


**GEI Consultants, Inc.**
**400 Unicorn Park Drive**
**Woburn, MA 01801**


**Figure**

**Tested By:** S. Larson 2/1/24 **Checked By:** W. Lukas 2/1/24

## GRAIN SIZE DISTRIBUTION TEST DATA

**2/2/2024**

**Client:** Greenman Pedersen
**Project:** Culvert Replacement - Brandy Brow Road over East Meadow River
**Project Number:** 2400197
**Location:** Downstream Sample
**Depth:** -
**Sample Number:** -
**Material Description:** FINE TO COARSE SAND AND FINE TO COARSE GRAVEL, trace inorganic silt.
**USCS Classification:** SP
**Testing Remarks:** As Received WC=12.1%
Fines classified visually
**Tested by:** S. Larson 2/1/24
**Checked by:** W. Lukas 2/1/24

### Sieve Test Data

| Dry Sample and Tare (grams) | Tare (grams) | Cumulative Pan Tare Weight (grams) | Sieve Opening Size | Cumulative Weight Retained (grams) | Percent Finer |
|---|---|---|---|---|---|
| 3184.81 | 0.00 | 412.80 | 1.5 | 412.80 | 100.0 |
| | | | 3/4 | 721.55 | 90.3 |
| | | | 3/8 | 1129.45 | 77.5 |
| 610.99 | 0.00 | 229.63 | #4 | 303.37 | 68.1 |
| | | | #10 | 433.06 | 51.7 |
| | | | #16 | 561.19 | 35.4 |
| | | | #30 | 727.31 | 14.4 |
| | | | #50 | 803.87 | 4.7 |
| | | | #100 | 825.85 | 1.9 |
| | | | #200 | 832.24 | 1.1 |

### Fractional Components

| Boulders | Cobbles | Gravel | | | | Sand | | | Fines |
|---|---|---|---|---|---|---|---|---|---|
| | | Coarse | Medium | Fine | Total | Coarse | Fine | Total | |
| 0.0 | 0.0 | 5.2 | 17.3 | 25.8 | 48.3 | 43.6 | 7.0 | 50.6 | 1.1 |

| $D_{10}$ | $D_{15}$ | $D_{20}$ | $D_{30}$ | $D_{50}$ | $D_{60}$ | $D_{80}$ | $D_{85}$ | $D_{90}$ | $D_{95}$ |
|---|---|---|---|---|---|---|---|---|---|
| 0.4825 | 0.6158 | 0.7389 | 1.0057 | 1.8788 | 2.8758 | 11.0059 | 14.3141 | 18.7165 | 25.8495 |

| Fineness Modulus | $C_u$ | $C_c$ |
|---|---|---|
| 4.52 | 5.96 | 0.73 |

# Appendix C

## Geotechnical Calculations

- Geology
- Soil Properties
- Bearing Resistance
- Earth Pressure Coefficients
- Site Class Evaluation

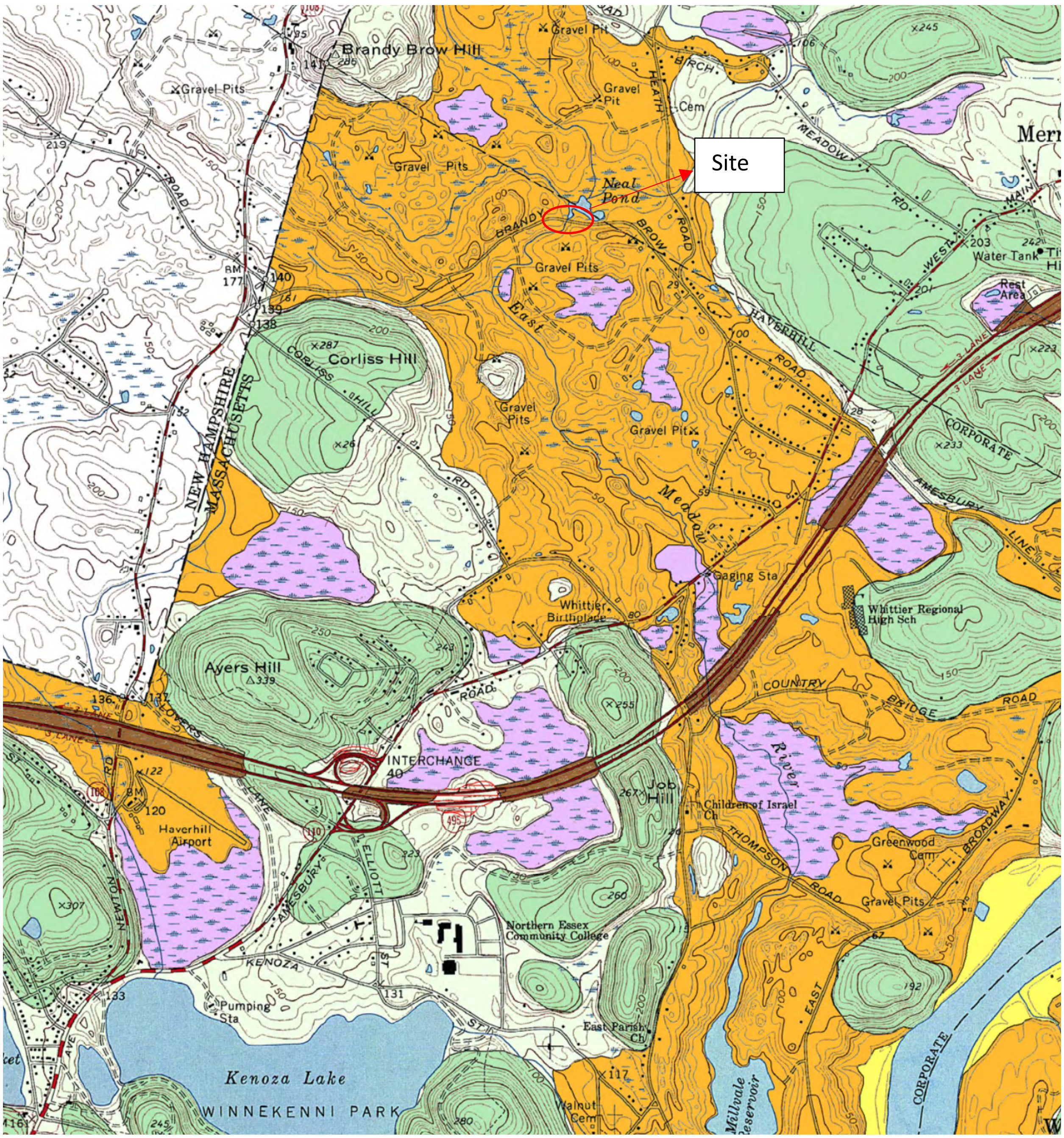
Geology Info

# Surficial Materials Map of the Haverhill Quadrangle, Massachusetts

Compiled by

Byron D. Stone, Janet R. Stone, and Mary L. DiGiacomo-Cohen

2018



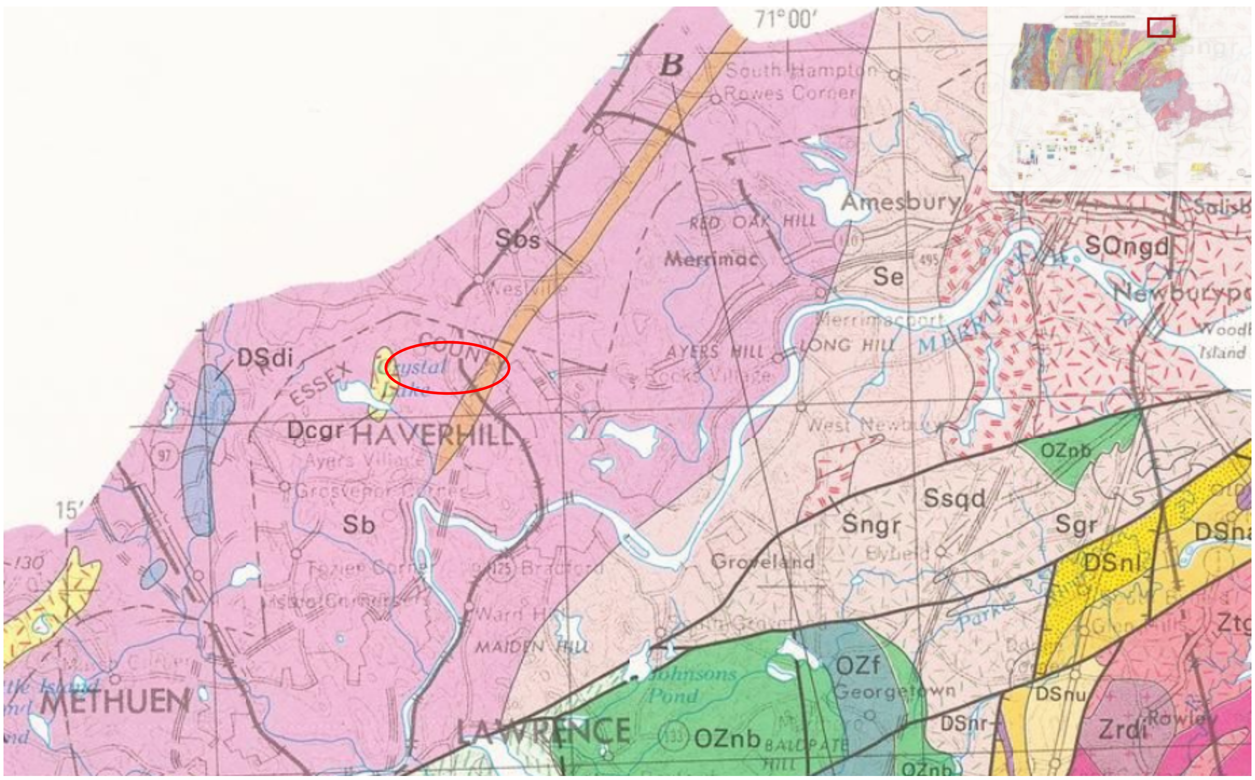
**Coarse deposits** consist of *gravel deposits*, *sand and gravel deposits*, and *sand deposits*, not differentiated in this report. *Gravel deposits* are composed of at least 50 percent gravel-size clasts; cobbles and boulders predominate; minor amounts of sand occur within gravel beds, and sand comprises a few separate layers. Gravel layers generally are poorly sorted, and bedding commonly is distorted and faulted due to postdepositional collapse related to melting of ice. *Sand and gravel deposits* occur as mixtures of gravel and sand within individual layers and as layers of sand alternating with layers of gravel. Sand and gravel layers generally range between 25 and 50 percent gravel particles and between 50 and 75 percent sand particles. Layers are well sorted to poorly sorted; bedding may be distorted and faulted due to postdepositional collapse. *Sand deposits* are composed mainly of very coarse to fine sand, commonly in well-sorted layers. Coarser layers may contain up to 25 percent gravel particles, generally granules and pebbles; finer layers may contain some very fine sand, silt, and clay

# BEDROCK GEOLOGIC MAP OF MASSACHUSETTS

E-an Zen, Editor

| Compiled By | Assisted By |
|---|---|
| Richard Goldsmith, Nicholas M. Ratcliffe, Peter Robinson, and Rolfe S. Stanley | Norman L. Hatch, Jr., Andrew F. Shride, Elaine G. A. Weed, and David R. Wones |

1983




| | |
|---|---|
| Sb | **Berwick Formation (Silurian)** Thin- to thick-bedded metamorphosed calcareous sandstone, siltstone, and minor muscovite schist |
| Sbs | Mica schist |
| Dcgr | **Chelmsford Granite (Lower Devonian)**—Light-gray, even and medium-grained, muscovite-biotite granite; locally foliated; intrudes Sb |

# *Recommended Soil Properties*

## Purpose:

The purpose of this evaluation is to select representative soil properties for the Culvert Replacement at Brandy Brow Road over East Meadow River Project. The soil properties will be used in our engineering analyses.

## Approach:

We selected values for unit weight and angle of internal friction of soils. Values were selected for the general soil layers observed in the borings and for proposed fills to be used during construction.

Unit Weight
We selected a saturated unit weight in pounds per cubic foot (pcf). The buoyant unit weight can then be determined by subtracting the unit weight of water (62.4 pcf).

Angle of Internal Friction
We selected an angle of internal friction ($\phi$) in degrees. We used Mohr-Coulomb's drained properties for each soil.

## References:

[1] AASHTO LRFD Bridge Design Specification, 8th Edition, 2017.

[2] Kulhawy, F.H. and Mayne, P.W, 1990. Manual on Estimating Soil Properties for Foundation Design, Cornell University, Ithaca, New York.

[3] Terzaghi, K., Peck, R.B., 1968. Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley & Sons, New York.

[4] Caltrans Geotechnical Manual, March 2014.

[5] Unified Facilities Criteria (UFC 3-220-10), Soil Mechanics (DM 7.1), 2022

[6] SPT Energy Calibration, GRL Engineers, 2018 provided by Northern Drill Services.

## Subsurface Investigation and SPT Correlations for Observed Soil Layers:

We reviewed Standard Penetration Test (SPT) N-Values performed during our subsurface investigation. We estimated angles of internal friction for soils above based on $N_{60}$ and $N_{1\,60}$. The SPTs were performed using an automatic hammer. We applied an efficiency of 80 percent for Automatic Hammer.

A summary of N-Values based on general soil type is shown below and our N-Value correction calculations are attached.

| Soil Layer | $N_{60ave}$ | $N_{1,60ave}$ |
|---|---|---|
| Sand and Gravel (above ~El. 56) | 24 | 32 |
| Sand and Gravel (below ~El. 56) | 16 | 16 |

Based on these N-values, we conservatively selected angles of internal friction for the soil.

For the existing Sand and Gravel, and correlations indicate that a friction angle of 30 to 39 degrees. Due to the potential for variability, we selected an angle of internal friction of 34 degrees.

## Proposed Fills:

We selected properties for ordinary borrow and gravel borrow based on the required material gradations and compaction requirements per MassDOT.

## Results:

We selected the following soil properties for the layer/soil type:

| Layer/Soil Type | Unit Weight, $\gamma$ (pcf) | Friction Angle, $\phi$ (deg) | Undrained Shear Strength (psf) |
|---|---|---|---|
| Sand and Gravel | 125/130[(1)] | 34 | - |

1. If above water table use lower unit weight to represent moist condition; if below water table use larger unit weight to represent saturated condition.

## Summary:

AASHTO Table 10.4.6.2.4-1 recommends using the following correlation to select friction angles of granular soils.

**Table 10.4.6.2.4-1—Correlation of *SPT* $N1_{60}$ Values to Drained Friction Angle of Granular Soils (modified after Bowles, 1977)**

| $N1_{60}$ | $\phi_f$ |
|---|---|
| <4 | 25–30 |
| 4 | 27–32 |
| 10 | 30–35 |
| 30 | 35–40 |
| 50 | 38–43 |

Additionally, we reviewed commonly used parameters for unit weight and friction angles published within available texts.

Soil Mechanics in Engineering Practice
In *Soil Mechanics in Engineering Practice,* Karl Terzaghi and Ralph Peck compiled various parameters of soils into the table below:

*Table 6.3*
*Porosity, Void Ratio, and Unit Weight of Typical Soils in Natural State*

| Description | Porosity, $n$ (%) | Void ratio, $e$ | Water content, $w$ (%) | Unit weight grams/cm³ $\gamma_d$ | Unit weight grams/cm³ $\gamma$ | Unit weight lb/ft³ $\gamma_d$ | Unit weight lb/ft³ $\gamma$ |
|---|---|---|---|---|---|---|---|
| 1. Uniform sand, loose | 46 | 0.85 | 32 | 1.43 | 1.89 | 90 | 118 |
| 2. Uniform sand, dense | 34 | 0.51 | 19 | 1.75 | 2.09 | 109 | 130 |
| 3. Mixed-grained sand, loose | 40 | 0.67 | 25 | 1.59 | 1.99 | 99 | 124 |
| 4. Mixed-grained sand, dense | 30 | 0.43 | 16 | 1.86 | 2.16 | 116 | 135 |
| 5. Glacial till, very mixed-grained | 20 | 0.25 | 9 | 2.12 | 2.32 | 132 | 145 |
| 6. Soft glacial clay | 55 | 1.2 | 45 | – | 1.77 | – | 110 |
| 7. Stiff glacial clay | 37 | 0.6 | 22 | – | 2.07 | – | 129 |
| 8. Soft slightly organic clay | 66 | 1.9 | 70 | – | 1.58 | – | 98 |
| 9. Soft very organic clay | 75 | 3.0 | 110 | – | 1.43 | – | 89 |
| 10. Soft bentonite | 84 | 5.2 | 194 | – | 1.27 | – | 80 |

$w$ = water content when saturated, in per cent of dry weight.
$\gamma_d$ = unit weight in dry state.
$\gamma$ = unit weight in saturated state.

Additionally, Terzaghi and Peck offer the following table, listing representative values for φ for various materials under effective pressures less than about 5 kg/cm².

*Table 17.1*
*Representative Values of $\phi$ for Sands and Silts*

| Material | Degrees Loose | Degrees Dense |
|---|---|---|
| Sand, round grains, uniform | 27.5 | 34 |
| Sand, angular grains, well graded | 33 | 45 |
| Sandy gravels | 35 | 50 |
| Silty sand | 27–33 | 30–34 |
| Inorganic silt | 27–30 | 30–35 |

<u>Manual on Estimating Soil Properties for Foundation Design</u>
Kulhawy and Mayne presented the following N value relationships based on Peck, Hanson, and Thornburn and Meyerhof.

| N Value (blows/ft or 305 mm) | Relative Density | Approximate $\bar{\phi}_{tc}$ (degrees) (a) | Approximate $\bar{\phi}_{tc}$ (degrees) (b) |
|---|---|---|---|
| 0 to 4 | very loose | < 28 | < 30 |
| 4 to 10 | loose | 28 to 30 | 30 to 35 |
| 10 to 30 | medium | 30 to 36 | 35 to 40 |
| 30 to 50 | dense | 36 to 41 | 40 to 45 |
| > 50 | very dense | > 41 | > 45 |

a - Source: Peck, Hanson, and Thornburn (12), p. 310.
b - Source: Meyerhof (13), p. 17.

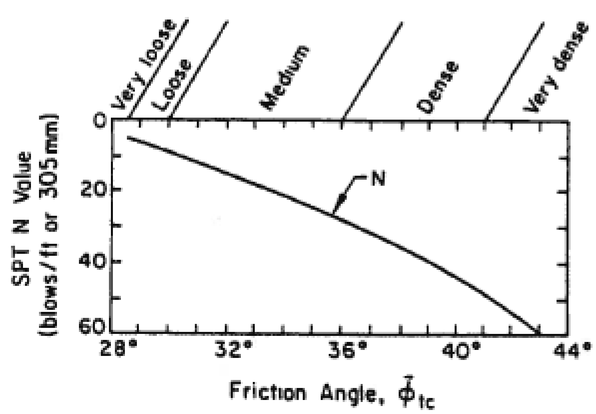
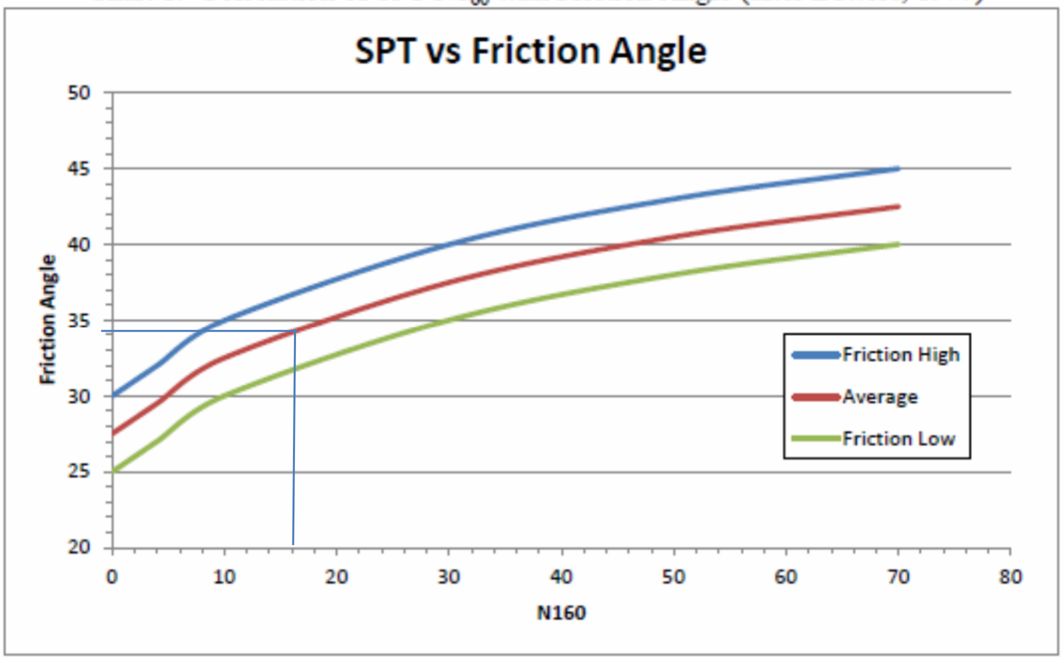



Figure 4-12. N versus $\bar{\phi}_{tc}$

Source: Peck, Hanson, and Thornburn (12), p. 310.

From the Caltrans Geotechnical Manual (March 2014):

Chart 1: Correlation of SPT $N1_{60}$ with Friction Angle (after Bowles, 1977)




Choose the friction angle (expressed to the nearest degree) based upon the soil type, particle size(s), and rounding or angularity. Experience should be used to select specific values within the ranges. In general, finer materials or materials with significant (about 30+ %) silt-sized material will fall in the lower portion of the range. Coarser materials with less than 5% fines will fall in the upper portion of the range. The extreme range of phi angles for any $N1_{60}$ is five degrees, so the adjustment factors for particle size and roundness should be only a degree or two. The following bullets provide help in determining which value to select for a given $N1_{60}$ and soil type:

- Use the maximum value for GW
- Use the average for GM and SP
- Use the minimum for SC
- Use the minimum + 0.5 for ML
- Use the average +1 for SW
- Use the average -1 for GC
- Use the Maximum -1 for GP

Values may also be increased with increasing grain size and/or particle angularity, and decreased with decreasing grain size and/or increasing roundness. For example, an SP with $N1_{60}$ = 30 could be assigned phi angles of 37, 38 or 39 degrees for fine, medium and coarse grain sizes respectively.

From the Unified Facilities Criteria (DM 7.1), 2022:

**Table 8-10 Approximate Undrained Shear Strength for Cohesive Soils Based on SPT *N***

| Soil Consistency | SPT *N* Value | Undrained Shear Strength (psf) | | |
|---|---|---|---|---|
| | | Parcher and Means (1968) | Tschebotarioff (1973) | Terzaghi et al. (1996) |
| Very soft | < 2 | 300 | - | < 250 |
| Soft | 2 – 4 | 300 – 600 | 250 – 500 | 250 – 500 |
| Medium | 4 – 8 | 600 – 1200 | 500 – 1000 | 500 – 1000 |
| Stiff | 8 – 15 | 1200 – 2400 | 1000 – 2000 | 1000 – 2000 |
| Very stiff | 15 – 30 | 2400 | 2000 – 4000 | 2000 – 4000 |
| Hard | > 30 | > 4500 | > 4000 | > 4000 |

| | | | |
|---|---|---|---|
| **Client:** | Greenman-Pedersen, Inc. | **Prepared By:** | S. Poudyal |
| **Project:** | Brandy Brow Road over East Meadow River | **Date:** | 2/7/2024 |
| **Project Number:** | 2400197 | **Checked By:** | D. Blanchard |
| **Subject:** | Corrected Blow Counts | **Date:** | 3/19/2024 |

**References:** 1) American Association of State Highway and Transportation Officials (AASHTO) "AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017"

**Equations:**

| Ref. 1 Eqn. No. | Equation | | |
|---|---|---|---|
| 10.4.6.2.4-2 | $N_{60}$ = (ER / 60%) * N | where: $N_{60}$ = | SPT blow count corrected for hammer efficiency (blows/ft) |
| | | ER = | hammer efficiency expressed as percent of theoretical free fall energy |
| | | N = | Uncorrected SPT blow count (blows/ft) |
| 10.4.6.2.4-3 | $N1_{60} = C_N * N_{60}$ | where: $N1_{60}$ = | SPT blow count corrected for overburden and hammer efficiency (blows/ft) |
| | | $C_N$ = | $0.77 * \log_{10}(40/\sigma'_v)$ [$C_N < 2.0$] |
| | | $\sigma'_v$ = | vertical effective stress (ksf) |

**Assumptions:**

| | | | |
|---|---|---|---|
| Ground Surface El.: | 70 | ft | *NAVD88 Datum* |
| Depth to Groundwater: | 8.6 | ft | |
| Average Total Unit Weight of Soil: | 120 | pcf | |

| Hammer Type | ER (%) | $C_E$ = ER / 60% |
|---|---|---|
| Donut | 45 | 0.75 |
| Safety | 60 | 1.00 |
| Automatic | 80 | 1.33 |

| ***Boring:*** | | ***B 1*** | | Corrected Blow Counts | | | | Overburden Correction | | | | | Hammer Efficiency Correction | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Depth (ft)** | **El. (ft)** | **Layer Name** | **N** | **$N_{60}$** | **$N1_{60}$** | **Avg. $N_{60}$** | **Avg. $N1_{60}$** | **$\sigma_v$ (psf)** | **u (psf)** | **$\sigma'_v$ (psf)** | **$\sigma'_v$ (ksf)** | **$C_N$** | **Hammer Type** | **ER (%)** | **$C_E$** |
| 1 | 69.0 | SAND AND GRAVEL | 99 | 100 | 100 | See Note 3 | | 120 | 0 | 120 | 0.120 | 1.94 | Automatic | 80 | 1.33 |
| 3 | 67.0 | SAND AND GRAVEL | 33 | 44 | 69 | 24 | 32 | 360 | 0 | 360 | 0.360 | 1.58 | Automatic | 80 | 1.33 |
| 5 | 65.0 | SAND AND GRAVEL | 13 | 17 | 24 | | | 600 | 0 | 600 | 0.600 | 1.40 | Automatic | 80 | 1.33 |
| 7 | 63.0 | SAND AND GRAVEL | 5 | 7 | 9 | | | 840 | 0 | 840 | 0.840 | 1.29 | Automatic | 80 | 1.33 |
| 9 | 61.0 | SAND AND GRAVEL | 18 | 24 | 29 | | | 1,080 | 25 | 1,055 | 1.055 | 1.22 | Automatic | 80 | 1.33 |
| 11 | 59.0 | SAND AND GRAVEL | 21 | 28 | 33 | | | 1,320 | 150 | 1,170 | 1.170 | 1.18 | Automatic | 80 | 1.33 |
| 13 | 57.0 | SAND AND GRAVEL | 16 | 21 | 25 | | | 1,560 | 275 | 1,285 | 1.285 | 1.15 | Automatic | 80 | 1.33 |
| 15 | 55.0 | SAND AND GRAVEL | 21 | 28 | 31 | 16 | 16 | 1,800 | 399 | 1,401 | 1.401 | 1.12 | Automatic | 80 | 1.33 |
| 20 | 50.0 | SAND AND GRAVEL | 4 | 5 | 6 | | | 2,400 | 711 | 1,689 | 1.689 | 1.06 | Automatic | 80 | 1.33 |
| 25 | 45.0 | SAND AND GRAVEL | 9 | 12 | 12 | | | 3,000 | 1,023 | 1,977 | 1.977 | 1.01 | Automatic | 80 | 1.33 |
| 30 | 40.0 | SAND AND GRAVEL | 13 | 17 | 17 | | | 3,600 | 1,335 | 2,265 | 2.265 | 0.96 | Automatic | 80 | 1.33 |
| 35 | 35.0 | SAND AND GRAVEL | 21 | 28 | 26 | | | 4,200 | 1,647 | 2,553 | 2.553 | 0.92 | Automatic | 80 | 1.33 |
| 40 | 30.0 | SAND AND GRAVEL | 15 | 20 | 18 | | | 4,800 | 1,959 | 2,841 | 2.841 | 0.88 | Automatic | 80 | 1.33 |
| 45 | 25.0 | SAND AND GRAVEL | 8 | 11 | 9 | | | 5,400 | 2,271 | 3,129 | 3.129 | 0.85 | Automatic | 80 | 1.33 |
| 50 | 20.0 | SAND AND GRAVEL | 5 | 7 | 5 | | | 6,000 | 2,583 | 3,417 | 3.417 | 0.82 | Automatic | 80 | 1.33 |

Notes:
1. For $N_{60}$ and $N1_{60}$ values greater than 100 blows/ft, we input the value 100 blows/ft.
2. N-Values from SPT's that encountered refusal prior to a penetration of 12 inches were not included in the averages.
3. The high N-value at surface was excluded from the average as it was likely due to base material for road construction and therefore not representative.

**FACTORED BEARING RESISTANCE CALCULATIONS**

The following calculation provides bearing resistance calculations for the culvert replacement at Brandy Brow Road over East Meadow Road in Haverhill, MA. Note that we have assumed that the culvert walls are traditional wingwalls with level ground at the time of preparing these calculations. The resistances were calculated based on the method, equations, and appropriate parameters provided in the 2017 AASHTO LRFD Bridge Design Specifications.

References utilized for these calculations (including those pertaining to resistance factors) are provided at the back of this calculation. Cross sections are attached for reference.

Bearing resistances were calculated with the following formula:

$$q_n = cN_{cm} + \gamma D_f N_{qm} C_{wq} + 0.5\gamma \ BN_{\gamma m} C_{w\gamma} \quad (10.6.3.1.2a\text{-}1)$$

in which:

$$N_{cm} = N_c s_c i_c \quad (10.6.3.1.2a\text{-}2)$$

$$N_{qm} = N_q s_q d_q i_q \quad (10.6.3.1.2a\text{-}3)$$

$$N_\gamma m = N_\gamma s_\gamma i_\gamma \quad (10.6.3.1.2a\text{-}4)$$

where:

$c$ = cohesion, taken as undrained shear strength (ksf)

$N_c$ = cohesion term (undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)

$N_q$ = surcharge (embedment) term (drained or undrained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)

$N_\gamma$ = unit weight (footing width) term (drained loading) bearing capacity factor as specified in Table 10.6.3.1.2a-1 (dim)

$\gamma_q$ = total (moist) unit weight of soil above the bearing depth of the footing (kcf)

$\gamma_f$ = total (moist) unit weight of soil below the bearing depth of the footing (kcf)

$D_f$ = footing embedment depth (ft)

$B$ = footing width (ft)

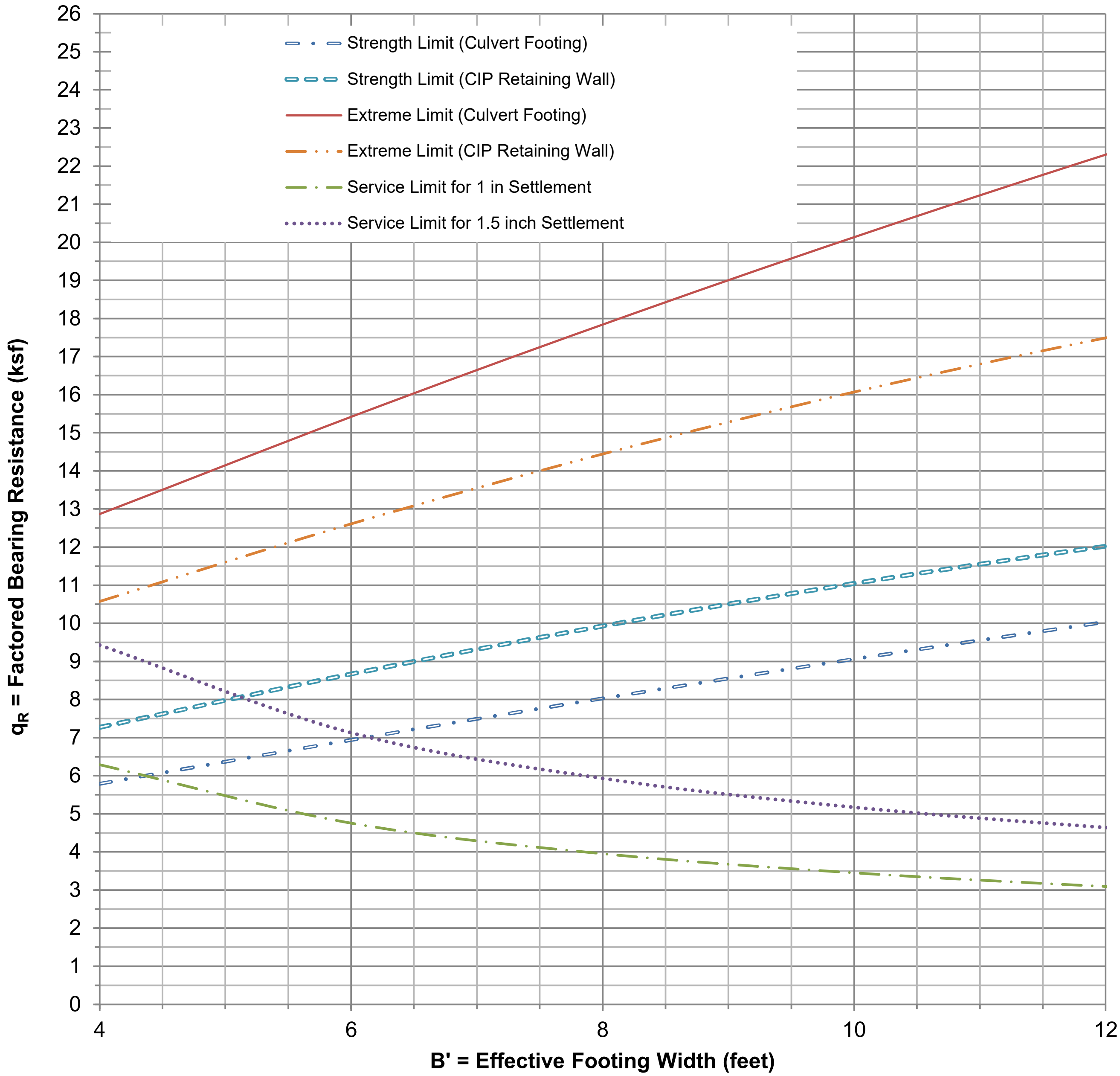
$C_{wq}, C_{w\gamma}$ = correction factors to account for the location of the groundwater table as specified in Table 10.6.3.1.2a-2 (dim)

$s_c, s_\gamma, s_q$ = footing shape correction factors as specified in Table 10.6.3.1.2a-3 (dim)

$d_q$ = depth correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation determined from Eq. 10.6.3.1.2a-10 (dim)

$i_c, i_\gamma, i_q$ = load inclination factors determined from Eqs. 10.6.3.1.2a-5 or 10.6.3.1.2a-6, and 10.6.3.1.2a-7 and 10.6.3.1.2a-8 (dim)

Additional formulas for correction factors are provided at the back of this calculation packet.
We assumed all load inclination factors to be 1.0, rather than use the provided formulas.




Notes:
1. The above resistances are based on methods provided in the 2020 AASHTO LRFD Bridge Specifications.
2. The resistances above are based on 4 feet of embedment and level ground (i.e., no sloping ground) below the existing brook bottom.
3. Groundwater was assumed to be at ground surface, based on nearby borings.
4. The Strength Limit values above are based on a Resistance Factor of 0.45 for abutments on shallow foundation and 0.55 for gravity and semi-gravity retaining walls. The Extreme Limit values above are based on Resistance Factors of 1.0 for abutments on shallow foundation and 0.8 for gravity and semi-gravity retaining walls. The Service Limit values are based on a Resistance Factor of 1.0.
5. The Service Limit resistance curve can be referenced to evaluate settlement associated with a net increase in loading.

| | | |
|---|---|---|
| Brandy Brow Road over East Meadow River<br>Haverhill, MA | GEI Consultants | FACTORED BEARING RESISTANCE<br>VS. EFFECTIVE FOOTING WIDTH |
| Greenman-Pedersen, Inc.<br>Wilmington, Massachusetts | Project No. 2400197 | March 2024 Fig. 3 |

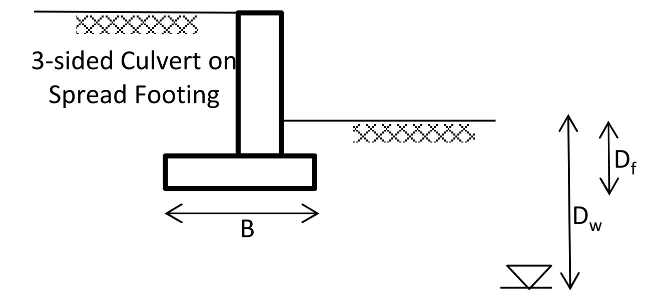
GEI Consultants

Project: Brandy Brow Road over East Meadow Road
Haverhill, MA
GEI Project No.: 2400197

By: D. Blanchard
Date: 03/19/2024
Checked By: L.Beintum
Date: 3/28/2024

**FACTORED BEARING RESISTANCE CALCULATIONS - CULVERT FOOTING**

Note: All references are to AASHTO LRFD Bridge Design Specifications, unless otherwise noted. See attached sheets with applicable table and equation references.




**RESISTANCE FACTORS**

| | | |
|---|---|---|
| Strength Limit | 0.45 | AASHTO, Table 10.5.5.2.2-1 |
| Extreme I Limit | 1.0 | AASHTO, 10.5.5.3 |
| Service Limit | 1.0 | AASHTO, 10.5.5.1 |

**BEARING SOIL PROPERTIES/SUBSURFACE INFORMATION**

| | | | |
|---|---|---|---|
| Bearing Soil Type | | Sand and Gravel | |
| Unit Weight of Bearing Soil (γ) | pcf | 125 | |
| | | 125 | |
| Cohesion of Bearing Soil ( c ) | psf | 0 | |
| Friction Angle of bearing Soil (ϕ') | ° | 34 | |
| Es, Modulus of Elasticity | ksi | 4.0 | (AASHTO 2017, Table C10.4.6.3-1) |
| ν, poissons ratio | | 0.3 | |
| Depth to Groundwater, Dw | ft | 0.0 | GW assumed at the surface |
| Bearing Capacity Factor ($N_c$) | | 42.2 | |
| Bearing Capacity Factor ($N_q$) | | 29.4 | |
| Bearing Capacity Factor ($N_\gamma$) | | 41.1 | |

**FOOTING GEOMETRY**

| | | | |
|---|---|---|---|
| Bottom of Footing Elevation (NAVD88) | ft | 10.0 | |
| Minimum Footing Depth ($D_f$) | ft | 4.0 | |
| Footing Length (L) | ft | 31.3 | Existing culvert length |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Effective Width, B' (B' = B - 2e) | ft | 4.0 | 6.0 | 8.0 | 10.0 | 12.0 | 14.0 |
| Effective Length, L' =L | ft | 31.3 | 31.3 | 31.3 | 31.3 | 31.3 | 31.3 |
| L'/B' | | 7.8 | 5.2 | 3.9 | 3.1 | 2.6 | 2.2 |
| Df/B' | | 1.0 | 0.7 | 0.5 | 0.4 | 0.3 | 0.3 |
| A' | sf | 125 | 188 | 250 | 313 | 375 | 438 |
| βz | | 1.33 | 1.23 | 1.19 | 1.16 | 1.14 | 1.12 |

**BEARING RESISTANCE EQUATION FACTORS/COEFFICIENTS**

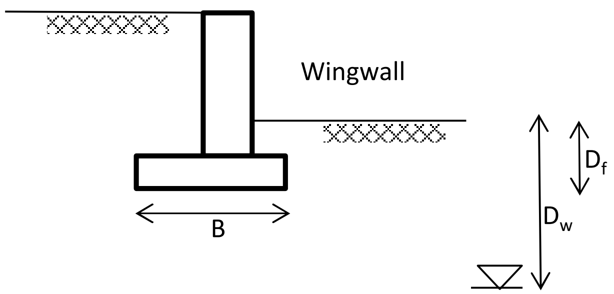
| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Effective Width, B' (B' = B - 2e) | ft | 4.0 | 6.0 | 8.0 | 10.0 | 12.0 | 14.0 |
| **$N_{cm}$** | | 45.9 | 47.8 | 49.7 | 51.6 | 53.5 | 55.4 |
| Shape Correction Factor ($s_c$) | | 1.09 | 1.13 | 1.18 | 1.22 | 1.27 | 1.31 |
| Load Inclination Factor ($i_c$) | | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| **$N_{qm}$** | | 32.0 | 33.3 | 34.5 | 35.8 | 37.1 | 38.3 |
| Shape Correction Factor ($s_q$) | | 1.09 | 1.13 | 1.17 | 1.22 | 1.26 | 1.30 |
| Load Inclination Factor ($i_q$) | | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| Depth Correction Factor ($d_q$) | | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| **$N_{\gamma m}$** | | 39.0 | 37.9 | 36.9 | 35.8 | 34.8 | 33.7 |
| Shape Correction Factor ($s_\gamma$) | | 0.95 | 0.92 | 0.90 | 0.87 | 0.85 | 0.82 |
| Load Inclination Factor ($i_\gamma$) | | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| Groundwater Coefficient, $C_{wq}$ | | 0.50 | 0.50 | 0.50 | 0.50 | 0.50 | 0.50 |
| Groundwater Coefficient, $C_{w\gamma}$ | | 0.50 | 0.50 | 0.50 | 0.50 | 0.50 | 0.50 |

**CALCULATED BEARING RESISTANCES**

| | | | | | | |
|---|---|---|---|---|---|---|
| Nominal Bearing Resistance ($q_n$, ksf) | 12.9 | 15.4 | 17.8 | 20.1 | 22.3 | 24.3 |
| **Strength Limit Factored Bearing Resistance: $q_R$ (ksf)** | **5.8** | **6.9** | **8.0** | **9.1** | **10.0** | **10.9** |
| **Extreme Limit Factored Bearing Resistance: $q_R$ (ksf)** | **12.9** | **15.4** | **17.8** | **20.1** | **22.3** | **24.3** |
| **Service Limit Bearing, qo, for 1 inch (Factored)** | **6.3** | **4.8** | **4.0** | **3.4** | **3.1** | **2.8** |
| **Service Limit Bearing, qo, for 1.5 inch (Factored)** | **9.4** | **7.1** | **5.9** | **5.2** | **4.6** | **4.2** |

**FACTORED BEARING RESISTANCE CALCULATIONS CAST-IN-PLACE RETAINING WALL FOOTING**

Note: All references are to AASHTO LRFD Bridge Design Specifications, unless otherwise noted. See attached sheets with applicable table and equation references.




**RESISTANCE FACTORS**

| | | | |
|---|---|---|---|
| Strength Limit | | 0.55 | AASHTO Table 11.5.7-1 |
| Extreme I Limit | | 0.8 | AASHTO 11.5.8 |
| Service Limit | | 1.0 | AASHTO 11.5.7 |

**BEARING SOIL PROPERTIES/SUBSURFACE INFORMATION**

| | | | |
|---|---|---|---|
| Bearing Soil Type | | Sand and Gravel | |
| Unit Weight of Bearing Soil (γ) | pcf | 125 | |
| | | 125 | |
| Cohesion of Bearing Soil ( c ) | psf | 0 | |
| Friction Angle of bearing Soil (ϕ') | ° | 34 | |
| Es, Modulus of Elasticity | ksi | 4.0 | (AASHTO 2017, Table C10.4.6.3-1) |
| ν, poissons ratio | | 0.3 | |
| Depth to Groundwater, Dw | ft | 0.0 | GW assumed at the surface |
| Bearing Capacity Factor ($N_c$) | | 42.2 | |
| Bearing Capacity Factor ($N_q$) | | 29.4 | |
| Bearing Capacity Factor ($N_\gamma$) | | 41.1 | |

**FOOTING GEOMETRY**

| | | |
|---|---|---|
| Bottom of Footing Elevation (NAVD88) | ft | 10.0 |
| Minimum Footing Depth ($D_f$) | ft | 4.0 |
| Footing Length (L) | ft | 16.0 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Effective Width, B' (B' = B - 2e) | ft | 4.0 | 6.0 | 8.0 | 10.0 | 12.0 | 14.0 |
| Effective Length, L' =L | ft | 16.0 | 16.0 | 16.0 | 16.0 | 16.0 | 16.0 |
| L'/B' | | 4.0 | 2.7 | 2.0 | 1.6 | 1.3 | 1.1 |
| Df/B' | | 1.0 | 0.7 | 0.5 | 0.4 | 0.3 | 0.3 |
| A' | sf | 64 | 96 | 128 | 160 | 192 | 224 |
| βz | | 1.19 | 1.14 | 1.11 | 1.10 | 1.09 | 1.08 |

**BEARING RESISTANCE EQUATION FACTORS/COEFFICIENTS**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Effective Width, B' (B' = B - 2e) | ft | 4.0 | 6.0 | 8.0 | 10.0 | 12.0 | 14.0 |
| **$N_{cm}$** | | 49.5 | 53.2 | 56.9 | 60.6 | 64.2 | 67.9 |
| Shape Correction Factor ($s_c$) | | 1.17 | 1.26 | 1.35 | 1.44 | 1.52 | 1.61 |
| Load Inclination Factor ($i_c$) | | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| **$N_{qm}$** | | 34.4 | 36.9 | 39.4 | 41.9 | 44.3 | 46.8 |
| Shape Correction Factor ($s_q$) | | 1.17 | 1.25 | 1.34 | 1.42 | 1.51 | 1.59 |
| Load Inclination Factor ($i_q$) | | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| Depth Correction Factor ($d_q$) | | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| **$N_{\gamma m}$** | | 37.0 | 34.9 | 32.9 | 30.8 | 28.7 | 26.7 |
| Shape Correction Factor ($s_\gamma$) | | 0.90 | 0.85 | 0.80 | 0.75 | 0.70 | 0.65 |
| Load Inclination Factor ($i_\gamma$) | | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| Groundwater Coefficient, $C_{wq}$ | | 0.50 | 0.50 | 0.50 | 0.50 | 0.50 | 0.50 |
| Groundwater Coefficient, $C_{w\gamma}$ | | 0.50 | 0.50 | 0.50 | 0.50 | 0.50 | 0.50 |

**CALCULATED BEARING RESISTANCES**

| | | | | | | |
|---|---|---|---|---|---|---|
| Nominal Bearing Resistance ($q_n$, ksf) | 13.2 | 15.8 | 18.1 | 20.1 | 21.9 | 23.4 |
| **Strength Limit Factored Bearing Resistance: $q_R$ (ksf)** | **7.3** | **8.7** | **9.9** | **11.0** | **12.0** | **12.9** |
| **Extreme Limit Factored Bearing Resistance: $q_R$ (ksf)** | **10.6** | **12.6** | **14.4** | **16.1** | **17.5** | **18.7** |
| **Service Limit Bearing, qo, for 1 inch (Factored)** | **7.8** | **6.1** | **5.2** | **4.6** | **4.1** | **3.8** |
| **Service Limit Bearing, qo, for 1.5 inch (Factored)** | **11.8** | **9.2** | **7.8** | **6.9** | **6.2** | **5.7** |

**Table 10.4.6.2.4-1—Correlation of *SPT* $N1_{60}$ Values to Drained Friction Angle of Granular Soils (modified after Bowles, 1977)**

| $N1_{60}$ | $\phi_f$ |
|---|---|
| <4 | 25–30 |
| 4 | 27–32 |
| 10 | 30–35 |
| 30 | 35–40 |
| 50 | 38–43 |

**Table 10.6.3.1.2a-1—Bearing Capacity Factors $N_c$ (Prandtl, 1921), $N_q$ (Reissner, 1924), and $N_\gamma$ (Vesic, 1975)**

| $\phi_f$ | $N_c$ | $N_q$ | $N_\gamma$ | $\phi_f$ | $N_c$ | $N_q$ | $N_\gamma$ |
|---|---|---|---|---|---|---|---|
| 0 | 5.14 | 1.0 | 0.0 | 23 | 18.1 | 8.7 | 8.2 |
| 1 | 5.4 | 1.1 | 0.1 | 24 | 19.3 | 9.6 | 9.4 |
| 2 | 5.6 | 1.2 | 0.2 | 25 | 20.7 | 10.7 | 10.9 |
| 3 | 5.9 | 1.3 | 0.2 | 26 | 22.3 | 11.9 | 12.5 |
| 4 | 6.2 | 1.4 | 0.3 | 27 | 23.9 | 13.2 | 14.5 |
| 5 | 6.5 | 1.6 | 0.5 | 28 | 25.8 | 14.7 | 16.7 |
| 6 | 6.8 | 1.7 | 0.6 | 29 | 27.9 | 16.4 | 19.3 |
| 7 | 7.2 | 1.9 | 0.7 | 30 | 30.1 | 18.4 | 22.4 |
| 8 | 7.5 | 2.1 | 0.9 | 31 | 32.7 | 20.6 | 26.0 |
| 9 | 7.9 | 2.3 | 1.0 | 32 | 35.5 | 23.2 | 30.2 |
| 10 | 8.4 | 2.5 | 1.2 | 33 | 38.6 | 26.1 | 35.2 |
| 11 | 8.8 | 2.7 | 1.4 | 34 | 42.2 | 29.4 | 41.1 |
| 12 | 9.3 | 3.0 | 1.7 | 35 | 46.1 | 33.3 | 48.0 |
| 13 | 9.8 | 3.3 | 2.0 | 36 | 50.6 | 37.8 | 56.3 |
| 14 | 10.4 | 3.6 | 2.3 | 37 | 55.6 | 42.9 | 66.2 |
| 15 | 11.0 | 3.9 | 2.7 | 38 | 61.4 | 48.9 | 78.0 |
| 16 | 11.6 | 4.3 | 3.1 | 39 | 67.9 | 56.0 | 92.3 |
| 17 | 12.3 | 4.8 | 3.5 | 40 | 75.3 | 64.2 | 109.4 |
| 18 | 13.1 | 5.3 | 4.1 | 41 | 83.9 | 73.9 | 130.2 |
| 19 | 13.9 | 5.8 | 4.7 | 42 | 93.7 | 85.4 | 155.6 |
| 20 | 14.8 | 6.4 | 5.4 | 43 | 105.1 | 99.0 | 186.5 |
| 21 | 15.8 | 7.1 | 6.2 | 44 | 118.4 | 115.3 | 224.6 |
| 22 | 16.9 | 7.8 | 7.1 | 45 | 133.9 | 134.9 | 271.8 |

**Table C10.4.6.3-1—Elastic Constants of Various Soils (modified after U.S. Department of the Navy, 1982; Bowles, 1988)**

| Soil Type | Typical Range of Young's Modulus Values, $E_s$ (ksi) | Poisson's Ratio, $\nu$ (dim) |
|---|---|---|
| Clay: | | |
| Soft sensitive | 0.347–2.08 | 0.4–0.5 (undrained) |
| Medium stiff to stiff | 2.08–6.94 | |
| Very stiff | 6.94–13.89 | |
| Loess | 2.08–8.33 | 0.1–0.3 |
| Silt | 0.278–2.78 | 0.3–0.35 |
| Fine Sand: | | |
| Loose | 1.11–1.67 | 0.25 |
| Medium dense | 1.67–2.78 | |
| Dense | 2.78–4.17 | |
| Sand: | | |
| Loose | 1.39–4.17 | 0.20–0.36 |
| Medium dense | 4.17–6.94 | |
| Dense | 6.94–11.11 | 0.30–0.40 |
| Gravel: | | |
| Loose | 4.17–11.11 | 0.20–0.35 |
| Medium dense | 11.11–13.89 | |
| Dense | 13.89–27.78 | 0.30–0.40 |
| Estimating $E_s$ from *SPT N* Value | | |
| Soil Type | $E_s$ (ksi) | |
| Silts, sandy silts, slightly cohesive mixtures | $0.056\ N1_{60}$ | |
| Clean fine to medium sands and slightly silty sands | $0.097\ N1_{60}$ | |
| Coarse sands and sands with little gravel | $0.139\ N1_{60}$ | |
| Sandy gravel and gravels | $0.167\ N1_{60}$ | |
| Estimating $E_s$ from $q_c$ (static cone resistance) | | |
| Sandy soils | $0.028q_c$ | |

**Table 10.6.3.1.2a-2—Coefficients $C_{wq}$ and $C_{w\gamma}$ for Various Groundwater Depths**

| $D_w$ | $C_{wq}$ | $C_{w\gamma}$ |
|---|---|---|
| 0.0 | 0.5 | 0.5 |
| $D_f$ | 1.0 | 0.5 |
| $>1.5B+D_f$ | 1.0 | 1.0 |

Where the position of groundwater is at a depth less than 1.5 times the footing width below the footing base, the bearing resistance is affected. The highest anticipated groundwater level should be used in design.

**Table 10.6.3.1.2a-3—Shape Correction Factors $s_c$, $s_\gamma$, $s_q$**

| Factor | Friction Angle | Cohesion Term ($s_c$) | Unit Weight Term ($s_\gamma$) | Surcharge Term ($s_q$) |
|---|---|---|---|---|
| Shape Factors $s_c$, $s_\gamma$, $s_q$ | $\phi_f = 0$ | $1+\left(\frac{B}{5L}\right)$ | 1.0 | 1.0 |
| | $\phi_f > 0$ | $1+\left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$ | $1-0.4\left(\frac{B}{L}\right)$ | $1+\left(\frac{B}{L}\tan\phi_f\right)$ |

$$d_q = 1 + 2\tan\phi_f(1-\sin\phi_f)^2 \arctan\left(\frac{D_f}{B}\right) \quad (10.6.3.1.2a\text{-}10)$$

where:

$d_q$ = depth correction factor to account for the shearing resistance along the failure surface passing through cohesionless material above the bearing elevation(dim)
$\phi_f$ = angle of internal friction of soil (degrees)
$D_f$ = footing embedment depth (ft)
$B$ = footing width (ft)

Arctan ($D_f/B$) is in radians.

The depth correction factor should be used only when the soils above the footing bearing elevation are as competent as the soils beneath the footing level; otherwise, the depth correction factor should be taken as 1.0. The depth correction factor, $d_q$, shall not exceed 1.4.

The parent information from which Table 10.6.3.1.2a-4 was developed covered the indicated range of friction angle, $\phi_f$. Information beyond the range indicated is not available at this time.

Elastic Half-Space Method (AASHTO 8th Ed.)

$$S_e = \frac{\left[q_o\left(1-\nu^2\right)\sqrt{A'}\right]}{144\, E_s \beta_z} \quad (10.6.2.4.2\text{-}1)$$

where:

$q_o$ = applied vertical stress (ksf)
$A'$ = effective area of footing (ft$^2$)
$E_s$ = Young's modulus of soil taken as specified in Article 10.4.6.3 if direct measurements of $E_s$ are not available from the results of in situ or laboratory tests (ksi)

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

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

**Table 10.5.5.2.2-1—Resistance Factors for Geotechnical Resistance of Shallow Foundations at the Strength Limit State**

| Method/Soil/Condition | | | Resistance Factor |
|---|---|---|---|
| Bearing Resistance | $\varphi_b$ | Theoretical method (Munfakh et al., 2001), in clay | 0.50 |
| | | Theoretical method (Munfakh et al., 2001), in sand, using *CPT* | 0.50 |
| | | Theoretical method (Munfakh et al., 2001), in sand, using *SPT* | 0.45 |
| | | Semi-empirical methods (Meyerhof, 1957), all soils | 0.45 |
| | | Footings on rock | 0.45 |
| | | Plate Load Test | 0.55 |
| Sliding | $\varphi_\tau$ | Precast concrete placed on sand | 0.90 |
| | | Cast-in-Place Concrete on sand | 0.80 |
| | | Cast-in-Place or precast Concrete on Clay | 0.85 |
| | | Soil on soil | 0.90 |
| | $\varphi_{ep}$ | Passive earth pressure component of sliding resistance | 0.50 |

**10.5.5—Resistance Factors**

**10.5.5.1—Service Limit States**

Resistance factors for the service limit states shall be taken as 1.0, except as provided for overall stability in Article 11.6.2.3.

A resistance factor of 1.0 shall be used to assess the ability of the foundation to meet the specified deflection criteria after scour due to the design flood.

**10.5.5.3—Extreme Limit States**

*10.5.5.3.1—General*

Design of foundations at extreme limit states shall be consistent with the expectation that structure collapse is prevented and that life safety is protected.

*10.5.5.3.2—Scour*

The provisions of Articles 2.6.4.4.2 and 3.7.5 shall apply to the changed foundation conditions resulting from scour. Resistance factors at the strength limit state shall be taken as specified herein. Resistance factors at the extreme event shall be taken as 1.0 except that for uplift resistance of piles and shafts, the resistance factor shall be taken as 0.80 or less.

The foundation shall resist not only the loads applied from the structure but also any debris loads occurring during the flood event.

*C10.5.5.3.2*

The specified resistance factors should be used provided that the method used to compute the nominal resistance does not exhibit bias that is unconservative. See Paikowsky et al. (2004) regarding bias values for pile resistance prediction methods.

Design for scour is discussed in Hannigan et al. (2005).

*10.5.5.3.3—Other Extreme Limit States*

Resistance factors for extreme limit state, including the design of foundations to resist earthquake, ice, vehicle or vessel impact loads, shall be taken as 1.0. For uplift resistance of piles and shafts, the resistance factor shall be taken as 0.80 or less.

*C10.5.5.3.3*

The difference between compression skin friction and tension skin friction should be taken into account through the resistance factor, to be consistent with how this is done for the strength limit state (see Article 10.5.5.2.3).

**Table 11.5.7-1—Strength Limit State Resistance Factors for Permanent Retaining Walls**

| Wall-Type and Condition | | Resistance Factor |
|---|---|---|
| **Nongravity Cantilevered and Anchored Walls** | | |
| Axial compressive resistance of vertical elements | | Article 10.5 applies |
| Passive resistance of vertical elements | | 0.75 |
| Pullout resistance of anchors [1] | • Cohesionless (granular) soils<br>• Cohesive soils<br>• Rock | 0.65 [1]<br>0.70 [1]<br>0.50 [1] |
| Pullout resistance of anchors [2] | • Where proof tests are conducted | 1.0 [2] |
| Tensile resistance of anchor tendon | • Mild steel (e.g., ASTM A615 bars)<br>• High-strength steel (e.g., ASTM A722 bars) | 0.90 [3]<br>0.80 [3] |
| Overall stability, soil failure | | Article 11.6.3.7 applies |
| Flexural capacity of vertical elements | | 0.90 |
| **Mechanically Stabilized Earth Walls, Gravity Walls, and Semigravity Walls** | | |
| Bearing resistance | • Gravity and semigravity walls<br>• MSE walls | 0.55<br>0.65 |
| Sliding | | 1.0 |
| Tensile resistance of metallic reinforcement and connectors | Strip reinforcements [4]<br>Grid reinforcements [4][5] | 0.75<br>0.65 |
| Tensile resistance of geosynthetic reinforcement and connectors | • Geotextile and geogrid reinforcements<br>• Geostrip reinforcements | 0.80<br>0.55 |
| Pullout resistance of metallic reinforcement | • Steel strip reinforcements<br>• Steel grid reinforcements | 0.90<br>0.90 |
| Pullout resistance of geosynthetic reinforcement | • Geotextiles and geogrids<br>• Geostrip reinforcements | 0.70<br>0.70 |
| Service Limit, for soil failure using stiffness method | | 1.0 |
| Overall and compound stability, soil failure | | Article 11.6.3.7 applies |
| **Prefabricated Modular Walls** | | |
| Bearing | | Article 10.5 applies |
| Sliding | | Article 10.5 applies |
| Passive resistance | | Article 10.5 applies |
| Overall stability, soil failure | | Article 11.6.3.7 applies |
| **Soil Nail Walls [6]** | | |
| Lateral sliding | | 1.00 |
| Overall and Compound stability, soil failure | | Article 11.6.3.7 applies |
| Tensile resistance of nail tendon | Mild steel bars (Grade 75)<br>High resistance bars (Grades 95 and 150) | 0.75<br>0.65 |
| Pullout resistance of nail | | 0.65 |
| Facing flexure | Initial and final facing | 0.90 |
| Facing punching shear | Initial and final facing | 0.90 |
| Tensile resistance of headed stud | A307 steel bolt [7]<br>A325 steel bolt | 0.70<br>0.80 |

[2] Apply where proof test(s) are conducted on every production anchor to a load of 1.0 or greater times the factored load on the anchor.

[3] Apply to maximum proof test load for the anchor. For mild steel, apply resistance factor to $F_y$. For high-strength steel, apply the resistance factor to guaranteed ultimate tensile strength.

[4] Apply to gross cross-section less sacrificial area. For sections with holes, reduce gross area in accordance with Article 6.8.3 and apply to net section less sacrificial area.

[5] Applies to grid reinforcements connected to a rigid facing element, e.g., a concrete panel or block. For grid reinforcements connected to a flexible facing mat or which are continuous with the facing mat, use the resistance factor for strip reinforcements.

[6] Additional, special cases of limit states, as well as corresponding resistance factors, for soil nail walls are presented in FHWA-NHI-14-007/FHWA GEC 7 (Lazarte et al. 2015).

[7] Equivalent to AWS D1.1 Type B studs, with $f_y$ = 60 ksi.

### 11.5.7—Resistance Factors—Service and Strength

Resistance factors for the service limit states shall be taken as 1.0.

For the strength limit state, the resistance factors provided in Table 11.5.7-1 shall be used for wall design, unless region-specific values or substantial successful experience is available to justify higher values. Resistance factors for geotechnical design of foundations that may be needed for wall support, unless specifically identified in Table 11.5.7-1, are as specified in Tables 10.5.5.2.2-1, 10.5.5.2.3-1, and 10.5.5.2.4-1.

If methods other than those prescribed in these Specifications are used to estimate resistance, the resistance factors chosen shall provide the same reliability as those given in Tables 10.5.5.2.2-1, 10.5.5.2.3-1, 10.5.5.2.4-1, and Table 11.5.7-1.

### 11.5.8—Resistance Factors—Extreme Event Limit State

Unless otherwise specified, all resistance factors shall be taken as 1.0 when investigating the extreme event limit state.

For overall stability of the retaining wall when earthquake loading is included, a resistance factor, $\phi$, of 0.9 shall be used. For bearing resistance, a resistance factor of 0.8 shall be used for gravity and semigravity walls and 0.9 for MSE walls.

For tensile resistance of metallic reinforcement and connectors, when earthquake loading is included, the following resistance factors shall be used:

- Strip reinforcements, $\phi = 1.0$
- Grid reinforcement, $\phi = 0.85$

Table 11.5.7-1 Notes 4 and 5 also apply to these resistance factors for metallic reinforcements.

For tensile and pullout resistance of geosynthetic reinforcement and connectors, a resistance factor, $\phi$, of 1.0 shall be used.

For pullout resistance of metallic reinforcement, a resistance factor, $\phi$, of 1.20 shall be used.

**Purpose:** Calculate lateral earth pressure coefficients

**Reference:** American Association of State Highway and Transportation Officials (AASHTO) "AASHTO LRFD Bridge Design Specifications, 8th Edition, 2017"

**Equations:** See attached

**Calculations: Lateral Earth Pressure Coefficients $k_0$, $k_a$, and $k_p$**

| | **Sand and Gravel** | **Gravel Borrow for Bridge Foundation** | **Gravel Borrow** | **Ordinary Borrow** |
|---|---|---|---|---|
| Effective Friction Angle of Soil, $\phi'_f$ (deg) | 34 | 37 | 35 | 32 |
| Friction Angle Between Fill and Wall, $\delta$ (deg) | 17.0 | 18.5 | 17.5 | 16.0 |
| Angle of Fill to the Horizontal, $\beta$ (deg) | 0 | 0 | 0 | 0 |
| Angle of Back Face of Wall to the Horizontal, $\theta$ (deg) | 90 | 90 | 90 | 90 |
| **At-Rest Lateral Earth Pressure Coefficient, $k_o$ (Eq. 3.11.5.2-1)** | **0.44** | **0.40** | **0.43** | **0.47** |
| $\Gamma$ (Eq. 3.11.5.3-2) | 2.80 | 2.97 | 2.86 | 2.69 |
| **Active Lateral Earth Pressure Coefficient, $k_a$ (Eq. 3.11.5.3-1)** | **0.26** | **0.23** | **0.25** | **0.28** |
| $-\delta/\phi_f$ | -0.5 | -0.5 | -0.5 | -0.5 |
| $\beta/\phi_f$ | 0.0 | 0.0 | 0.0 | 0.0 |
| Coefficient of Passive Pressure for $\beta/\phi_f$ = 0 and $-\delta/\phi f$ = -0.5, $k_p$ (Figure 3.11.5.4-2) | 9.0 | 13.0 | 10.3 | 7.5 |
| Reduction Factor of $k_p$, R (Figure 3.11.5.4-2) | 0.688 | 0.641 | 0.674 | 0.717 |
| **Coefficient of Passive Pressure, $k_p$** | **6.2** | **8.3** | **6.9** | **5.4** |

**Reference:** American Association of State Highway and Transportation Officials (AASHTO)
"AASHTO LRFD Bridge Design Specifications, Nineth Edition, 2020"

**Equations:** **At-Rest Lateral Earth Pressure Coefficient, $k_o$**

**3.11.5.2—At-Rest Lateral Earth Pressure Coefficient, $k_o$**

For normally consolidated soils, vertical wall, and level ground, the coefficient of at-rest lateral earth pressure may be taken as:

$$k_o = 1 - \sin\phi'_f \qquad (3.11.5.2\text{-}1)$$

where:

$\phi'_f$ = effective friction angle of soil
$k_o$ = coefficient of at-rest lateral earth pressure

**Active Lateral Earth Pressure Coefficient, $k_a$**

**3.11.5.3—Active Lateral Earth Pressure Coefficient, $k_a$**

Values for the coefficient of active lateral earth pressure may be taken as:

$$k_a = \frac{\sin^2(\theta + \phi'_f)}{\Gamma\,[\sin^2\theta \sin(\theta - \delta)]} \qquad (3.11.5.3\text{-}1)$$

in which:

$$\Gamma = \left[1 + \sqrt{\frac{\sin(\phi'_f + \delta)\sin(\phi'_f - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right]^2 \qquad (3.11.5.3\text{-}2)$$

where:

$\delta$ = friction angle between fill and wall taken as specified in Table 3.11.5.3-1 (degrees)
$\beta$ = angle of fill to the horizontal as shown in Figure 3.11.5.3-1 (degrees)
$\theta$ = angle of back face of wall to the horizontal as shown in Figure 3.11.5.3-1 (degrees)
$\phi'_f$ = effective angle of internal friction (degrees)

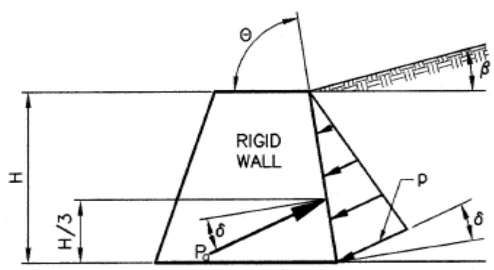



**Figure 3.11.5.3-1—Notation for Coulomb Active Earth Pressure**

**Table C3.11.5.3-1—Friction Angle for Dissimilar Materials (U.S. Department of the Navy, 1982a)**

| Interface Materials | Friction Angle, δ (degrees) | Coefficient of Friction, tan δ (dim.) |
|---|---|---|
| Mass concrete on the following foundation materials: | | |
| • Clean sound rock | 35 | 0.70 |
| • Clean gravel, gravel-sand mixtures, coarse sand | 29 to 31 | 0.55 to 0.60 |
| • Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel | 24 to 29 | 0.45 to 0.55 |
| • Clean fine sand, silty or clayey fine to medium sand | 19 to 24 | 0.34 to 0.45 |
| • Fine sandy silt, nonplastic silt | 17 to 19 | 0.31 to 0.34 |
| • Very stiff and hard residual or preconsolidated clay | 22 to 26 | 0.40 to 0.49 |
| • Medium stiff and stiff clay and silty clay | 17 to 19 | 0.31 to 0.34 |
| Masonry on foundation materials has same friction factors. | | |
| Steel sheet piles against the following soils: | | |
| • Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls | 22 | 0.40 |
| • Clean sand, silty sand-gravel mixture, single-size hard rock fill | 17 | 0.31 |
| • Silty sand, gravel or sand mixed with silt or clay | 14 | 0.25 |
| • Fine sandy silt, nonplastic silt | 11 | 0.19 |
| Formed or precast concrete or concrete sheet piling against the following soils: | | |
| • Clean gravel, gravel-sand mixture, well-graded rock fill with spalls | 22 to 26 | 0.40 to 0.49 |
| • Clean sand, silty sand-gravel mixture, single-size hard rock fill | 17 to 22 | 0.31 to 0.40 |
| • Silty sand, gravel or sand mixed with silt or clay | 17 | 0.31 |
| • Fine sandy silt, nonplastic silt | 14 | 0.25 |
| Various structural materials: | | |
| • Masonry on masonry, igneous and metamorphic rocks: | | |
| ○ dressed soft rock on dressed soft rock | 35 | 0.70 |
| ○ dressed hard rock on dressed soft rock | 33 | 0.65 |
| ○ dressed hard rock on dressed hard rock | 29 | 0.55 |
| • Masonry on wood in direction of cross grain | 26 | 0.49 |
| • Steel on steel at sheet pile interlocks | 17 | 0.31 |

**Passive Lateral Earth Pressure Coefficient, $k_p$**

**3.11.5.4—Passive Lateral Earth Pressure Coefficient, $k_p$**

For noncohesive soils, values of the coefficient of passive lateral earth pressure may be taken from Figure 3.11.5.4-1 for the case of a sloping or vertical wall with a horizontal backfill or from Figure 3.11.5.4-2 for the case of a vertical wall and sloping backfill. For conditions that deviate from those described in Figures 3.11.5.4-1 and 3.11.5.4-2, the passive pressure may be calculated by using a trial procedure based on wedge theory, e.g., see Terzaghi et al. (1996). When wedge theory is used, the limiting value of the wall friction angle should not be taken larger than one-half the angle of internal friction, $\phi_f$.

REDUCTION FACTOR (R) OF $k_p$ FOR VARIOUS RATIOS OF $-\delta/\phi_f$

| $\phi_f$ \ $\delta/\phi_f$ | −0.7 | −0.6 | −0.5 | −0.4 | −0.3 | −0.2 | −0.1 | 0.0 |
|---|---|---|---|---|---|---|---|---|
| 10 | .978 | .962 | .946 | .929 | .912 | .898 | .881 | .864 |
| 15 | .961 | .934 | .907 | .881 | .854 | .830 | .803 | .775 |
| 20 | .939 | .901 | .862 | .824 | .787 | .752 | .716 | .678 |
| 25 | .912 | .860 | .808 | .759 | .711 | .666 | .620 | .574 |
| 30 | .878 | .811 | .746 | .686 | .627 | .574 | .520 | .467 |
| 35 | .836 | .752 | .674 | .603 | .536 | .475 | .417 | .362 |
| 40 | .783 | .682 | .592 | .512 | .439 | .375 | .316 | .262 |
| 45 | .718 | .600 | .500 | .414 | .339 | .339 | .221 | .174 |

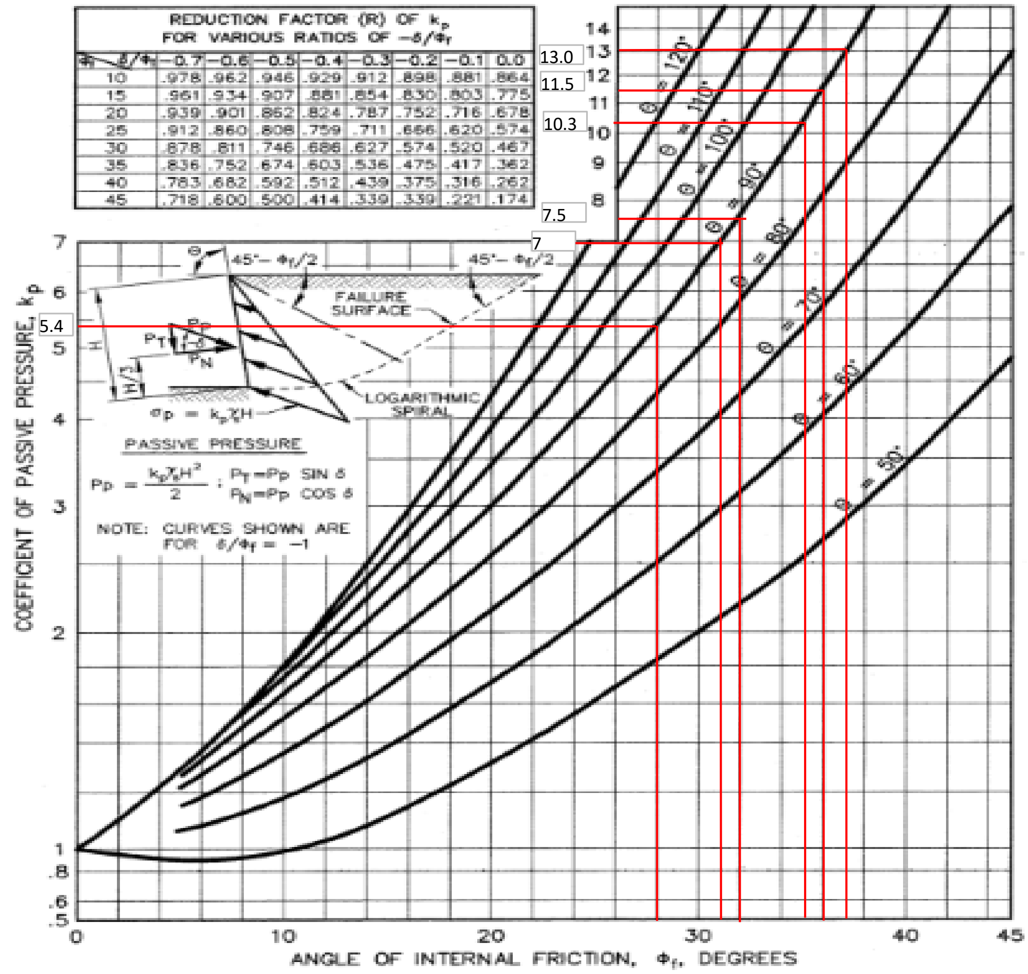



**Figure 3.11.5.4-1—Computational Procedures for Passive Earth Pressures for Vertical and Sloping Walls with Horizontal Backfill (U.S. Department of the Navy, 1982a)**

**Seismic Site Class Evaluation – Brandy Brow Road over East Meadow River**

Purpose: Evaluate seismic design criteria in accordance with 2011 AASHTO Guide Specifications for LRFD Seismic Bridge Design with 2012 through 2015 Interim Revisions and MassDOT 2020 LRFD Bridge Manual. Evaluate borings BB-1 based on $N_{60}$ values (Assuming CE=1.33 for automatic hammer).

| Layer | BB-1 [a] | | |
|---|---|---|---|
| | $N_i$ | Layer ($D_i$) | $D_i/N_i$ |
| 1[b] | 100 | 0 | 0.00 |
| 2 | 44 | 4 | 0.09 |
| 3 | 17 | 2 | 0.12 |
| 4 | 7 | 2 | 0.29 |
| 5 | 24 | 2 | 0.08 |
| 6 | 28 | 2 | 0.07 |
| 7 | 21 | 2 | 0.10 |
| 8 | 28 | 5 | 0.18 |
| 9 | 5 | 5 | 1.00 |
| 10 | 12 | 5 | 0.42 |
| 11 | 17 | 5 | 0.29 |
| 12 | 28 | 5 | 0.18 |
| 13 | 20 | 5 | 0.25 |
| 14 | 11 | 5 | 0.45 |
| 15 | 7 | 2 | 0.29 |
| 16 | 19 | 49 | 2.58 |
| | Σ = | 100.00 | 6.38 |
| | | $\overline{\mathbf{N}}$ | **15.7** |

**Notes**

a. Boring B-1 was terminated at 51 feet. Therefore, soil beneath bottom of boring to a depth of 100 feet is assumed to be Sand and Gravel layer with average N60 =19.

b. The high N-value at the surface was from the calculation as it was likely due to base material for road construction and therefore not representative.

$$N_{60} = N * C_E$$
$$where\ C_E = 1.33\ (from\ automatic\ hammer)$$

$$\overline{N} = \frac{\sum d_i}{\sum {d_i}/{N_i}}$$

From AASHTO Table C3.10.3.1-1

**From AASHTO Table 3.10.3.1-1 where 15 < $\overline{N}$ < 50**
**Site Class D**

**Site Seismic Coefficients**

| | | | |
|---|---|---|---|
| Horizontal Peak Ground Acceleration, | $PGA$ = | *0.119* | AASHTO Figs. 3.4.1-2b, -3b, and -4b (1000-yr return period) |
| Horizontal Response Spectral Acceleration (0.2 sec), | $S_s$ = | *0.197* | |
| Horizontal Response Spectral Acceleration (1 sec), | $S_1$ = | *0.038* | |

| | | |
|---|---|---|
| $F_{PGA}$ = | *1.6* | AASHTO Table 3.4.2.3-1 |
| $F_A$ = | *1.6* | AASHTO Table 3.4.2.3-1 |
| $F_V$ = | *2.4* | AASHTO Table 3.4.2.3-2 |

**Design Response Spectra**

| | | | | |
|---|---|---|---|---|
| Acceleration Coefficient | $A_s = PGA \times F_{PGA}$ | $A_S$ = | 0.190 | AASHTO Eq. 3.4.1-1 |
| Design Spectral Acceleration (0.2 sec), | $S_{DS} = S_S \times F_A$ | $S_{DS}$ = | 0.315 | AASHTO Eq. 3.4.1-2 |
| Design Spectral Acceleration (1 sec), | $S_{D1} = S_1 \times F_V$ | $S_{D1}$ = | 0.091 | AASHTO Eq. 3.4.1-3 |

**From AASHTO Table 3.5-1**
**Seismic Design Category A**

**2017 AASHTO Guide Specifications for LRFD Seismic Bridge Design:**

**Table 3.10.3.1-1—Site Class Definitions**

| Site Class | Soil Type and Profile |
|---|---|
| A | Hard rock with measured shear wave velocity, $\overline{v}_s > 5{,}000$ ft/s |
| B | Rock with 2,500 ft/sec $< \overline{v}_s < 5{,}000$ ft/s |
| C | Very dense soil and soil rock with 1,200 ft/sec $< \overline{v}_s < 2{,}500$ ft/s, or with either $\overline{N} > 50$ blows/ft, or $\overline{s}_u > 2.0$ ksf |
| D | Stiff soil with 600 ft/s $< \overline{v}_s < 1{,}200$ ft/s, or with either $15 < \overline{N} < 50$ blows/ft, or $1.0 < \overline{s}_u < 2.0$ ksf |
| E | Soil profile with $\overline{v}_s < 600$ ft/s or with either $\overline{N} < 15$ blows/ft or $\overline{s}_u < 1.0$ ksf, or any profile with more than 10.0 ft of soft clay defined as soil with $PI > 20$, $w > 40$ percent and $\overline{s}_u < 0.5$ ksf |
| F | Soils requiring site-specific evaluations, such as:<br>• Peats or highly organic clays ($H > 10.0$ ft of peat or highly organic clay where $H$ = thickness of soil)<br>• Very high plasticity clays ($H > 25.0$ ft with $PI > 75$)<br>• Very thick soft/medium stiff clays ($H > 120$ ft) |

**Table 3.4.2.3-1—Values of $F_{pga}$ and $F_a$ as a Function of Site Class and Mapped Peak Ground Acceleration or Short-Period Spectral Acceleration Coefficient**

| Site Class | Mapped Peak Ground Acceleration or Spectral Response Acceleration Coefficient at Short Periods | | | | |
|---|---|---|---|---|---|
| | $PGA \leq 0.10$ $S_s \leq 0.25$ | $PGA = 0.20$ $S_s = 0.50$ | $PGA = 0.30$ $S_s = 0.75$ | $PGA = 0.40$ $S_s = 1.00$ | $PGA \geq 0.50$ $S_s \geq 1.25$ |
| A | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |
| B | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| C | 1.2 | 1.2 | 1.1 | 1.0 | 1.0 |
| D | 1.6 | 1.4 | 1.2 | 1.1 | 1.0 |
| E | 2.5 | 1.7 | 1.2 | 0.9 | 0.9 |
| F | [a] | [a] | [a] | [a] | [a] |

Note: Use straight line interpolation for intermediate values of $PGA$ and $S_s$, where $PGA$ is the peak ground acceleration and $S_s$ is the spectral acceleration coefficient at 0.2 sec obtained from the ground motion maps.

[a] Site-specific response geotechnical investigation and dynamic site response analyses should be considered (Article 3.4.3).

**Table 3.4.2.3-2—Values of $F_v$ as a Function of Site Class and Mapped 1-sec Period Spectral Acceleration Coefficient**

| Site Class | Mapped Spectral Response Acceleration Coefficient at 1-sec Periods | | | | |
|---|---|---|---|---|---|
| | $S_1 \leq 0.1$ | $S_1 = 0.2$ | $S_1 = 0.3$ | $S_1 = 0.4$ | $S_1 \geq 0.5$ |
| A | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |
| B | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| C | 1.7 | 1.6 | 1.5 | 1.4 | 1.3 |
| D | 2.4 | 2.0 | 1.8 | 1.6 | 1.5 |
| E | 3.5 | 3.2 | 2.8 | 2.4 | 2.4 |
| F | [a] | [a] | [a] | [a] | [a] |

Note: Use straight line interpolation for intermediate values of $S_1$, where $S_1$ is the spectral acceleration coefficient at 1.0 sec obtained from the ground motion maps.

[a] Site-specific response geotechnical investigation and dynamic site response analyses should be considered (Article 3.4.3).

procedures.

**Table 3.5-1—Partitions for Seismic Design Categories A, B, C, and D**

| Value of $S_{D1} = F_v S_1$ | SDC |
|---|---|
| $S_{D1} < 0.15$ | A |
| $0.15 \leq S_{D1} < 0.30$ | B |
| $0.30 \leq S_{D1} < 0.50$ | C |
| $0.50 \leq S_{D1}$ | D |