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September 29, 2026

606352-136278

ADDENDUM NO. 2

To Prospective Bidders and Others on:

WAREHAM

**Federal Aid Project No. PRC(PRC80)-003S(925)X
Culvert and Dam Replacement on Cranberry Highway at Route 28 and Route 6,
Mill Pond Dam over Agawam River**

THIS PROPOSAL TO BE OPENED AND READ: WEDNESDAY, OCTOBER 14, 2026, at 2:00 P.M.

Transmitting revisions to the Contract Documents as follows:

<u>QUESTIONS AND RESPONSES:</u>	2 pages.
<u>DOCUMENT 00010:</u>	Revised page 2.
<u>DOCUMENT A00804:</u>	Inserted new document (322 pages).
<u>PLANS:</u>	Revised Sheets 5 and 106 of 125.

Please take note of the above, substitute the revised page for the original, insert the new document in proper order, substitute the revised plan sheets for the originals, and acknowledge Addendum No. 2 in your Expedite Proposal file before submitting your bid.

Very truly yours,

Eric M. Cardone Digitally signed by Eric M. Cardone
Date: 2026.09.29 08:20:57 -04'00'

Eric M. Cardone, P.E.
Construction Contracts Engineer

JPB
cc: Harry Adolphe, Project Manager

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WAREHAM
Federal Aid Project No. PRC(PRC80)-003S(925)X
Culvert and Dam Replacement on Cranberry Highway at Route 28 and Route 6,
Mill Pond Dam over Agawam River
(606352-136278)

Questions and Responses

Addendum No. 2, September 29, 2026

Northern Construction Services e-mail dated September 21, 2026.

Question 5.) Please provide the Geotechnical Report for the above MassDOT project, Letting Date 10.14.2026.

Response 5.) See revised document 00010 and new document A00804 Geotechnical Report.

MAS Building & Bridge, Inc. e-mail dated September 21, 2026.

Question 6.) The PUC form details two moves of the Aerial utilities. Phase 1 temporarily shifts Verizon from the South to the North to allow construction of the South half of the new spillway, bridge, and temp bridge. The 2nd move is listed as "Permanent" and shifts all Aerial utilities to the South. This same phasing sequence is listed on Page 22 of the Application for WQC. Phase 2A states to "permanently" relocate the overhead utilities to the South of the Dam. However, there are three (3) moves detailed on the plans. 1. Temporary relocation of Verizon to the North, 2. Temporary relocation of all aerial utilities to the South, 3. Permanent re-alignment along the South. Multiple drawings and notes detail a temporary alignment along the South, including Sheets 11, 28, 29, 50, and 54, as well as the Phasing sheets 59-61. If the Aerial Utilities are supposed to be installed in a temporary alignment along the South, can the Department please provide an updated PUC form to include the additional phase and anticipated duration?

Response 6.) On the current PUC Form, the durations listed for Permanent Overhead Utilities Stage 2, Tasks 1 – 6, account for both the temporary relocations and permanent realignment on the south side shown on the staging plans for Stage 2A and 6.

Question 7.) Control of Water for Phase 3 (construction of the new fish ladder) cuts off the existing primary spillway and existing fish ladder. Note 3 on Sheet 48 states that control of water and SOE shall be sequenced to prevent obstructing the fish ladder during migratory seasons. Please confirm the intent is to complete Phase 3 work over the winter between the Fall and Spring TOY windows.

Response 7.) Any restrictions or diversion of flow to remove the existing fish way or construct the proposed fish ladder shall only occur outside the Fall and Springtime of Year Restrictions.

Question 8.) The Profile view on Sheet 106 shows a temporary gas main supported under the temp bridge. The utility drawings do not show temporary gas supported under the temp bridge, and instead show the temp gas routed along the Southern limit of the project. The permanent gas is shown roadside (North) of the temp bridge location. Please confirm that a temporary gas line is not intended to be mounted to the temporary bridge.

WAREHAM
Federal Aid Project No. PRC(PRC80)-003S(925)X
Culvert and Dam Replacement on Cranberry Highway at Route 28 and Route 6,
Mill Pond Dam over Agawam River
(606352-136278)

Questions and Responses

Addendum No. 2, September 29, 2026

- Response 8.) There is no gas main proposed to be attached to the temporary bridge. See revised plan sheet 106.
- Question 9.) Phase 1 shows installation of the secant wall along the phase line, crossing the primary and auxiliary spillways. In order to complete the primary spillway secant penetration (per detail on Sheet 63 of 125), all flow must be diverted to the existing auxiliary spillway and fish ladder. Please confirm the existing auxiliary spillway and fish ladder provide the capacity to meet all flow requirements of the dam. If not, please provide flow rate capacity information for the entire system so any necessary bypasses can be sized appropriately.
- Response 9.) The auxiliary spillway and fish ladder are capable of being used to divert flow from the main spillway during construction of the proposed secant cutoff wall. The Contractor shall monitor and maintain the normal pool elevation in Mill Pond Dam by operating the stop logs at the auxiliary spillway based upon the flow conditions in the pond/river. The Contractor shall limit the duration that the flow is diverted from the main spillway as much as practicable. The Contractor shall include the proposed diversion procedure, including emergency contingency plans in the event of a large storm event, as part of their Control of Water procedure submittal for review by MassDOT and MADCR Office of Dam Safety
- Question 10.) Sheet 5 of 125, has a note to Reconstruct Fishway Channel and “Repair Fishway Roof”. Similarly, the review by the State Historic Commission, page A00840-273, states that the underground fishway remains in place and requires repairs to the culvert roof. Paragraph 3.8 under the Application for WQC also states, “The portion under the roadway and in the downstream areas will be preserved to the maximum extent practicable with repair and rehabilitation likely for certain areas, mostly anticipated under the roadway. However, Sheet 48 of 125 (demolition plan) shows the fishway under the road and to the South of the dam to be completely removed. Sheet 59 states “R&D portion of existing fish ladder” pointing to the section under the road. Also, under Section IV (Construction Sequence) of the Application for WQC, the Phase 5 note states, “Remove and Dispose portions of existing primary spillway and the remaining portion of the existing fish ladder under the roadway.” Please confirm the fishway under the roadway and underground along the south side of the road is to be removed completely without any repairs to the fishway roof as noted on Sheet 5.
- Response 10.) The fishway under the roadway and underground portion in the south embankment will be removed completely. See revised plan sheet 5.

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GEOTECHNICAL REPORT

Addendum No. 2, September 29, 2026

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**PREPARED FOR: MASSACHUSETTS DEPARTMENT OF
TRANSPORTATION – HIGHWAY (MASSDOT)**

GEOTECHNICAL REPORT

**Rehabilitation of Cranberry Highway (Route 28) / Mill Pond Dam
over the Agawam River**
National Dam ID#: MA00027 / State Dam ID#: 7-12-310-3
MassDOT Project File No. 606352
Proposed Bridge No. W-06-020
Wareham, Massachusetts



PREPARED BY:
PARE CORPORATION
10 LINCOLN ROAD, SUITE 210
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PARE PROJECT NUMBERS 14199.14 & 22147.02

December 2021
Revised March 2026



EXECUTIVE SUMMARY

The Massachusetts Department of Transportation (MassDOT) Highway Division has retained Pare Corporation (Pare) to complete a Geotechnical Investigation for the rehabilitation of Mill Pond Dam in Wareham Massachusetts.

This project consists of the removal and replacement of the existing primary and secondary spillways, fish way and fish ladder. The existing spillway system will be replaced by a single arched spillway built upstream of Cranberry Highway (Route 28) directing flow beneath the roadway via a new single span bridge carrying traffic over the Agawam River. The abutments of the concrete spillway will tie into the bridge abutments. The existing herring run fish way and fish ladder will be replaced by a pool and weir fish ladder with an entrance downstream of the bridge opening before completing a series of returns to gradually step up the passageway before passing under the proposed bridge superstructure and exiting upstream of the spillway.

A subsurface exploration program, consisting of three (3) soil borings (BB17-1, BB17-2, and BB17-3) was performed at the project site between September 28 and October 5, 2017 (for BB17-1 and BB17-2) and from April 11 to April 12, 2018 (for BB17-3) by New England Boring Contractors of Brockton, Massachusetts under subcontract with Pare.

In general, the subsurface profile consists of the following:

BB17-1 and BB17-2 (Roadway Borings)

- Up to approximately 8 to 10 inches of ASPHALT PAVEMENT (Stratum 1), underlain by,
- Approximately 6 to 17 feet of very loose to very dense GRANULAR FILL (Stratum 2); underlain by,
- Approximately 2 to 3 feet of TIMBER CONSTRUCTION DEBRIS (Stratum 3); underlain by,
- Approximately 56 to 60 feet of loose to very dense GLACIAL DEPOSITS (Stratum 4); underlain by
- Approximately 8 to 9 feet of WEATHERED SCHIST (Stratum 5); underlain by,
- SCHIST BEDROCK in BB17-2 or
- Approximately 5 feet of SCHIST BEDROCK (Stratum 6) underlain by GRANITE BEDROCK (Stratum 7) in BB17-1.

BB17-3 (Impoundment Boring)

- Approximately 4 feet of POND MUCK (Stratum 1B); underlain by,
- At least 49 feet of GLACIAL DEPOSITS (Stratum 4) to boring termination.

At the time of the drilling, groundwater depths along the dam's phreatic surface at BB17-1 and BB17-2 were measured at approximately 18.7 and 10.4 feet below the existing ground surface, corresponding to elevations¹ 0.8 and 8.9 feet with no artesian conditions encountered. The pond surface was at approximately El. 12.3 feet during the completion of Boring No. B17-3.

Based upon the observed relative densities, groundwater elevation, and material composition, it appears that the in-situ soils are NOT susceptible to liquefaction.

¹ NAVD88



From the American Association of State Highway and Transportation Officials (AASHTO), 9th Edition, Table 3.10.3.1-1, the project site is generally characterized as Site Class Profile D, “Stiff Soil Profile” (i.e., $15 < \bar{N} < 50$).

An analysis has been conducted of the potential earth pressure generated during a seismic event in accordance with the Mononobe-Okabe method. For this analysis, an Acceleration Coefficient of 0.079 was used, as retrieved from the Mass DOT LRFD Bridge Design Manual, figure title “USGS Seismic Hazard Map for the 2500-Yr Event PGA Contours. The results indicate that the dynamic active earth pressure coefficient (K_{ae}) is approximately 0.40.

Based on the results of the subsurface investigation and the proposed structures, Pare recommends the substructure for the proposed primary spillway and proposed Bridge No. W-06-020 to be supported on a continuous concrete mat foundation from the downstream face of the spillway to the downstream end the bridge. Associated bridge return walls are recommended to be supported by conventional shallow foundations or spread footings. The proposed foundations would be founded on a minimum 12-inch-thick layer of compacted “Gravel Borrow” placed on the insitu GLACIAL DEPOSITS (Stratum 4). If the footprint of proposed foundations extend over areas that contain GRANULAR FILL (Stratum 2) or TIMBER CONSTRUCTION DEBRIS (Stratum 3), it is recommended that the GRANULAR FILL and TIMBER CONSTRUCTION DEBRIS be removed from within the influence zone below foundations and replaced with compacted “Gravel Borrow”. Estimated bearing capacities and corresponding foundation widths are summarized in Table 3-1:

Due to the close proximity of the Agawam River and the requirement to reduce impacts on environmental features, temporary earth support will likely be required to maintain the river channel and embankment. Depending on the temporary earth support system implemented by the Contractor, existing structures (if any) to remain within 10 feet of earth support system should be stabilized and/or underpinned and monitored for settlement. If the Contractor installs a driven earth support system (sheet piles, soldier piles and lagging, etc.), due to the presence of the GRANULAR FILL stratum, Pare recommends that the temporary earth support system remains in place after the structures are built, and the elements of the support are cut off to no less than 12 inches below finish grade. If the temporary earth support systems were to be removed, the vibrations caused during the removal could temporarily weaken the surrounding soils and possibly cause excessive settlement or lateral movement of the new structures.

Groundwater was encountered near the excavation depths likely for this project. Temporary dewatering is proposed for this project. The impoundment should be drawdown, and/or a cofferdam/portable dam system should be installed throughout the construction duration to reduce and to better-control the hydrostatic head pressure through the dam. A drawdown below normal pool was rejected by local authorities. The countermeasures were included in the proposed temporary construction methods. The proposed bridge and other improvements are located on an active dam; therefore, dewatering methods will require specialized methods beyond a typical bridge foundation dewatering program. Therefore, it is recommended that the dewatering program design be stamped by a Massachusetts-registered professional engineer experienced in dam construction and repair and submitted to the State prior to the start of construction.

It is recommended to keep the existing fish ladder and primary spillway in operation to maintain flows through the dam while the existing auxiliary spillway is demolished and removed and until the proposed spillway, fish ladder and bridge structures are completed and in operation.

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1.0 INTRODUCTION

The following report presents and interprets the results of a geotechnical investigation for the design and construction of the rehabilitation of Mill Pond Dam, including, but not limited to, the installation of proposed Bridge No. W-06-020 in Wareham, Massachusetts, as shown on Figure 1: Locus Plan. The dam allows Cranberry Highway (Route 6/28) to pass over Agawam River. The resulting impoundment to the north of the dam is Agawam Mill Pond. It is anticipated that phased construction will be needed in order to maintain traffic flows along Cranberry Highway during construction. The report was prepared for the Massachusetts Department of Transportation – Highway Division (MassDOT), and is subject to Pare Corporation's (Pare's) Geotechnical Limitations included in Appendix D.

1.1 Scope of Investigation

The purpose of this geotechnical investigation is to characterize the subsurface conditions at the site; interpret the results of the subsurface investigation; provide design recommendations and criteria; and construction considerations/recommendations.

The scope of work included:

- Reviewing previous reports,
- Advancing three (3) soil borings (BB17-1 through BB17-3), to collect and document soil and rock core sampling,
- Completing foundation analyses including bearing capacity, settlement, and seismic,
- Developing design and construction recommendations, and
- Preparing this Geotechnical Report.

1.2 Subject Background, Proposed Construction, History

The proposed rehabilitation includes the following work:

- The removal of the existing primary spillway and culvert (W-06-020(9LH));
- The abandonment¹ and replacement of the existing fish ladder and fish way (W-06-020(8KD));
- The removal of the existing secondary (auxiliary) spillway and culvert (W-06-020(8M1));
- The partial removal of stone masonry walls at the dam; and
- The construction of a new primary spillway and bridge (W-06-020).

The dam structure for which all the above-listed structures and improvements are located is the Mill Pond Dam (National Dam ID#: MA00027).

It is Pare's understanding that the proposed bridge is to be located in the area of the auxiliary spillway. The proposed bridge's single span superstructure will be prefabricated bridge units (PBU), made of rolled steel stringers with an 8-inch composite concrete deck. The superstructure would be fabricated off-site and transported to the site for installation. Pare understands that the proposed bridge will be built in multiple phases to allow Cranberry Highway to remain open during the demolition and construction process.

¹ The existing upstream fish way open channel is proposed to be filled in with flowable fill, sealed with a concrete plug at the upstream end, and left in place due to historical requirements. A pipe is proposed to be installed through the existing buried section of the fish way to control the level of the resulting wetland located to the east.

The existing spillway system is proposed to be replaced by an arched sharp-crested-weir spillway built upstream of Cranberry Highway (Route 28) directing flow beneath the roadway via a proposed single span bridge carrying traffic over the Agawam River. The geometry of the proposed spillway weir will consist of a segmental arch with an approximate 50-foot radius and 13.5-foot-long linear segment, providing a total weir length of approximately 184 feet. The abutments of the concrete spillway will tie into the proposed bridge abutments. The proposed primary spillway will be capable of passing flows associated with a ½ Probable Maximum Flood (1/2-PMF) Spillway Design Flood (SDF), for which the existing system could not accommodate.

A gated low-level outlet is proposed to be built into the west side of the new spillway near the base of its arch with flows discharging into the new spillway channel upstream of the bridge.

The existing herring run fish way and fish ladder will be replaced by a more efficient Denil fish ladder composed of a rectangular chute which starts approximately 25 feet downstream from the bridge opening before completing a series of returns to gradually step up the passageway before passing under the proposed bridge superstructure and exiting upstream of the spillway.

1.3 Site Reconnaissance and Overall Description

As described in Pare's Type Study Selection Worksheet, dated May 2015:

The existing primary spillway, located on the east side of the embankment, is 11'6" long and consists of a concrete box culvert carrying flows from the impoundment below the roadway to a timber-stop-log-controlled concrete spill structure on the downstream side of the embankment.

The existing auxiliary spillway, located on the west side of the embankment, consists of a timber-stop-log-controlled concrete structure on the impoundment side followed by a 5-foot diameter riveted steel pipe which carries flows below the roadway and discharges into the Agawam River through a concrete headwall at the downstream side of the dam.

A herring run fish way is present at the left abutment, east of the primary spillway. The fish way is a rectangular shaped conduit about 5 feet wide and 3.5 feet high. The fish way starts underground at the toe of the energy-dissipating step in the primary spillway discharge channel, and steps up and passes under the roadway into a stonewall-lined open channel that extends about 325 feet upstream of the roadway before exiting into the impoundment.

The section of Cranberry Highway that crosses the Mill Pond Dam carries traffic in the westbound direction via two (2) lanes with 4- to 5-foot-wide sidewalks on both sides.

The existing site grades are generally flat along Cranberry Highway. Local businesses and residential houses are in the surrounding area.

Mill Pond Dam is considered a Large, High Hazard Potential structure with a drainage area of approximately 11.4 square miles as indicated in the National Inventory of Dams. During construction, the flow of water will need to be maintained for environmental reasons and during harvesting of the cranberry bogs downstream of the dam. The fish ladder must remain operational at specific times during construction so that anadromous fish and other marine wildlife movement patterns are not disturbed.



2.0 SUBSURFACE CONDITIONS

2.1 Subsurface Exploration Program

A subsurface exploration program, consisting of three (3) soil borings (BB17-1 through BB17-3) was performed at the project site between September 28 and October 5, 2017 (for BB17-1 and BB17-2) and on April 11 and 12, 2018 (for BB17-3) by New England Boring Contractors of Brockton, Massachusetts under subcontract with Pare. The soil borings were undertaken to develop a subsurface profile and model as described herein. The logs of the soil borings are included in Appendix A and their locations are indicated on Figure 2: Exploration Location Plan.

Pare's field personnel observed the drilling conditions; visually identified soil and rock core samples, and recorded groundwater levels (on date of boring) at the end of the exploration. Soil boring No. BB17-1 and BB17-2 were advanced using a truck-mounted drill rig while soil boring No. BB17-3 was advanced using a barge-mounted skid rig. Both rigs used drilling and wash techniques with 4-inch diameter casing.

2.1.1 Available Subsurface Information

Information from subsurface investigations previously performed in the area was available during the preparation of this report. This information includes Boring B-1 through boring B-3 performed between November 24 and December 1, 2008; and boring B-4 through boring B-6 performed between May 3 and May 5, 2010 by New Hampshire Boring, Inc. HA-1 through HA-4 performed by Pare on November 24, 2008 are also included within this report. The respective boring logs are included in Appendix A and their locations are shown on Figure 2: Exploration Location Plan.

2.1.2 Regional Geological Setting

This section presents surficial and bedrock geological information for the project area, based on United States Geological Survey (USGS) maps.

2.1.2.1 Surficial Geology

Based on the "Surficial Geologic Map of the Wareham Quadrangle, Massachusetts" compiled by Bryon D. Stone, Janet R. Stone, Mary L. DiGiacomo-Cohen, and Kevin A. Kincare, 2011 Mass USGS, the surficial geology underlying the project area is identified as "Artificial Fill" and "Coarse Stratified Deposits" and is described as follows:

"Artificial Fill" includes earth materials and manmade material that have been artificially emplaced, primarily in highway and railroad embankments, and in dams; may also include landfills, urban develop areas, and filled coastal wetlands.

"Coarse Stratified Deposits" include gravel deposits composed of mainly gravel-sized clasts; cobbles and boulders predominate; minor amounts of and within gravel beds, and sand comprises few separate layers. Gravel layers generally are poorly sorted, and bedding commonly is distorted and faulted due to post depositional collapse related to melting of ice. Sand and gravel deposits composed of mixtures of gravel and sand within individual layers and as alternating layers. Sand and gravel layers generally range from 25 to 50 percent gravel particles and from 50 to 75 percent sand particles. Layers are well to poorly sorted; bedding may be distorted and faulted due to post depositional 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.

2.1.2.2 Bedrock Geology

According to the Bedrock Geologic Map of Massachusetts, the bedrock throughout the area of the project consists of “granite, gneiss, and schist, undivided (Proterozoic Z). Plutonic and metamorphic rocks of portable Proterozoic Z age. May include phylutonic and volcanic rocks of Paleozoic or younger age.”

2.1.3 Boring/Soil/Pavement Sample Review

As illustrated on Figure 3: Exploration Location Plan, Soil boring No. BB17-1 and BB17-2 were advanced through the southern edge of the existing roadway on Mill Pond Dam with BB17-1 located west of the existing auxiliary spillway and BB17-2 approximately halfway between the primary spillway and the auxiliary spillway. Boring No. BB17-1 and BB17-2 were advanced to a depth of approximately 95 and 94 feet below the existing ground surface, respectively.

Continuous split spoon samples were obtained within BB17-1 from 1 to 21 feet, within BB17-2 from 1 to 11 feet, and within BB17-3 from 0 to 20 feet below the existing roadway surface. Unless noted otherwise on the logs, standard sampling was performed at minimum 5-foot intervals within BB17-1 from 21 feet to 85 feet, within BB17-2 from 11 to 61 feet, and within BB17-3 from 20 to 48.5 feet below the existing roadway surface.

Sampling was performed in general accordance with ASTM D-1586, the Standard Penetration Test (SPT), to obtain an indication of the relative density and consistency of the underlying soils. The SPT consists of driving a 1 3/8-inch inside diameter standard split spoon sampler at least 18 inches with a 140-pound hammer dropping from a height of 30 inches. The standard penetration value (N) is the number of blows required to drive the sampler from 6 to 18 inches of penetration.

2.2 Verification of Sample Descriptions on Boring Logs

During the explorations, subsurface soils were visually classified utilizing the Burmister Classification System. This system describes soil composition based upon the percentage of soil particle size present by weight in the sample with the major soil particle size listed first following other soil components described as “trace” indicating 0-10% by weight, “little” indicating 10-20% by weight, “some” indicating 20-35% by weight, or “and” indicating 35-50% by weight. The Burmister Classification System as modified by MassDOT describes soil components using the same procedure except the original “little” component was deleted and the “some” component was redefined to indicate 10-40% by weight. For consistency, this report will use the Burmister Classification.

2.3 Subsurface Profile

In general, the subsurface profile consists of the following:

BB17-1 and BB17-2 (Roadway Borings)

- Up to approximately 8 to 10 inches of ASPHALT PAVEMENT (Stratum 1), underlain by,
- Approximately 6 to 17 feet of very loose to very dense GRANULAR FILL (Stratum 2); underlain by,



- Approximately 2 to 3 feet of TIMBER CONSTRUCTION DEBRIS (Stratum 3); underlain by,
- Approximately 56 to 60 feet of loose to very dense GLACIAL DEPOSITS (Stratum 4); underlain by
- Approximately 8 to 9 feet of WEATHERED SCHIST (Stratum 5); underlain by,
- SCHIST BEDROCK in BB17-2 or
- Approximately 5 feet of SCHIST BEDROCK (Stratum 6) underlain by GRANITE BEDROCK (Stratum 7) in BB17-1.

BB17-3 (Impoundment Boring)

- Approximately 4 feet of POND MUCK (Stratum 1B); underlain by,
- At least 49 feet of GLACIAL DEPOSITS (Stratum 4) to boring termination.

The subsurface profiles are described further in the following sections.

2.3.1 Stratum 1 – PAVEMENT

ASPHALT PAVEMENT was encountered at the top of boring No. BB17-1 and BB17-2, ranging from 8 to 10 inches thick.

2.3.2 Stratum 1A – POND MUCK

Approximately 4 feet of POND MUCK was encountered at the top of boring No. BB17-3. POND MUCK is described as a heterogeneous mixture of sand, gravel, and organic silt.

The POND MUCK is not considered a suitable bearing stratum for foundations due to its high organic content.

2.3.3 Stratum 2 – GRANULAR FILL

GRANULAR FILL, described as dark to light brown, fine to coarse SAND, “some” to “and” amounts of fine to coarse gravel, “trace” to “some” amounts of silt, and “trace” amounts of coal, extends from directly below Stratum 2 at BB17-1 and BB17-2 to respective depths of about 13 and 14 feet below the ground surface. SPTs performed in Stratum 2 indicated a relative density of very loose to very dense.

The GRANULAR FILL is not considered a suitable bearing stratum for foundations in its current state due to its irregular consistency and densities (i.e., concern for excessive differential settlement and inadequate bearing capacity).

2.3.4 Stratum 3 – TIMBER CONSTRUCTION DEBRIS

TIMBER CONSTRUCTION DEBRIS extends from directly below Stratum 2 (at BB17-1 and BB17-2) to about 19 feet below the ground surface.

The TIMBER CONSTRUCTION DEBRIS is not considered a suitable bearing stratum for foundations due to its compressible and degradable properties.



2.3.5 Stratum 4 – GLACIAL DEPOSITS

GLACIAL DEPOSITS, generally described as light brown, light gray, or gray, fine to coarse SAND, “trace” to “and” amount of SILT, “trace” to “and” amounts of fine to coarse gravel extends from directly below Stratum 3 at BB17-1 and BB17-2 to respective depths of about 77 and 75 feet, or directly below Stratum 1A at BB17-3 to boring termination at 53 feet. SPTs performed in Stratum 4 indicate a relative density ranging from medium dense to very dense with the exception of a loose area as represented by sample S-7 retrieved from BB17-3 from 12 to 14 feet below the existing mudline.

The in-situ Glacial Deposits Stratum is considered a suitable bearing stratum for foundations.

2.3.6 Stratum 5 – WEATHERED BEDROCK

WEATHERED BEDROCK extends from directly below Stratum 4 at BB17-1 and BB17-2 to respective depths of about 85 and 84 feet below the existing ground surface. SPTs performed in Stratum 5 indicate a very dense relative density.

The WEATHERED BEDROCK is considered a suitable bearing stratum for foundations.

2.3.7 Stratum 6 – SCHIST BEDROCK

SCHIST BEDROCK, described as “fair” to “very poor”, gray, moderately to extremely fractured, hard SCHIST, extends from directly below Stratum 5 at BB17-1 and BB17-2 to a depth of about 93 feet at BB17-1 and to boring termination (94 feet) at BB17-2.

The SCHIST BEDROCK is considered a suitable bearing stratum for foundations.

2.3.8 Stratum 7 – GRANITE BEDROCK

GRANITE BEDROCK, described as fair, orange, moderately fractured, hard GRANITE, extends directly below Stratum 6 at BB17-1 to boring termination (95 feet).

The GRANITE BEDROCK is considered a suitable bearing stratum for foundations.

2.3.9 Rock Quality Designations (RQD's)

Rock Quality Designations (RQD's) were estimated as follows:

<u>Boring, Core No., Depth</u>	<u>Recovery (%)</u>	<u>RQD (%)</u>	<u>Description</u>
BB17-1, C-1, 85 to 90 feet	97	71.7	Fair
BB17-1, C-2, 90 to 95 feet	100	65	Fair
<u>Boring, Core No., Depth</u>	<u>Recovery (%)</u>	<u>RQD (%)</u>	<u>Description</u>
BB17-2, C-1, 84 to 89 feet	50	0	Very Poor
BB17-2, C-2, 89 to 94 feet	27	0	Very Poor



2.3.10 Phreatic Surface

At the time of the drilling, groundwater depths along the dam's phreatic surface at BB17-1 and BB17-2 were respectively measured at approximately 18.7 and 10.4 feet below the existing ground surface, corresponding to elevations of 0.8 and 8.9 feet with no artesian conditions encountered. Groundwater and impoundment level measurements During the drilling of BB17-1 and BB17-2, the impoundment water elevation was not measured. For informational purposes only, historic impoundment water levels are typically within 1 foot of the normal operating level (El. 13.6).

During the drilling of boring BB17-3, the impoundment water (Agawam Mill Pond) was measured at El. 12.3 feet.

2.4 Grain Size Analysis Results

The laboratory testing program included 2 mechanical grain size determinations performed upon samples from the strata encountered during the investigation. The tests were performed in general accordance with ASTM D-6913. The results of the laboratory testing are summarized below. The data sheets are included in Appendix B.

TABLE 2-1: RESULTS OF GRAIN SIZE ANALYSES

Test No.	Boring No.	Sample No.	Depth (Feet)	Representative Soil Strata	% Gravel	% Sand	% Fines
1	BB17-1	S-10	19-21	GLACIAL DEPOSITS	0.4	98	1.8
2	BB17-2	S-5	9-11	GRANULAR FILL	9	78	13

It should be noted that the results presented above may not give a true representation of the gradation of the soils present below the site as the soils were sampled using a 1 3/8-inch inside diameter standard split spoon sampler in the borings. This sampler will collect limited coarse gravel and no cobbles or boulders. Due to these limitations, these particles are not represented within the grain size distribution.

2.5 Seismic Design Category and Liquefaction Evaluation Potential

2.5.1 Site Criteria

In accordance with the AASHTO LRFD Bridge Design Specification, 9th edition, Table 3.10.3.1-1 and based on the boring information, the soil profile at the project site is characterized as Site Class Profile D, "Stiff Soil Profile" (i.e., $15 < \bar{N} < 50$).

2.5.2 Seismic Design Parameters

Seismic design parameters were calculated in accordance with *AASHTO Guide Specifications for LRFD Seismic Bridge Design*, second edition. The table below lists the seismic design parameters for this project:

TABLE 2-2: SEISMIC DESIGN PARAMETERS

Parameter	Definition	Value
A_s	Acceleration Coefficient	0.126g
S_{DS}	Design Spectral Acceleration Coefficient at 0.2-sec period	0.21
S_{D1}	Design Spectral Acceleration Coefficient at 1.0-sec period	0.113
SDC	Seismic Design Category	A

2.5.3 Liquefaction Evaluation Potential

Liquefaction is the tendency for a soil type, particularly fine sands, to lose a significant amount of strength and behave similar to a liquid in the event of an earthquake, or sufficient vibrations. Liquefaction analyses generally relate SPT N values, corrected for overburden, hammer efficiency, fines content, and measured groundwater levels, to the liquefaction potential of the materials in question. In general, in order for liquefaction to occur three conditions have to be met simultaneously. These are: 1.) loose sandy soils susceptible to liquefaction, 2.) saturated soil conditions, and 3.) vibration.

The liquefaction analyses completed during the preparation of this report was performed in general accordance with the summary report from the 1996 NCEER (National Center for Earthquake Engineering Research) and 1998 NCEER/NSF (National Science Foundation) workshops on “Evaluation of Liquefaction Resistance of Soil” prepared by Youd et al. (2001). The analysis takes into account the soil and groundwater conditions encountered at the time of the subsurface exploration program. It should be noted that fluctuations in groundwater levels can have a significant effect upon the liquefaction potential of soils. If the groundwater is observed to change during the construction process or future explorations, Pare should be informed as it may be necessary to re-analyze the soil for liquefaction potential.

Based upon the observed relative densities, groundwater elevations, and material composition, it appears that the in-situ soils are NOT susceptible to liquefaction.

2.5.4 Seismic Active Pressure Coefficient

An analysis has been conducted of potential earth pressure generated during a seismic event in accordance with the Mononobe-Okabe method. For this analysis, an Acceleration Coefficient (A) of 0.079 was used, as interpolated from *MassDOT LRFD Bridge Manual*, “USGS Seismic Hazard Map for the 2500-YR Event GPA Contours.”

For free standing abutments or retaining walls, which may displace horizontally without significant restraint (e.g., superstructure supported by sliding bearings), with drained backfill, an effective active earth pressure coefficient (K_{ae}) of **0.40** is recommended.



3.0 RECOMMENDED FOUNDATION SYSTEM

3.1 Basis for Foundation Recommendations

Based on the results of the subsurface investigation and the proposed structures, Pare recommends the substructure for the proposed primary spillway and proposed Bridge No. W-06-020 to be supported on a continuous concrete mat foundation from the downstream face of the spillway to the downstream end the bridge. Associated bridge return walls are recommended to be supported by conventional shallow foundations or spread footings. The proposed foundations would be founded on a minimum 12-inch-thick layer of compacted “Gravel Borrow” placed on the insitu GLACIAL DEPOSITS (Stratum 4). If the footprint of proposed foundations extend over areas that contain GRANULAR FILL (Stratum 2) or TIMBER CONSTRUCTION DEBRIS (Stratum 3), it is recommended that the GRANULAR FILL and TIMBER CONSTRUCTION DEBRIS be removed from within the influence zone below foundations and replaced with compacted “Gravel Borrow”. The influence zone below foundations can be defined by the area bound by a one horizontal to one vertical plane extending downward from the outside edges of the foundation.

In addition to the proposed substructure recommendations above, a temporary bridge will be required during Phase 2 of construction to maintain two lanes of traffic for that phase. The bridge will span over the existing main culvert and spillway to accommodate one lane of vehicular traffic and a temporary pedestrian walkway. Pare recommends the temporary bridge be supported by abutments with shallow foundations or spread footings. Due to the temporary nature of the structure, Pare recommends stub abutments to minimize disturbance to the earthen dam and facilitate removal of the concrete after Phase 2 is complete. If TIMBER CONSTRUCTION DEBRIS (Stratum 3) is encountered, Pare recommends removing the material from the influence zone and replacing with compacted “Gravel Borrow”.

3.1.1 Seepage Control

Prior to foundation installations, Pare recommends to design and install a continuous cutoff wall in areas below the footprint of the proposed spillway and bridge mat foundation. The purpose of this wall is to lengthen the flow path through the dam, reducing the seepage gradient, resulting in a higher factor of safety against piping failure. The cutoff wall could consist of a reinforced secant wall.

Based on the findings of Boring No. BB17-1 and BB17-2, TIMBER CONSTRUCTION DEBRIS (Stratum 3) extends directly below the FILL (Stratum 2) from about 13 to 19 feet below the existing roadway surface. It is possible that the timber could be the remains of a timber cribbing mat used for ground stabilization, a known construction practice at the time the typically used for construction prior to 1900.

In addition to the cutoff wall, water stops are recommended to be installed at all joints exposed to phreatic and open flow areas.

The temporary bridge will be located behind the cutoff wall placed during Phase 1 of construction, therefore, seepage should not be a concern for the structure.

3.1.2 Frost Depth

In conformance Subchapter 3.2 of the *MassDOT LRFD Bridge Manual*, Part II, exterior footings founded over soils should be placed at a minimum depth of 48 inches below the finished grade in order to



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provide for frost protection. Preparation for slabs and paved areas should consider the frost heave susceptibility of subgrade soils.



3.1.3 Shallow Foundation Bearing Capacity and Settlement

3.1.3.1 Bearing Resistance

Factored bearing resistances at the strength limit state (q_R) and settlements are summarized in Table 3-1 with a bearing resistance factor (ϕ_b) of 0.45 for mat and shallow foundations of varying widths. For mat foundations, widths ranging from 15 to 25 feet were assumed for phased construction, and widths ranging from 60 to 65 feet wide were assumed for final construction. For shallow foundations, widths ranging from 10 to 20 feet were assumed for full construction. Foundation embedments were assumed at 5 feet for mat foundations and 4 feet for shallow foundations.

TABLE 3-1: FACTORED BEARING RESISTANCES AT THE STRENGTH LIMIT STATE ($q_R = \phi_b q_n$)

Case	Description	Width, B (ft)	q_R (psf)	Total Settlement (in.)
1	Mat Foundation under Bridge (Phased Construction)	15	6054	0.89
		20	7152	1.37
		25	8147	1.81
2	Mat Foundation under Spillway (Phased Construction)	20	8866	3.12
		25	10329	4.15
		30	11724	5.11
3	Shallow Foundations under Downstream Training Walls	10	4311	0.42
		15	5314	0.78
		20	6128	1.21
4	Entire Mat Foundation Under Spillway	65	15315	6.76
5	Entire Mat Foundation Under Bridge	60	15768	4.43
6	Temporary Bridge	10	5970	1.36

3.1.3.2 Settlement

It is recommended that at any given time during construction, total settlements should not exceed 1 inch and differential settlements between foundation elements be no greater than 0.5 inch. For the foundation designer's use in limiting total and differential settlements throughout the construction phases to final construction, Table No. 3-2A and 3-2B provide Service Limit Design Applied Bearing Pressures (q_0) for a range of estimated settlements.



**TABLE 3-2A: SERVICE LIMIT DESIGN APPLIED BEARING PRESSURES (q_0)
FOR CASE 1&2 FOUNDATIONS**

Total Settlement* (in.)	Short-Term Settlement* (in.)	Case 1, q_0 (psf)			Case 2, q_0 (psf)		
		Width, B (ft)			Width, B (ft)		
		15	20	25	15	20	25
0.2	0.13	2000	1400	1550	1400	1225	1150
0.4	0.27	3250	2750	2500	2175	1850	1725
0.6	0.40	4450	3750	3400	2900	2475	2275
0.8	0.53	5550	4650	4250	3600	3050	2825
1	0.67	-	5400	4900	4200	3500	3200

* Total Settlement is the estimated long-term settlement occurring over an approximate 10-year span with 2/3rds of the total settlement occurring immediately (within the first month), hence called Short-Term Settlement.

**TABLE 3-2B: SERVICE LIMIT DESIGN APPLIED BEARING PRESSURES (q_0)
FOR CASE 3 THRU CASE 5 FOUNDATIONS**

Total Settlement* (in.)	Short Term Settlement* (in.)	Case 3, q_0 (psf)			Case 4, q_0 (psf)	Case 5, q_0 (psf)
		Width, B (ft)			Width, B (ft)	Width, B (ft)
		10	15	20	65	60
0.2	0.13	2500	1900	1600	1100	1400
0.4	0.27	4150	3150	2650	1650	2250
0.6	0.40	4300	4300	3625	2150	3050
0.8	0.57	-	5400	4550	2650	3800
1	0.67	-	-	5300	3150	4550

* Total Settlement is the estimated long-term settlement occurring over an approximate 10-year span with 2/3rds of the total settlement occurring immediately (within the first month), hence called Short-Term Settlement.

Where,

Case 1: Mat Foundation under Bridge

Case 2: Mat Foundation under Spillway

Case 3: Spread Footings under Downstream Training Walls

Case 4: Entire Mat Foundation Under Spillway

Case 5: Entire Mat Foundation Under Bridge

Case 6, the temporary bridge foundation, was not included in the tables due to the temporary condition and a settlement less than 1" shown in Table 3-1.

3.1.4 Expansion Joints

For the continuous concrete mat foundation supporting the proposed spillway and bridge and for spread footings supporting associated walls, the total settlements of the foundations are expected to be less than or equal to 1.0 inch and the maximum differential settlements between foundation elements are expected to be less than or equal to 0.5 inches. It is recommended that expansion joints be installed as a minimum between retaining walls and abutments and across the mat foundation to allow for differential settlement.



3.1.5 Lateral Earth Pressures and Retaining Wall Design

For the design of abutment and spillway weir walls with level backfill, recommended lateral earth pressure coefficients are indicated in Table 3-3. The lateral earth pressure coefficient should be increased where the ground surface slopes up behind the wall. The retaining walls should be designed to withstand surcharge loading which may be present over the life of the structure. These would include traffic loads, dead and live loads associated with equipment expected to operate or be stored within the influence area of the wall, fill, or construction equipment which may be placed adjacent to the wall. The influence area behind the wall can be defined by a one horizontal to one vertical line extending upward from the outside edge of the wall footing.

The magnitude of lateral earth pressure against retaining walls is dependent upon the type of backfill, method of fill placement, drainage provisions, and the amount of yielding the wall is permitted to undergo after the placement of the backfill. Pare recommends that the foundation and retaining walls be backfilled with a free draining "Gravel Borrow" that meets M1.03.0 Type b of the Massachusetts Standard Specification for Highways and Bridges

The lateral earth pressure distribution against retaining walls should be computed using the appropriate value of K, the coefficient of lateral earth pressure. Recommended values of K are presented in Table 3-3 below. Friction factors are also presented for use in checking resistance to unbalanced forces on walls.

For flexible structures, active and/or passive lateral earth pressures are recommended. However, each wall should be checked during design to determine if wall movement relative to the height, H, of the wall, is such that at-rest, active, or passive lateral earth pressures are more appropriate. The amount of wall rotation necessary to develop the active pressure state (i.e., wall rotates away from backfill) is about 0.001H to 0.004H. The amount of wall rotation to develop the passive pressure state (i.e., wall rotates toward backfill) is about 0.005H to 0.01H.

**TABLE 3-3: RECOMMENDED EARTH PRESSURE
AND FRICTION COEFFICIENTS**

Material	At-Rest Coefficient (K)	Active Coefficient (K _a)	Passive Coefficient (K _p)
Imported Gravel Borrow ($\Phi = 35^\circ$)	0.43	0.25	3.7
Existing GRANULAR FILL ($\Phi = 28^\circ$)	0.53	0.32	2.8
Improved Existing GRANULAR FILL ($\Phi = 33^\circ$)	0.46	0.27	3.4
FRICTION COEFFICIENTS			
Concrete Poured on Imported Gravel Borrow or on Glacial Deposits (Stratum 4)			$\tan \delta = 0.45$
Precast Concrete on Imported Gravel Borrow or on Glacial Deposits (Stratum 4)			$\tan \delta = 0.30$

Traffic loads and other anticipated loadings that could occur behind the walls should be considered. In addition, the effect of adjacent footings on lateral walls should be accounted for during design.



3.1.6 Global Stability (Substructure-Slope-Subsoil System)

To provide global stability for proposed abutments and associated walls, it is recommended that slopes adjacent to and beyond the walls and/or slopes adjacent to the bridge abutments have a minimum slope of 2H to 1V to provide a minimum factor of safety of 1.5 with respect to slope stability. The design of slopes on the Mill Pond Dam should also satisfy 302 CMR 10.14(9) of the Commonwealth of Massachusetts Regulations for Dam Safety.

3.1.7 Subgrade Preparation, Subgrade Protection, Placement of Fill Materials, and Compaction

3.1.7.1 Subgrade Preparation

The location of the proposed structure and the temporary bridge should be cleared of the vegetation, with root systems grubbed, the topsoil stripped for reuse. Construction debris from demolished roadway or structures should be removed and properly disposed of, and any existing GRANULAR FILL (Stratum 2) and TIMBER CONSTRUCTION DEBRIS (Stratum 3) underlying the footprint of the proposed foundation locations should be excavated and replaced (See Section 3.1). After rough grades have been established, and before the placement of compacted fill materials, exposed surfaces should be visually inspected and probed. Frozen, wet, or loose soils and other undesirable materials should be removed. The exposed subgrade should be further tested by proof rolling with a minimum of 10 passes of a 10,000-pound static weight roller to identify loose or soft pockets that may be present.

In areas to be cut, the cut should extend to 12 inches below the bottom of foundations and be backfilled with a 12-inch layer of compacted "Gravel Borrow for Bridge Foundations".

3.1.7.2 Subgrade Protection

Subgrade soils exposed during foundation excavation should be disturbed as little as possible. The final 12 inches of excavation should be completed with a smooth-edge excavator bucket to reduce disturbance to the subgrade. Care should be taken to limit proof rolling efforts, as needed, in saturated soils as these soils are more vulnerable to disturbance in comparison to soils in their non-saturated state. The natural site soils are susceptible to disturbance by exposure to the elements (i.e., water, freezing, and drying) and by inadequate construction practices. Extended exposure of the natural backfill to the elements should be avoided. Should the subgrade become disturbed, the disturbed material should be over excavated and replaced with compacted "Gravel Borrow". ***To reduce the chance of creating an uncontrolled flow path through the dam, the use of crushed stone in place of "Gravel Borrow" Fill should NOT be permitted.*** The extent of preparation and compaction of the excavation bottoms should be at the discretion of a qualified geotechnical engineer in the field at the time of excavation.

3.1.7.3 Placement of Fill Materials

Imported Materials – Fill material should consist of hard, durable, sand and gravel should be free from ice and snow, roots, and other deleterious or organic matter. Imported materials expected to be used for site civil work include:

- "Gravel Borrow" for use below foundations and slabs, regrading slopes, and fill behind walls should conform to Item M1.03.0 Type a of the MHD Standard Specifications.



- Gravel Borrow utilized as “Structural Fill” directly below proposed foundations and slabs should conform to M1.03.0 Type b of the MHD Standard Specifications.

Onsite Materials – Based on the visual classifications and the laboratory testing results, the GRANULAR FILL (Stratum 2) will not meet the gradation requirements for reuse as “Gravel Borrow” or “Structural Fill” for backfill against below-grade walls, or beneath foundations and slabs. The fine-grained nature of the soils will make compaction of the material difficult. Achieving the required compaction will be difficult if the material is wet or dry of the optimum moisture content.

Compaction – Recommended minimum compaction requirements for fill are outlined in Table 3-4.

TABLE 3-4: RECOMMENDED MINIMUM COMPACTION REQUIREMENTS

Location	Percent of Maximum Dry Density ¹
Backfill below foundations and below approach slabs	98 ²
Backfill behind abutments and retaining walls	98 ^{2,3}
Backfill in front of abutments and retaining walls	98 ^{2,3}
Backfill within pavement base and sub base layers	98 ²
Backfill below pavement sub base layers	98 ²
Backfill within landscaped areas	98 ²

¹ Maximum dry density as determined by the Modified Proctor test (ASTM D 1557).

² Since the proposed structure is founded within a dam, it is recommended to establish a tight soil matrix directly below substructures as recommended by the MADCR Office of Dam Safety Program.

³ During compaction of fill placed behind retaining walls, care shall be taken so as to maintain uniform elevation along both sides within the embedded areas, and to not overstress the wall by applying too much compactive energy at the top of the wall.

3.1.8 Scour Protection

As paraphrased from the Hydraulic Study Report for this project (ref. Pare Feb. 2016):

Typical contraction scour and local abutment scour at the bridge are not anticipated given that the design of the proposed bridge includes a continuous concrete mat foundation from the downstream face of the spillway to the downstream end the bridge. During the final design, incorporating steps at the downstream side of the spillway wall will be evaluated to dissipate energy during low flow conditions. This will prevent deterioration of the concrete mat slab and issues at the joint between the mat slab and spillway resulting in a longer structure life and fewer maintenance issues and costs.

4.0 RECOMMENDED COFFERDAM AND CUT-OFF WALL SYSTEM

4.1 Basis for vertical walls Recommendations

The application of vertical walls beneath grade are required to reduce hydraulic gradient, to hold back pond water and stabilize the slope during construction. The subsurface exploration indicated wooden debris and boulders within dense soils below the immediate surficial pond sediments. Construction methods with rammed or vibrated installation may be unsuitable or require precautions leading to delays or unanticipated costs. The recommendations herein are designed to minimize risks, while not being the sole solution to obtain desirable conditions.

The means and method should address the following functions.

- Viable installation passing through construction debris and boulders with small positioning tolerances.
- Watertight seal to minimize seepage into excavation or through embankment and foundation.
- Temporary structural capacity to withstand forces during excavation and drawdown of pond water during construction of bridge and spillway.

4.2 Recommended installation method – Secant Walls

4.2.1 Methodology

The installation of reinforced secant wall is recommended for this project as temporary cofferdam as well as permanent cut-off wall. Secant walls consist of primary unreinforced piles and reinforced secondary piles built overlapping with the primary piles. Secant walls are being built using a stencil from the surface being drilled with a retractable casing. Installation is commonly supported by slurries and concrete casting by tremie method beneath ground water level.

4.2.2 Seepage Control

The secant wall is recommended to extend into the less permeable soil strata of GLACIAL DEPOSITS, to reduce seepage and risk of piping. Secant walls are generally being built below grade and not recommended for above ground applications, therefore not suitable for cofferdams to retain water within the impoundment. It is recommended to convert the retaining wall to a soldier pile wall within the impoundment beyond the application of the above-described secant wall. The pile diameter is designed to support soldier piles with lagging above the mudline for the proposed water level. Soldier pile walls and secant walls are compatible for transitions by application of steel H-piles as reinforcements that are continuous through both systems and can be selected for lagging spacing. Precautions to reduce leakage through exposed lagging or transitions of the systems will be required and should be reviewed by an engineer registered in the state of Massachusetts.

4.2.3 Structural Elements

The permanent cut-off wall will temporarily act as a retaining wall for the construction of the proposed bridge and spillway. The direction of active pressure will switch sides during the different stages of construction and be significantly reduced in the final design. The retained structure and traffic loads result in significant active earth pressure. The reinforcement in the secondary piles should be designed accordingly. Earth anchors are recommended to reduce the proposed wall section thickness. Anchors will require wailers to distribute the forces over multiple piles, since the unreinforced primary piles reduce the



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capacity of the wall to distribute stresses along the wall. The anchors and wailers will be abandoned or deconstructed after completion of the bridge and spillway.

The soldier pile wall is anticipated to be constructed as a cantilever system without braces, compression ring or anchor, due to space restrictions within the excavation or reach to supporting soils.



4.0 CONSTRUCTION CONSIDERATIONS

4.1 Control of Water

As discussed in Section 2.3.10, groundwater was encountered near the excavation depths likely for this project. Temporary dewatering is anticipated for this project. The impoundment should be drawdown, and/or a cofferdam/portable dam system should be installed throughout the construction duration to reduce and to better-control the hydrostatic head pressure through the dam. The proposed bridge and other improvements are located on an active dam; therefore, dewatering methods will require specialized methods beyond a typical bridge foundation dewatering program. The Contractor should be required to submit a dewatering plan for review prior to undertaking excavation work. The dewatering program should be designed by a Massachusetts-registered professional engineer experienced in dam construction and repair.

Crushed stone should not be used to stabilize excavation bottoms as this material creates a higher permeability pathway for seepage (which could give rise to piping) and would also be a weaker zone within the dam.

4.2 Recommended Method for By-Passing Flows During Construction

It is recommended to keep the existing fish ladder and primary spillway in operation to maintain flows through the dam while the existing auxiliary spillway is demolished and removed and until the proposed spillway, fish ladder, and bridge structures are completed and in operation. The Contractor should be required to submit a by-pass plan for review prior to undertaking excavation work.

4.3 Excavation Support

During the replacement of the existing spillway, demolition of the auxiliary spillway, decommissioning, and replacement of the fish ladder, temporary earth support will be required at the phase lines during the different stages of construction to support the roadway and water bodies adjacent to the construction area. In areas where an open cut is possible without a temporary support system, the final slopes should conform to all local, state, and federal safety requirements. Once the final design of the substructure is determined, these assumptions should be confirmed.

During the installation of the temporary earth support systems, existing structures to remain within the vicinity of operations should be stabilized and/or underpinned and monitored for settlement.

Due to the presence of the saturated upper layers of the GLACIAL DEPOSITS stratum, Pare recommends that driven temporary earth support systems remain in place after the structures are built and cutoff no higher than 12 inches below finish grade. If the temporary earth support systems were to be removed, the vibrations caused during the removal could temporarily weaken the surrounding soils and possibly cause excessive settlement or lateral movement of the new structures. During the installation of the temporary earth support systems, existing structures/utilities to remain within 10 feet of driving should be stabilized and/or underpinned and monitored for settlement.

4.4 Protection of Adjacent Structures and Utilities

Prior to the start of construction and after construction has been completed, it is recommended that video and/or photo pre- and post-construction surveys be performed throughout the project area and its perimeter.

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These surveys would help record before- and after-construction-conditions of existing structures and utilities that are expected to remain. These records are helpful in resolving potential project claim disputes.

Demolition of the existing culverts, spillway, auxiliary spillway, and fish ladder, site preparation, excavation and backfill, compaction, dewatering, and foundation installation should be monitored and observed by geotechnical field staff experienced in foundation and dam engineering and construction. This would help provide an accurate record of construction, alert the designer to changed conditions, ensure the design assumptions are met, and provide useful data for future construction.

In the event where a crack (or cracks) develop in new foundation walls, it is recommended that crack gauges be installed to monitor movement to help identify and address changed conditions during preliminary stages of construction.

4.5 Sequence of Construction Activities – Staged Construction

Excavation sequencing for the demolition and construction of the herein referenced structures should be coordinated internally for structures within the project area, and externally for third-party structures that are adjacent to the project area. This is recommended so that excavation stability concerns can be identified and addressed before construction.

4.6 Special Geotechnical Monitoring and Instrumentation

Foundation excavations should be inspected to confirm that all loose, soft, and undesirable material is removed and that the structure will bear on a satisfactory material. Excavation subgrade inspections should be made visually and include hand auger borings or hand probing.

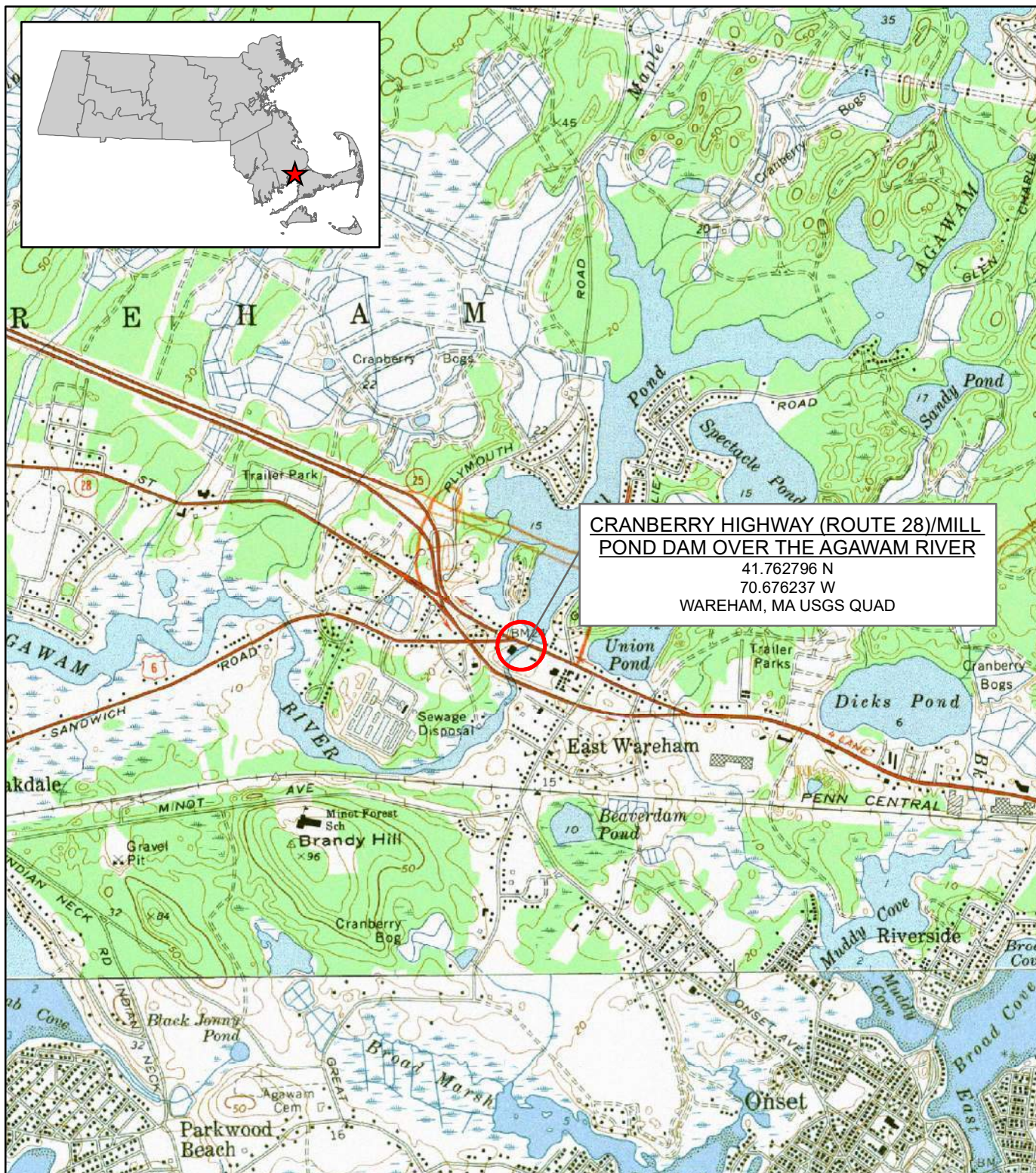
Compaction criteria for the various imported materials should be developed and included in the specifications. Field density testing should be performed by the contractor (Quality Control) and engineer (Quality Assurance), using a nuclear density gauge to confirm that adequate compaction is being achieved. During construction, representative samples of all materials to be used as backfill should also be tested for conformance with the specified material properties.

Cranberry Highway/Mill Pond Dam over the Agawam River

FIGURES

Cranberry Highway (Route 6/28) over Mill Pond Dam





LOCUS MAP

SCALE:1"=2,000'



8 BLACKSTONE VALLEY PLACE
 LINCOLN, RI 02865
 (401) 334-4100

10 LINCOLN ROAD, SUITE 210
 FOXBORO, MA 02035
 (508) 543-1755

PARE PROJECT NO.14199.14

AUGUST 2018

OWNER

MASSACHUSETTS DEPARTMENT OF TRANSPORTATION

FIGURE 1

MILL POND (ROUTE 28) DAM
 MA00027 / 7-12-310-3
 WAREHAM, MASSACHUSETTS



AERIAL PLAN

SCALE: 1"=500'



8 BLACKSTONE VALLEY PLACE
LINCOLN, RI 02865
(401) 334-4100

10 LINCOLN ROAD, SUITE 210
FOXBORO, MA 02035
(508) 543-1755

PARE PROJECT NO.14199.14

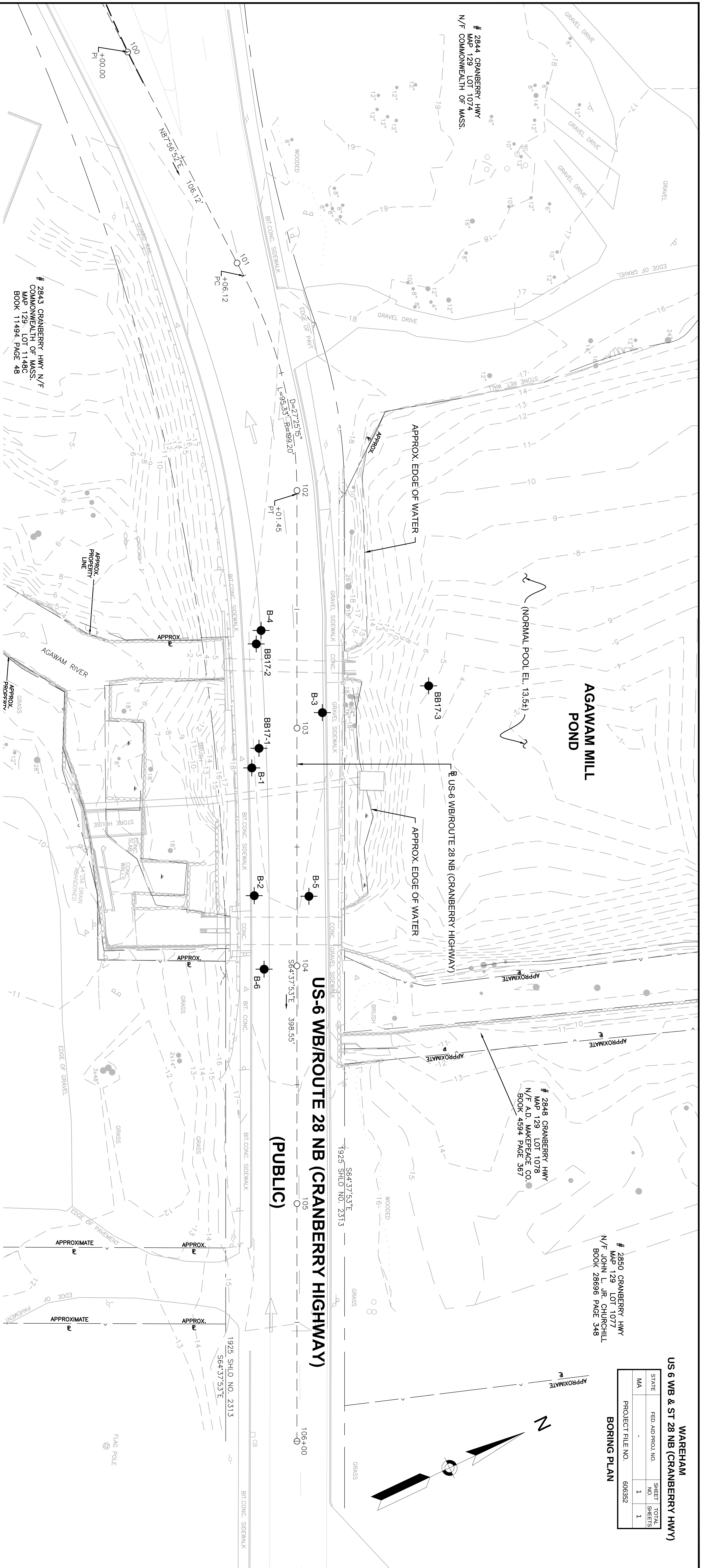
AUGUST 2018

OWNER

FIGURE 2

MILL POND (ROUTE 28) DAM
MA00027 / 7-12-310-3
WAREHAM, MASSACHUSETTS

MASSACHUSETTS DEPARTMENT OF TRANSPORTATION



BORING LOCATIONS (FEET)							
BORING NO.	NORTHING	EASTING	STATION	OFFSET	SURFACE EL	DEPTH	YEAR
B-1	2739594.0	880949.2	103+17	19' RT	17.9±	27'	2008
B-2	2739571.9	880998.2	103+70	18' RT	17.7±	27'	2008
B-3	2739630.8	880940.9	102+93	11' LT	18.2±	27'	2008
B-4	2739622.3	880898.7	102+59	15' RT	18.1±	32'	2010
B-5	2739592.6	881008.3	103+71	5' LT	17.9±	32'	2010
B-6	2739562.4	881028.0	104+01	14' RT	17.5±	32'	2010
BB17-1	2739600.3	880943.1	103+08	16' RT	18.1±	85' AND 10' BEDROCK CORE	2017
BB17-2	2739618.0	880902.9	102+65	17' RT	18.1±	84' AND 10' BEDROCK CORE	2017
BB17-3	2739676.0	880949.9	102+82	55' LT	5.1±	53'	2017

NOTES:

1. LOCATION OF DRIVE SAMPLE BORINGS ARE SHOWN THUS: 
2. SEE THE BORING LOCATIONS TABLE FOR THE SPECIFIED DEPTH OF EACH BORING.
3. BENCH MARK: GPS PNT #1, N2740804.864, E879795.245, ELEV.=26.91
4. BORINGS ARE LOCATED FROM THE US-6 WB/ROUTE 28 NB BASELINE.

BORINGS COMPLETED BY NEW ENGLAND BORING CONTRACTORS. BORINGS WERE

2. SEE THE BORING LOCATIONS TABLE FOR THE SPECIFIED DEPTH OF EACH BORING.

3. BENCH MARK: GPS PNT #1, N2740804.864, E879795.245, ELEV.=26.91

4. BORINGS ARE LOCATED FROM THE US-6 WB/ROUTE 28 NB BASELINE.

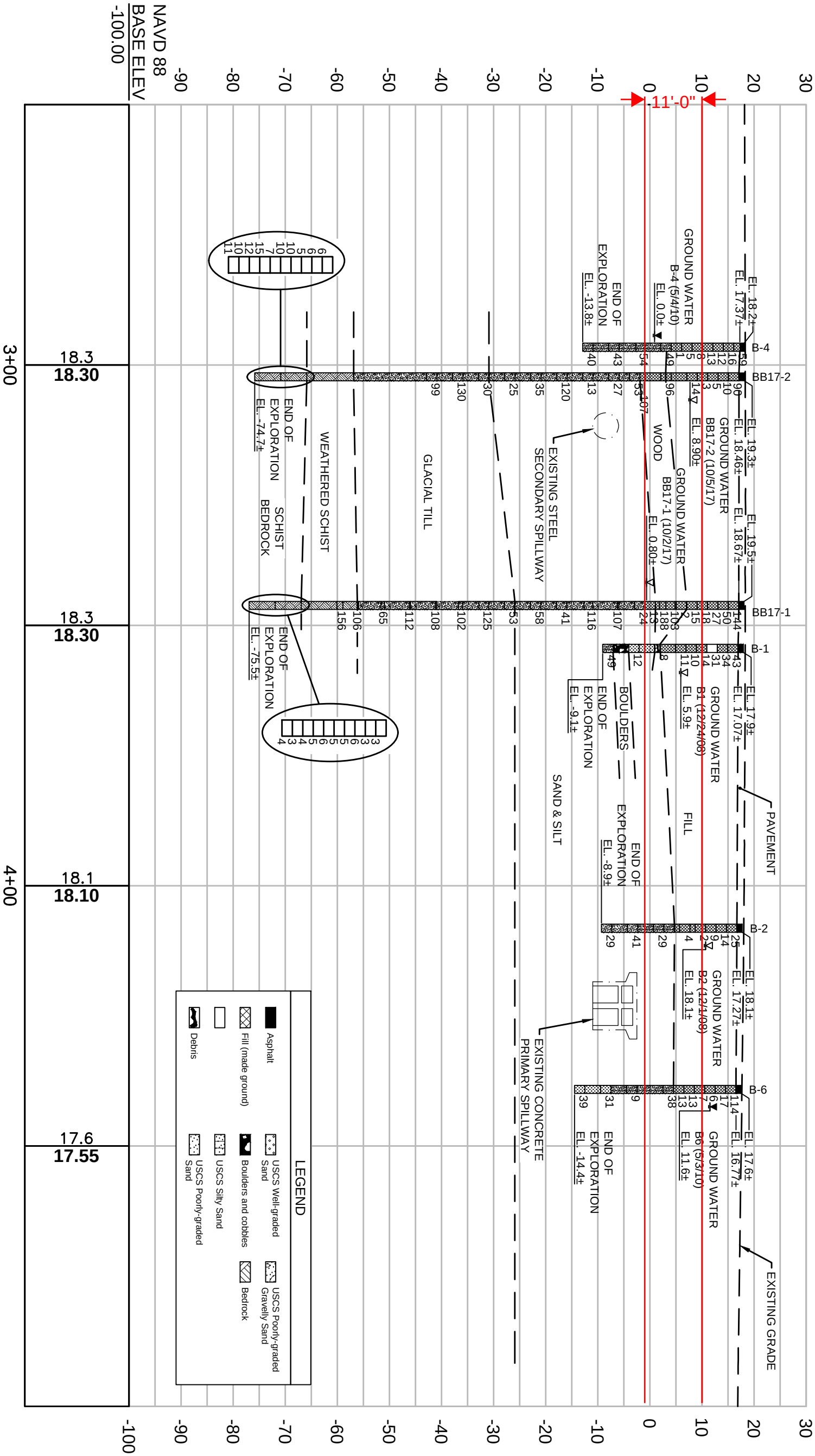
US 6 WB/ST 28 NB (CRANBERRY HWY)

OVER AGAWAM RIVER

MASSACHUSETTS DEPARTMENT OF TRANSPORTATION
HIGHWAY DIVISION

SCALE: 1" = 20'-0" SEPTEMBER 20, 2018

BRIDGE NO. W-06-020 (8M1)



Cranberry Highway/Mill Pond Dam over the Agawam River

APPENDIX A

Boring Logs

Cranberry Highway/Mill Pond Dam over the Agawam River





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Foxboro, MA 02035
T: 508-543-1755
F: 508-543-1881

BORING NUMBER B-1

PAGE 1 OF 1

CLIENT MassDOT

PROJECT NAME Mill Pond Dam

PROJECT NUMBER 14199.14

PROJECT LOCATION Wareham, MA

DATE STARTED 11/24/08 COMPLETED 11/24/08

GROUND ELEVATION 17.9 ft HOLE SIZE 4 inches

DRILLING CONTRACTOR New Hampshire Boring, Inc.

GROUND WATER LEVELS:

DRILLING METHOD Wash and Drive

▽ AT TIME OF DRILLING 12.00 ft / Elev 5.90 ft

LOGGED BY Varinia Arauco Vera CHECKED BY

AT END OF DRILLING ---

BORING LOCATION 2739594.01°N / 880949.2°E

PARE BORING LOG - GINT STD US LAB GDT - 8/16/18 15:21 - Y:\JOBS\14 JOBS\14199.14 MASSDOT MILL POND DAMBORING LOG\14199.14 BORING LOGS.GPJ

DEPTH (ft)	CASING (bl/ft)	SAMPLE TYPE NUMBER	RECOVERY/PEN. (in)	DEPTH (FT)	BLOW COUNTS/6"	MIN/FT	GRAPHIC LOG	SAMPLE DESCRIPTION	STRATUM DESCRIPTION
0									
5		S-1	24 / 24	1 - 3	34-26-17-12 (43)			Dry, dense, black/brown, fine to coarse SAND, some fine gravel, trace ash, trace inorganic silt.	ASPHALT GRAVEL ROADBASE 5'
		S-2	12 / 24	3 - 5	11-19-15-11 (34)			Dry, dense, brown, coarse to fine SAND, trace fine gravel, trace inorganic silt.	
			0 / 24	5 - 7	18-17-14-13 (31)			No Recovery.	
10		S-3	10 / 24	7 - 9	10-9-5-6 (14)			Dry, medium dense, brown/tan, fine to coarse SAND, trace inorganic silt, trace fine gravel, trace ash.	FILL
		S-4	20 / 24	9 - 11	11-5-5-5 (10)			Dry, loose, black, fine to coarse SAND, some fine gravel, trace ash.	
		S-5	22 / 24	11 - 13	8-6-5-4 (11)			Wet, medium dense, tan, fine to coarse SAND, trace inorganic silt.	
15									
		S-6	10 / 24	15 - 17	9-4-4-2 (8)			Wet, loose, gray, fine to coarse SAND, some wood, trace inorganic silt.	SOME WOOD
20									
		S-7	12 / 24	20 - 22	6-5-7-7 (12)			Wet, medium dense, gray, fine to coarse SAND, trace inorganic silt.	SAND WITH SILT
								Boulder 23'5" to 24'9".	BOULDER
25									
		S-8	15 / 24	25 - 27	20-23-26-25 (49)			Wet, dense, tan, fine SAND and inorganic SILT, trace fine gravel.	
30								Bottom of borehole at 27.0 feet.	
35									

GRANULAR SOILS		COHESIVE SOILS		REMARKS:	BURMISTER CLASSIFICATION
BLOWS/FT	DENSITY	BLOWS/FT	DENSITY		
0 - 4	V. LOOSE	<2	V. SOFT		TRACE 0 - 10%
4 - 10	LOOSE	2 - 4	SOFT		LITTLE 10 - 20%
10 - 30	M. DENSE	4 - 8	M. STIFF		SOME 20 - 35%
30 - 50	DENSE	8 - 15	STIFF		AND 35 - 50%
>50	V. DENSE	15 - 30	V. STIFF		PERCENT BY WEIGHT
		>30	HARD		

NOTES: 1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES, TRANSITIONS MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.



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T: 508-543-1755
F: 508-543-1881

BORING NUMBER B-2

PAGE 1 OF 1

CLIENT MassDOTPROJECT NAME Mill Pond DamPROJECT NUMBER 14199.14PROJECT LOCATION Wareham, MADATE STARTED 12/1/08 COMPLETED 12/1/08GROUND ELEVATION 18.1 ft HOLE SIZE 4 inchesDRILLING CONTRACTOR New Hampshire Boring, Inc.

GROUND WATER LEVELS:

DRILLING METHOD Wash and Drive▽ AT TIME OF DRILLING 7.00 ft / Elev 11.10 ftLOGGED BY Varinia Arauco Vera CHECKED BY _____AT END OF DRILLING ---BORING LOCATION 2739571.92°N / 880998.23°E

PARE BORING LOG - GINT STD US LAB GDT - 8/16/18 15:21 - Y:\JOBS\14 JOBS\14199.14 MASSDOT MILL POND DAMBORING LOG\14199.14 BORING LOGS.GPJ

DEPTH (ft)	CASING (bl/ft)	SAMPLE TYPE NUMBER	RECOVERY/PEN. (in)	DEPTH (FT)	BLOW COUNTS/6"	MIN/FT	GRAPHIC LOG	SAMPLE DESCRIPTION	STRATUM DESCRIPTION
0									
		S-1	12 / 24	1 - 3	15-13-12-7 (25)			Dry, medium dense, brown, fine to coarse SAND, some fine gravel, trace inorganic silt.	ASPHALT Gravel ROADBASE 3"
5		S-2	10 / 24	3 - 5	7-8-6-6 (14)			Moist, medium dense, tan, fine to coarse SAND, some fine gravel, some inorganic silt.	FILL
		S-3	10 / 24	5 - 7	7-5-4-2 (9)			Wet, loose, tan, fine to coarse SAND, some fine gravel, trace inorganic silt.	
		S-4	8 / 24	7 - 9	2-1-1-2 (2)			Wet, very loose, tan, fine to coarse SAND, some inorganic silt, trace fine gravel. Switched to 4" casing 9".	
10									
		S-5	8 / 24	10 - 12	4-2-2-5 (4)			Wet, very loose, tan, fine to coarse SAND, trace fine gravel, trace inorganic silt.	
15									SAND WITH SILT
		S-6	20 / 24	15 - 17	13-15-14-21 (29)			Wet, medium dense, tan, fine SAND, some inorganic silt.	
20									
		S-7	10 / 24	20 - 22	23-21-20-17 (41)			Wet, dense, tan, fine to coarse SAND, some inorganic silt, trace fine gravel.	
25									
		S-8	10 / 24	25 - 27	15-16-13-17 (29)			Wet, medium dense, tan, fine to coarse SAND, some inorganic silt, some fine gravel.	
								Bottom of borehole at 27.0 feet.	
30									
35									

GRANULAR SOILS		COHESIVE SOILS		REMARKS:	BURMISTER CLASSIFICATION
BLOWS/FT	DENSITY	BLOWS/FT	DENSITY		
0 - 4	V. LOOSE	<2	V. SOFT		TRACE 0 - 10%
4 - 10	LOOSE	2 - 4	SOFT		LITTLE 10 - 20%
10 - 30	M. DENSE	4 - 8	M. STIFF		SOME 20 - 35%
30 - 50	DENSE	8 - 15	STIFF		AND 35 - 50%
>50	V. DENSE	15 - 30	V. STIFF		PERCENT BY WEIGHT
		>30	HARD		

NOTES: 1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES, TRANSITIONS MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

BORING NUMBER B-3

PAGE 1 OF 1



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10 Lincoln Road, Suite 210
Foxboro, MA 02035
T: 508-543-1755
F: 508-543-1881

CLIENT MassDOTPROJECT NAME Mill Pond DamPROJECT NUMBER 14199.14PROJECT LOCATION Wareham, MADATE STARTED 12/1/08 COMPLETED 12/1/08GROUND ELEVATION 18.2 ft HOLE SIZE 4 inchesDRILLING CONTRACTOR New Hampshire Boring, Inc.

GROUND WATER LEVELS:

DRILLING METHOD Wash and DriveAT TIME OF DRILLING ---LOGGED BY Varinia Arauco Vera CHECKED BY ---AT END OF DRILLING 8.00 ft / Elev 10.20 ftBORING LOCATION 2739630.81°N / 880940.98°E

PARE BORING LOG - GINT STD US LAB GDT - 8/16/18 15:21 - Y:\JOBS\14 JOBS\14199.14 MASSDOT MILL POND DAMBORING LOG\14199.14 BORING LOGS.GPJ

DEPTH (ft)	CASING (bl/ft)	SAMPLE TYPE NUMBER	RECOVERY/PEN. (in)	DEPTH (FT)	BLOW COUNTS/6"	MIN/FT	GRAPHIC LOG	SAMPLE DESCRIPTION	STRATUM DESCRIPTION
0									
		S-1	20 / 24	1 - 3	41-37-25-25 (62)			Dry, very dense, black, fine to coarse SAND, some inorganic silt, trace fine gravel.	ASPHALT Gravel ROADBASE 3"
5		S-2	20 / 18	3 - 5	19-23-24- 16/0"			Dry, dense, tan/gray, fine to coarse SAND, some inorganic silt, trace fine gravel.	
		S-3	22 / 24	5 - 7	12-12-14-13 (26)			Moist, medium dense, tan, fine SAND, some inorganic silt, trace fine gravel.	
		S-4	24 / 24	7 - 9	12-10-7-8 (17)			Moist, medium dense, tan, fine SAND, trace inorganic silt.	
10		S-5	24 / 24	9 - 11	4-5-7-10 (12)			Wet, medium dense, gray, fine to coarse SAND, some inorganic silt.	FILL
								Wet, medium dense, black WOOD and organic MATERIAL (leaves and roots).	
								Wet, medium dense, gray, fine SAND, trace inorganic silt, trace wood.	
15		S-6	16 / 24	15 - 17	1-1-1-1 (2)			Wet, very loose, tan, fine SAND, trace inorganic silt.	
								Wet, very loose, black, fine SAND, trace organic silt.	
20		S-7	10 / 24	20 - 22	11-12-23-17 (35)			Wet, dense, gray, fine SAND and inorganic SILT lenses, some coarse sand, trace cobbles.	SAND WITH SILT
25		S-8	12 / 24	25 - 27	23-23-34-55 (57)			Wet, very dense, tan, fine to coarse SAND, trace inorganic silt, trace cobbles.	
30									
35									

Bottom of borehole at 27.0 feet.

GRANULAR SOILS		COHESIVE SOILS		REMARKS:	BURMISTER CLASSIFICATION
BLOWS/FT	DENSITY	BLOWS/FT	DENSITY		
0 - 4	V. LOOSE	<2	V. SOFT		TRACE 0 -10%
4 - 10	LOOSE	2 - 4	SOFT		LITTLE 10 - 20%
10 - 30	M. DENSE	4 - 8	M. STIFF		SOME 20 - 35%
30 - 50	DENSE	8 - 15	STIFF		AND 35 - 50%
>50	V. DENSE	15 - 30	V. STIFF		PERCENT BY WEIGHT
		>30	HARD		

NOTES: 1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES, TRANSITIONS MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

BORING NUMBER B-4

PAGE 1 OF 1



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10 Lincoln Road, Suite 210
Foxboro, MA 02035
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F: 508-543-1881

CLIENT MassDOT

PROJECT NAME Mill Pond Dam

PROJECT NUMBER 14199.14

PROJECT LOCATION Wareham, MA

DATE STARTED 5/4/10 COMPLETED 5/5/10

GROUND ELEVATION 18.2 ft HOLE SIZE inches

DRILLING CONTRACTOR New Hampshire Boring, Inc.

GROUND WATER LEVELS:

DRILLING METHOD Wash and Drive/Automatic Hammer

AT TIME OF DRILLING ---

LOGGED BY Shane Souza

CHECKED BY

AT END OF DRILLING 18.20 ft / Elev 0.00 ft

BORING LOCATION 2739622.3°N / 880898.7°E

PARE BORING LOG - GINT STD US LAB GDT - 8/16/18 15:21 - Y:\JOBS\14 JOBS\14199.14 MASSDOT MILL POND DAMBORING LOG\14199.14 BORING LOGS.GPJ

DEPTH (ft)	CASING (bl/ft)	SAMPLE TYPE NUMBER	RECOVERY/PEN. (in)	DEPTH (FT)	BLOW COUNTS/6"	MIN/FT	GRAPHIC LOG	SAMPLE DESCRIPTION	STRATUM DESCRIPTION
0									
5		S-1	9 / 24	1 - 3	51-39-20-12 (59)			Dry, very dense, brown, fine SAND, some coarse sand, trace inorganic silt.	FILL
		S-2	14 / 24	3 - 5	7-8-8-6 (16)			Dry, medium dense, brown, fine SAND, trace coarse sand, trace inorganic silt.	
		S-3	6 / 24	5 - 7	8-6-6-7 (12)			Dry, medium dense, brown, fine SAND, trace coarse sand, trace inorganic silt.	
		S-4	7 / 24	7 - 9	6-7-6-6 (13)			Moist, medium dense, brown, fine SAND, trace coarse sand, trace inorganic silt.	
10		S-5	6 / 24	9 - 11	7-5-3-1 (8)			Wet, loose, brown, fine SAND, some coarse sand, trace inorganic silt.	
		S-6	10 / 24	11 - 13	2-3-2-2 (5)			Wet, loose, brown, coarse SAND and fine SAND, trace inorganic silt.	
15		S-7	12 / 24	13 - 15	3-1-0-1 (1)			Wet, very loose, black, coarse SAND and fine SAND, trace coarse sand.	
		S-8	10 / 24	15 - 17	8-16-33-27 (49)			Wet, hard, light brown, inorganic SILT and fine SAND, trace coarse sand.	SAND WITH SILT
20									
		S-9	16 / 22	20 - 21.8	11-22-32-120/4"			Wet, very dense, light brown, fine SAND and inorganic SILT, trace coarse sand.	
25									
		S-10	15 / 24	25 - 27	19-21-22-28 (43)			Wet, dense, light brown, fine SAND, some inorganic silt, trace coarse sand.	
30									
		S-11	3 / 24	30 - 32	10-14-26-33 (40)			Wet, dense, light brown, fine SAND, some inorganic silt, trace coarse sand.	
35								Bottom of borehole at 32.0 feet.	

GRANULAR SOILS		COHESIVE SOILS		REMARKS:	BURMISTER CLASSIFICATION
BLOWS/FT	DENSITY	BLOWS/FT	DENSITY		
0 - 4	V. LOOSE	<2	V. SOFT	1.Changed location: boring moved from N2739619.6, E880897.4 to N2739622.3, E880898.7 because of gas line.	TRACE 0 -10%
4 -10	LOOSE	2 - 4	SOFT		LITTLE 10 - 20%
10 - 30	M. DENSE	4 - 8	M. STIFF		SOME 20 - 35%
30 - 50	DENSE	8 - 15	STIFF		AND 35 - 50%
>50	V. DENSE	15 - 30	V. STIFF		PERCENT BY WEIGHT
		>30	HARD		

NOTES: 1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES, TRANSITIONS MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

A00604-37

BORING NUMBER B-5

PAGE 1 OF 1



Pare Corporation
10 Lincoln Road, Suite 210
Foxboro, MA 02035
T: 508-543-1755
F: 508-543-1881

CLIENT MassDOTPROJECT NAME Mill Pond DamPROJECT NUMBER 14199.14PROJECT LOCATION Wareham, MADATE STARTED 5/4/10 COMPLETED 5/5/10GROUND ELEVATION 17.9 ft HOLE SIZE inchesDRILLING CONTRACTOR New Hampshire Boring, Inc.

GROUND WATER LEVELS:

DRILLING METHOD Wash and Drive/Automatic Hammer▽ AT TIME OF DRILLING 9.00 ft / Elev 8.90 ftLOGGED BY Shane Souza

CHECKED BY _____

AT END OF DRILLING ---BORING LOCATION 2739592.6°N / 881008.3°E

PARE BORING LOG - GINT STD US LAB GDT - 8/16/18 15:21 - Y:\JOBS\14 JOBS\14199.14 MASSDOT MILL POND DAMBORING LOG\14199.14 BORING LOGS.GPJ

DEPTH (ft)	CASING (bl/ft)	SAMPLE TYPE NUMBER	RECOVERY/PEN. (in)	DEPTH (FT)	BLOW COUNTS/6"	MIN/FT	GRAPHIC LOG	SAMPLE DESCRIPTION	STRATUM DESCRIPTION
0									
		S-1	4 / 4	1 - 1.3	120/4"			Dry, very dense, black, coarse SAND, some fine sand.	6" ASPHALT
5		S-2	20 / 24	3 - 5	8-13-10-8 (23)			Dry, medium dense, black, fine SAND, trace inorganic silt.	FILL
		S-3	16 / 24	5 - 7	6-6-5-5 (11)			Wet, medium dense, brown, fine SAND, trace coarse sand.	
		S-4	24 / 24	7 - 9	4-2-2-2 (4)			Wet, loose, brown, fine SAND.	
10		S-5	6 / 24	9 - 11	5-5-4-4 (9)			Wet, loose, brown, fine SAND, trace coarse sand.	
		S-6	12 / 24	11 - 13	10-11-11-12 (22)			Wet, medium dense, brown, fine SAND.	SAND WITH SILT
15		S-7	14 / 24	13 - 15	11-11-10-9 (21)			Wet, medium dense, dark brown, fine SAND, some coarse sand.	
20									
		S-8	12 / 24	20 - 22	15-18-17-16 (35)			Wet, dense, light brown, fine SAND, some coarse sand, trace inorganic silt.	
25									
		S-9	15 / 24	25 - 27	14-16-20-14 (36)			Wet, dense, light brown, fine SAND, some coarse sand, trace inorganic silt.	SAND WITH SILT
30									
		S-10	14 / 24	30 - 32	13-14-16-17 (30)			Wet, dense, brown, fine SAND, some coarse sand.	
Bottom of borehole at 32.0 feet.									
35									

GRANULAR SOILS		COHESIVE SOILS		REMARKS:	BURMISTER CLASSIFICATION
BLOWS/FT	DENSITY	BLOWS/FT	DENSITY		
0 - 4	V. LOOSE	<2	V. SOFT		TRACE 0 - 10%
4 - 10	LOOSE	2 - 4	SOFT		LITTLE 10 - 20%
10 - 30	M. DENSE	4 - 8	M. STIFF		SOME 20 - 35%
30 - 50	DENSE	8 - 15	STIFF		AND 35 - 50%
>50	V. DENSE	15 - 30	V. STIFF		PERCENT BY WEIGHT
		>30	HARD		

NOTES: 1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES, TRANSITIONS MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

BORING NUMBER B-6

PAGE 1 OF 1



Pare Corporation
10 Lincoln Road, Suite 210
Foxboro, MA 02035
T: 508-543-1755
F: 508-543-1881

CLIENT MassDOTPROJECT NAME Mill Pond DamPROJECT NUMBER 14199.14PROJECT LOCATION Wareham, MADATE STARTED 5/3/10 COMPLETED 5/3/10GROUND ELEVATION 17.6 ft HOLE SIZE inchesDRILLING CONTRACTOR New Hampshire Boring, Inc.

GROUND WATER LEVELS:

DRILLING METHOD Wash and Drive/Automatic HammerAT TIME OF DRILLING ---LOGGED BY Shane SouzaCHECKED BY ▼ AT END OF DRILLING 6.00 ft / Elev 11.60 ftBORING LOCATION 2739562.4°N / 881028.0°E

PARE BORING LOG - GINT STD US LAB GDT - 8/16/18 15:21 - Y:\JOBS\14 JOBS\14199.14 MASSDOT MILL POND DAMBORING LOG\14199.14 BORING LOGS.GPJ

DEPTH (ft)	CASING (bl/ft)	SAMPLE TYPE NUMBER	RECOVERY/PEN. (in)	DEPTH (FT)	BLOW COUNTS/6"	MIN/FT	GRAPHIC LOG	SAMPLE DESCRIPTION	STRATUM DESCRIPTION
0									
		S-1	12 / 24	1 - 3	74-85-29-17 (114)			Dry, very dense, black, fine SAND, trace coarse sand.	ASPHALT
5		S-2	8 / 24	3 - 5	11-9-8-7 (17)			Dry, medium dense, light brown, fine SAND, trace coarse sand.	FILL
		S-3	2 / 24	5 - 7	1-3-3-2 (6)			Moist, loose, light brown, fine SAND, some coarse sand.	
		S-4	7 / 24	7 - 9	2-2-5-12 (7)			Moist, loose, light brown, fine SAND, trace inorganic silt, trace coarse sand.	
10		S-5	6 / 24	9 - 11	5-7-6-5 (13)			Wet, medium dense, dark brown, fine SAND, some coarse sand, trace inorganic silt.	
		S-6	4 / 24	11 - 13	4-7-6-7 (13)			Wet, medium dense, black, fine SAND, some coarse sand, trace inorganic silt.	
15		S-7	8 / 24	13 - 15	7-16-22-37 (38)			Wet, dense, brown, fine SAND, some inorganic silt, trace coarse sand.	SAND WITH SILT
20		S-8	8 / 24	20 - 22	9-3-6-12 (9)			Wet, loose, light brown, fine SAND, some inorganic sil, trace coarse sand.	
25		S-9	9 / 24	25 - 27	37-15-16-20 (31)			Wet, dense, light brown, fine SAND, trace coarse sand.	
30		S-10	14 / 24	30 - 32	9-17-22-18 (39)			Wet, dense, light brown, coarse SAND, trace fine sand.	
35								Bottom of borehole at 32.0 feet.	

GRANULAR SOILS		COHESIVE SOILS		REMARKS:	BURMISTER CLASSIFICATION
BLOWS/FT	DENSITY	BLOWS/FT	DENSITY		
0 - 4	V. LOOSE	<2	V. SOFT		TRACE 0 - 10%
4 - 10	LOOSE	2 - 4	SOFT		LITTLE 10 - 20%
10 - 30	M. DENSE	4 - 8	M. STIFF		SOME 20 - 35%
30 - 50	DENSE	8 - 15	STIFF		AND 35 - 50%
>50	V. DENSE	15 - 30	V. STIFF		PERCENT BY WEIGHT
		>30	HARD		

NOTES: 1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES, TRANSITIONS MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

BORING NUMBER BB17-1

PAGE 1 OF 4



Pare Corporation
10 Lincoln Road, Suite 210
Foxboro, MA 02035
508-543-1755

CLIENT MassDOTPROJECT NAME Mill Pond DamPROJECT NUMBER 14199.14PROJECT LOCATION Wareham, MADATE STARTED 9/28/17 COMPLETED 10/2/17GROUND ELEVATION 19.5 ft NAVD88 HOLE SIZE 4 in.DRILLING CONTRACTOR New England Boring Contractors, Inc.

GROUND WATER LEVELS:

DRILLING METHOD Wash and Drive▽ AT TIME OF DRILLING 18.70 ft / Elev 0.80 ftLOGGED BY M. Pham CHECKED BY SJM

▼ AT END OF DRILLING

BORING LOCATION 2739600.3°N / 880943.07°E

PARE BORING LOG - GINT STD US LAB GDT - 12/7/21 16:10 - Y:\JOBS\14 JOBS\14199.14 MASSDOT MILL POND DAMBORING LOG\14199.14A BORING LOGS.GPJ

DEPTH (ft)	CASING (bl/ft)	SAMPLE TYPE NUMBER	RECOVERY/PEN. (in)	DEPTH (FT)	BLOW COUNTS/6"	MIN/FT	GRAPHIC LOG	SAMPLE DESCRIPTION	STRATUM DESCRIPTION
0									
		S-1	12 / 24	1 - 3	14-55-89-67 (144)			Wet, very dense, dark brown to black, fine to coarse GRAVEL and fine to coarse SAND, trace silt.	8-10" PAVEMENT
5		S-2	16 / 24	3 - 5	51-28-22-22 (50)			2A: Wet, dense/very dense, dark brown to black, fine to coarse SAND, some fine gravel, trace silt. 2B: Wet, very dense, light brown, fine to coarse SAND, trace fine gravel, trace silt.	
		S-3	6 / 24	5 - 7	11-16-11-10 (27)			Wet, medium dense, light brown to dark brown, fine to coarse SAND, some fine to coarse gravel, trace silt.	
		S-4	16 / 24	7 - 9	12-8-10-11 (18)			4A: Wet, medium dense, brown, fine to coarse SAND, some fine to coarse gravel. 4B: Wet, medium dense, light tan, fine to medium SAND, little fine gravel, trace silt.	FILL
10		S-5	6 / 24	9 - 11	13-9-6-5 (15)			Wet, medium dense, tan, (light to medium brown), medium SAND, trace coarse sand, trace silt. (FILL)	
		S-6	10 / 24	11 - 13	2-1-1-3 (2)			6A: Wet, loose, light brown, medium to coarse SAND, trace fine gravel, trace silt. 6B: Wet, loose, reddish to dark brown, fine to coarse SAND, some fine gravel, trace wood, trace silt.	
		S-7	0 / 3	13 - 13.3	103/3"			No recovery (Wood in tip)	
15		S-8	9 / 15	15 - 16.3	17-85-103/3"			Wet, very dense, light brown, WOOD.	WOOD
		S-9	7 / 24	17 - 19	1-3-10-10 (13)			Wet, medium dense, light brown, medium to coarse SAND, trace fine to coarse gravel, trace fine sand, trace wood, odor.	FILL
20		S-10	13 / 24	19 - 21	8-11-13-18 (24)			Wet, medium dense, light gray, medium SAND, trace coarse sand, trace fine gravel.	
									GLACIAL DEPOSITS
25		S-11	4 / 4	24 - 24.3	107/4"			Wet, very dense, light brown, SILT, trace coarse sand, trace fine gravel.	

GRANULAR SOILS		COHESIVE SOILS		REMARKS:	BURMISTER CLASSIFICATION
BLOWS/FT	DENSITY	BLOWS/FT	CONSIST.		
0 - 4	V. LOOSE	<2	V. SOFT	1.S-11: Slow drilling. Obstruction at 24.5' (1').	TRACE 0 - 10%
4 - 10	LOOSE	2 - 4	SOFT		LITTLE 10 - 20%
10 - 30	M. DENSE	4 - 8	M. STIFF		SOME 20 - 35%
30 - 50	DENSE	8 - 15	STIFF		AND 35 - 50%
>50	V. DENSE	15 - 30	V. STIFF		PERCENT BY WEIGHT
		>30	HARD		

NOTES: 1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES, TRANSITIONS MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

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Pare Corporation
10 Lincoln Road, Suite 210
Foxboro, MA 02035
508-543-1755

BORING NUMBER BB17-1

PAGE 2 OF 4

CLIENT MassDOTPROJECT NAME Mill Pond DamPROJECT NUMBER 14199.14PROJECT LOCATION Wareham, MA

DEPTH (ft)	CASING (bl/ft)	SAMPLE TYPE NUMBER	RECOVERY/PEN. (in)	DEPTH (FT)	BLOW COUNTS/6"	MIN/FT	GRAPHIC LOG	SAMPLE DESCRIPTION	STRATUM DESCRIPTION
25									
30		S-12	12 / 20	29 - 30.7	53-57-59-70/2"			Wet, very dense, light brown, fine SAND and SILT, some coarse sand, trace fine gravel.	
35		S-13	12 / 24	34 - 36	17-21-20-33 (41)			Wet, dense, light brown, fine SAND and SILT, some coarse sand, some fine to coarse gravel.	
40		S-14	10 / 24	39 - 41	19-23-35-36 (58)			Wet, very dense, light gray, fine to coarse GRAVEL and fine to coarse SAND, trace silt.	GLACIAL DEPOSITS
45		S-15	11 / 24	44 - 46	25-23-30-34 (53)			Wet, very dense, light gray, fine to coarse SAND and fine to coarse GRAVEL, trace silt, trace black/dark green fine sand.	
50		S-16	9 / 21	49 - 50.9	31-67-58-90/3"			Wet, very dense, light gray, fine to medium SAND, little coarse sand, trace coarse gravel, trace silt.	
55		S-17	7 / 10	54 - 54.8	56-102/4"			Wet, very dense, light brown, fine to coarse SAND and fine gravel, trace coarse sand, trace silt.	

2.S-12 & S-14: Rock in tip.

PARE BORING LOG - GINT STD US LAB GDT - 12/7/21 16:10 - Y:\JOBS\14 JOBS\14199.14 MASSDOT MILL POND DAM BORING LOG\14199.14A BORING LOGS.GPJ

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Pare Corporation
10 Lincoln Road, Suite 210
Foxboro, MA 02035
508-543-1755

BORING NUMBER BB17-1

PAGE 3 OF 4

CLIENT MassDOTPROJECT NAME Mill Pond DamPROJECT NUMBER 14199.14PROJECT LOCATION Wareham, MA

PARE BORING LOG - GINT STD US LAB GDT - 12/7/21 16:10 - Y:\JOBS\14 JOBS\14199.14 MASSDOT MILL POND DAMBORING LOG\14199.14A BORING LOGS.GPJ

DEPTH (ft)	CASING (bl/ft)	SAMPLE TYPE NUMBER	RECOVERY/PEN. (in)	DEPTH (FT)	BLOW COUNTS/6"	MIN/FT	GRAPHIC LOG	SAMPLE DESCRIPTION	STRATUM DESCRIPTION
60		S-18	0 / 3	59 - 59.3	108/3"			No recovery	GLACIAL DEPOSITS
65		S-19	2 / 3	64 - 64.3	112/3"			Wet, very dense, light brown, fine to coarse SAND, trace fine gravel, trace silt.	
70		S-20	5 / 8	69 - 69.7	115-65/2"			Wet, very dense, light brown SILT and fine to coarse SAND, trace fine gravel.	
75		S-21	0 / 3	74 - 74.3	106/3"			No Recovery.	
80		S-22	2 / 14	77 - 78.2	9-26-130/2"			Wet, very dense, gray, weathered SCHIST.	WEATHERED SCHIST
85								Fair, gray, fresh to slightly weathered, hard, moderately fractured SCHIST. REC = 96.7% RQD = 71.7%	
	C1		58	85		3 3 6			SCHIST

3.S-18: Slow drilling. Possible boulder (1') at 62'.
4.S-20: Boulder at 69.5'.
5.Slow drilling. Possible bedrock at 82'.



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BORING NUMBER BB17-1

PAGE 4 OF 4

CLIENT MassDOT PROJECT NAME Mill Pond Dam
PROJECT NUMBER 14199.14 PROJECT LOCATION Wareham, MA

DEPTH (ft)	CASING (bl/ft)	SAMPLE TYPE NUMBER	RECOVERY/PEN. (in)	DEPTH (FT)	BLOW COUNTS/6"	MIN/FT	GRAPHIC LOG	SAMPLE DESCRIPTION	STRATUM DESCRIPTION
90			7 / 60	- 90		5		Fair, gray, fresh to slightly weathered, hard, moderately fractured SCHIST. REC = 96.7% RQD = 71.7% (continued)	SCHIST
						5			
						6		Fair, orangy, fresh, hard, moderately fractured SCHIST/GRANITE. REC = 100% RQD = 65%	
						5			
						4			
		C2	60 / 60	90 - 95		4			GRANITE
						3			
95						4			

Bottom of borehole at 95.0 feet.

PARE BORING LOG - GINT STD US LAB GDT - 12/7/21 16:10 - Y:\JOBS\14 JOBS\14199.14 MASSDOT MILL POND DAM BORING LOG\14199.14A BORING LOGS.GPJ



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BORING NUMBER BB17-2

PAGE 1 OF 4

CLIENT MassDOTPROJECT NAME Mill Pond DamPROJECT NUMBER 14199.14PROJECT LOCATION Wareham, MADATE STARTED 10/3/17 COMPLETED 10/5/17GROUND ELEVATION 19.3 ft NAVD88 HOLE SIZE 4 in.DRILLING CONTRACTOR New England Boring Contractors, Inc.

GROUND WATER LEVELS:

DRILLING METHOD Wash and Drive▽ AT TIME OF DRILLING 10.40 ft / Elev 8.90 ftLOGGED BY M. Pham CHECKED BY SJMAT END OF DRILLING ---BORING LOCATION 2739618.04°N / 880902.9°E

PARE BORING LOG - GINT STD US LAB GDT - 12/7/21 16:10 - Y:\JOBS\14 JOBS\14199.14 MASSDOT MILL POND DAM BORING LOG\14199.14A BORING LOGS.GPJ

DEPTH (ft)	CASING (bl/ft)	SAMPLE TYPE NUMBER	RECOVERY/PEN. (in)	DEPTH (FT)	BLOW COUNTS/6"	MIN/FT	GRAPHIC LOG	SAMPLE DESCRIPTION	STRATUM DESCRIPTION
0									
5		S-1	14 / 24	1 - 3	100-73-17-9 (90)			Wet, very dense, brown, fine to coarse GRAVEL, some fine to coarse sand, trace silt.	8-10" PAVEMENT
		S-2	3 / 24	3 - 5	6-6-4-3 (10)			Wet, loose, brown, fine to coarse SAND, little silt, trace coal.	
		S-3	6 / 24	5 - 7	5-2-3-2 (5)			Wet, loose, brown, fine to coarse SAND, some silt, trace fine gravel.	
		S-4	7 / 24	7 - 9	3-1-2-3 (3)			Wet, very loose, brown, fine to coarse SAND and coarse GRAVEL, some silt.	
10		S-5	10 / 24	9 - 11	17-9-5-4 (14)			Wet, medium dense, dark brown, fine to coarse SAND, little silt, trace fine gravel, trace coal, trace fish scale.	FILL
15		S-6	9 / 24	14 - 16	25-37-59-60 (96)			Wet, very dense, dark brown, fine to coarse SAND, trace fine gravel, trace silt, trace wood.	
20		S-7	24 / 0	19 - 19	107/0"			Split Spoon Refusal.	WOOD
		S-8	9 / 24	20 - 22	26-24-29-24 (53)			Wet, very dense, light brown, fine to coarse SAND, trace fine gravel, trace silt.	GLACIAL DEPOSITS
25								Wet, medium dense, light brown, fine to coarse SAND and fine to coarse GRAVEL, little silt.	

GRANULAR SOILS		COHESIVE SOILS		REMARKS:	BURMISTER CLASSIFICATION
BLOWS/FT	DENSITY	BLOWS/FT	CONSIST.		
0 - 4	V. LOOSE	<2	V. SOFT	1.S-6: Black wash 12'-15'. 2.S-7: Obstruction at 19'. 1' boulder.	TRACE 0 - 10%
4 - 10	LOOSE	2 - 4	SOFT		LITTLE 10 - 20%
10 - 30	M. DENSE	4 - 8	M. STIFF		SOME 20 - 35%
30 - 50	DENSE	8 - 15	STIFF		AND 35 - 50%
>50	V. DENSE	15 - 30	V. STIFF		PERCENT BY WEIGHT
		>30	HARD		

NOTES: 1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES, TRANSITIONS MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

(Continued Next Page)

A00604-44



Pare Corporation
10 Lincoln Road, Suite 210
Foxboro, MA 02035
508-543-1755

BORING NUMBER BB17-2

PAGE 2 OF 4

CLIENT MassDOTPROJECT NAME Mill Pond DamPROJECT NUMBER 14199.14PROJECT LOCATION Wareham, MA

PARE BORING LOG - GINT STD US LAB GDT - 12/7/21 16:10 - Y:\JOBS\14 JOBS\14199.14 MASSDOT MILL POND DAM BORING LOG\14199.14A BORING LOGS.GPJ

DEPTH (ft)	CASING (bl/ft)	SAMPLE TYPE NUMBER	RECOVERY/PEN. (in)	DEPTH (FT)	BLOW COUNTS/6"	MIN/FT	GRAPHIC LOG	SAMPLE DESCRIPTION	STRATUM DESCRIPTION
25		S-9	6 / 24	24 - 26	31-15-12-19 (27)				
30		S-10	0 / 24	29 - 31	9-7-6-19 (13)			No Recovery	
35		S-11	5 / 24	34 - 36	13-78-42-14 (120)			Wet, very dense, light brown, fine to coarse SAND, little fine to coarse gravel, trace silt.	
40		S-12	5 / 24	39 - 41	24-18-17-17 (35)			Wet, dense, light brown, coarse GRAVEL and fine to coarse SAND, little silt.	GLACIAL DEPOSITS
45		S-13	8 / 24	44 - 46	22-14-11-19 (25)			Wet, medium dense, light brown, fine to coarse SAND, little fine to coarse gravel, trace silt.	
50		S-14	0 / 24	49 - 51	21-17-13-20 (30)			No Recovery.	
55		S-15	5 / 24	54 - 56	19-76-54-34 (130)			Wet, very dense, light brown, fine to coarse SAND, some coarse gravel, trace silt.	

3.S-11: Obstruction at 35'.
4.S-14 & S-15: Rock in tip.



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BORING NUMBER BB17-2

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CLIENT MassDOTPROJECT NAME Mill Pond DamPROJECT NUMBER 14199.14PROJECT LOCATION Wareham, MA

PARE BORING LOG - GINT STD US LAB GDT - 12/7/21 16:10 - Y:\JOBS\14 JOBS\14199.14 MASSDOT MILL POND DAM BORING LOG\14199.14A BORING LOGS.GPJ

DEPTH (ft)	CASING (bl/ft)	SAMPLE TYPE NUMBER	RECOVERY/PEN. (in)	DEPTH (FT)	BLOW COUNTS/6"	MIN/FT	GRAPHIC LOG	SAMPLE DESCRIPTION	STRATUM DESCRIPTION
60		S-16	9 / 24	59 - 61	30-65-34-25 (99)			Wet, very dense, light brown, fine to coarse SAND, little silt, trace fine gravel.	GLACIAL DEPOSITS
65								Roller bit down to bedrock.	
70									WEATHERED SCHIST
75									
80									SCHIST
85		C1	30 / 60	84 - 89		6 6 5 10		Very poor, moderately weathered, hard, extremely fractured SCHIST. REC = 50% RQD = 0%	

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Pare Corporation
10 Lincoln Road, Suite 210
Foxboro, MA 02035
508-543-1755

BORING NUMBER BB17-2

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CLIENT MassDOTPROJECT NAME Mill Pond DamPROJECT NUMBER 14199.14PROJECT LOCATION Wareham, MA

DEPTH (ft)	CASING (bl/ft)	SAMPLE TYPE NUMBER	RECOVERY/PEN. (in)	DEPTH (FT)	BLOW COUNTS/6"	MIN/FT	GRAPHIC LOG	SAMPLE DESCRIPTION	STRATUM DESCRIPTION
90						10			
						7		Very poor, moderately weathered, hard, extremely fractured SCHIST. REC = 26.7% RQD = 0%	
						15			
		C2	16 / 60	89 - 94		12			SCHIST
						10			
						11			

Bottom of borehole at 94.0 feet.

PARE BORING LOG - GINT STD US LAB GDT - 12/7/21 16:10 - Y:\JOBS\14 JOBS\14199.14 MASSDOT MILL POND DAM BORING LOG\14199.14A BORING LOGS.GPJ

95

100

105

110

115

BORING NUMBER BB17-3

PAGE 1 OF 2



Pare Corporation
10 Lincoln Road, Suite 210
Foxboro, MA 02035
508-543-1755

CLIENT MassDOTPROJECT NAME Mill Pond DamPROJECT NUMBER 14199.14PROJECT LOCATION Wareham, MADATE STARTED 4/11/18 COMPLETED 4/12/18GROUND ELEVATION 14.3 ft NAVD88 HOLE SIZE 4 in.DRILLING CONTRACTOR New England Boring Contractors, Inc.

GROUND WATER LEVELS:

DRILLING METHOD Wash and DriveAT TIME OF DRILLING --- Mudline is 10' below pond surfaceLOGGED BY J. Costa CHECKED BY SJMAT END OF DRILLING --- N/ABORING LOCATION 2739676.04°N / 880949.9°E

PARE BORING LOG - GINT STD US LAB GDT - 12/7/21 16:10 - Y:\JOBS\14 JOBS\14199.14 MASSDOT MILL POND DAMBORING LOG\14199.14A BORING LOGS.GPJ

DEPTH (ft)	CASING (bl/ft)	SAMPLE TYPE NUMBER	RECOVERY/PEN. (in)	DEPTH (FT)	BLOW COUNTS/6"	MIN/FT	GRAPHIC LOG	SAMPLE DESCRIPTION	STRATUM DESCRIPTION
0									
		S-1	12 / 24	0 - 2	1-14-7-9 (21)			Wet, medium dense, brown to black, fine to coarse SAND, little fine gravel, trace silt, trace organics, organic scented/odor.	POND MUCK
		S-2	11 / 24	2 - 4	8-5-6-8 (11)			Wet, medium dense, brown to black, fine to coarse SAND and fine to coarse GRAVEL, trace silt, trace organics.	
5		S-3	2 / 24	4 - 6	22-15-12-19 (27)			Wet, medium dense, tan, fine to coarse GRAVEL, trace coarse sand.	GRAVEL
		S-4	9 / 24	6 - 8	26-22-21-14 (43)			Wet, dense, light gray, fine to medium SAND, little silt, little coarse sand, little fine to coarse gravel.	GLACIAL DEPOSITS
10		S-5	14 / 24	8 - 10	16-33-25-14 (58)			Wet, very dense, light gray, fine to medium SAND, little silt, little coarse sand, little fine to coarse gravel.	
		S-6	9 / 24	10 - 12	50-11-10-8 (21)			Wet, medium dense, light gray, fine to medium SAND, little coarse sand, trace fine gravel, trace silt.	
		S-7	10 / 24	12 - 14	9-4-2-4 (6)			Wet, loose, light gray, fine to medium SAND, little coarse sand, little silt, trace fine gravel.	
15		S-8	9 / 24	14 - 16	13-22-18-18 (40)			Wet, dense, light gray, fine to coarse SAND, little fine gravel, trace silt.	
		S-9	6 / 24	16 - 18	15-36-23-38 (59)			Wet, very dense, light gray, fine to coarse SAND, little fine gravel, trace silt.	
		S-10	9 / 24	18 - 20	50-16-15-12 (31)			Wet, dense, light gray, fine to coarse SAND and fine to coarse GRAVEL, trace silt.	
20									
25								Wet, very dense, light gray, fine to coarse SAND and fine to coarse GRAVEL, trace silt.	

GRANULAR SOILS		COHESIVE SOILS		REMARKS:	BURMISTER CLASSIFICATION
BLOWS/FT	DENSITY	BLOWS/FT	CONSIST.		
0 - 4	V. LOOSE	<2	V. SOFT		TRACE 0 - 10%
4 - 10	LOOSE	2 - 4	SOFT		LITTLE 10 - 20%
10 - 30	M. DENSE	4 - 8	M. STIFF		SOME 20 - 35%
30 - 50	DENSE	8 - 15	STIFF		AND 35 - 50%
>50	V. DENSE	15 - 30	V. STIFF		PERCENT BY WEIGHT
		>30	HARD		

NOTES: 1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES, TRANSITIONS MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

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CLIENT MassDOTPROJECT NAME Mill Pond DamPROJECT NUMBER 14199.14PROJECT LOCATION Wareham, MA

PARE BORING LOG - GINT STD US LAB GDT - 12/7/21 16:10 - Y:\JOBS\14 JOBS\14199.14 MASSDOT MILL POND DAM BORING LOG\14199.14A BORING LOGS.GPJ

DEPTH (ft)	CASING (bl/ft)	SAMPLE TYPE NUMBER	RECOVERY/PEN. (in)	DEPTH (FT)	BLOW COUNTS/6"	MIN/FT	GRAPHIC LOG	SAMPLE DESCRIPTION	STRATUM DESCRIPTION
25		S-11	6 / 24	24 - 26	51-27-55-39 (82)				
30		S-12	7 / 24	29 - 31	53-44-18-17 (62)			Wet, very dense, light gray, fine to coarse SAND and fine GRAVEL, trace silt.	GLACIAL DEPOSITS
35		S-13	9 / 24	34 - 36	16-6-7-8 (13)			Wet, medium dense, light gray, fine to coarse SAND and fine to coarse GRAVEL, trace silt.	
40		S-14A	0 / 0	39 - 40	46-40/0"			BOULDER, very strong, red to gray GRANITE, massive, fresh.	BOULDER
		S-14B	7 / 7	40 - 40.6	33-50/1"			Wet, very dense, medium to coarse SAND, trace silt (appears to be wash).	
45		S-15	13 / 18	43 - 44.5	33-81-80 (161)			Wet, very dense, light gray/light tan, fine to coarse SAND, some fine gravel, trace silt.	GLACIAL DEPOSITS
50		S-16	1 / 1	48 - 48.1	50/1"			Wet, very dense, medium to coarse SAND, trace silt (appears to be wash).	
55								Bottom of borehole at 53.0 feet.	

- S-14A: 46 blows with 140 lb. hammer, Driller switched to the 300 lb. Hammer to attempt to retrieve a sample, performed 40 blows, no penetration. 6" of wash was recovered in the sample.
- C-1 was performed from 39'-40', NX size.
- Casing refusal at 49'±, used roller bit to advance to 53'. Driller reported cobble/boulder at 53'.
- S-16: Driller used 300 lb. hammer to retrieve sample. 17 blows for zero penetration.
- Borehole was tremie grouted through drill rods with two-94 lbs. Portland cement, 40 gal. of water and 5 lbs. Bentonite powder.

BORING NUMBER HA-1

PAGE 1 OF 1



Pare Corporation
10 Lincoln Road, Suite 210
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T: 508-543-1755
F: 508-543-1881

CLIENT MassDOTPROJECT NAME Mill Pond DamPROJECT NUMBER 14199.14PROJECT LOCATION Wareham, MADATE STARTED 11/24/08COMPLETED 11/24/08

GROUND ELEVATION _____

HOLE SIZE inches

DRILLING CONTRACTOR _____

GROUND WATER LEVELS:

DRILLING METHOD _____

AT TIME OF DRILLING ---LOGGED BY Varinia Arauco Vera

CHECKED BY _____

AT END OF DRILLING ---

BORING LOCATION _____

PARE BORING LOG - GINT STD US LAB GDT - 8/16/18 15:21 - Y:\JOBS\14 JOBS\14199.14 MASSDOT MILL POND DAMBORING LOG\14199.14 BORING LOGS.GPJ

DEPTH (ft)	CASING (bl/ft)	SAMPLE TYPE NUMBER	RECOVERY/PEN. (in)	DEPTH (FT)	BLOW COUNTS/6"	MIN/FT	GRAPHIC LOG	SAMPLE DESCRIPTION	STRATUM DESCRIPTION
0				0 - 0.4				Topsoil	
				0.4 - 2				Moist, dark brown, fine to medium SAND, some inorganic silt, little fine gravel, trace roots.	
				2 - 2.5				Moist, dark black, fine to coarse SAND, some inorganic silt, little gravel, trace cinder ash.	
5				2.5 - 2.5				Moist, tan/orange, fine to coarse SAND, little gravel, trace inorganic silt.	
								Bottom of borehole at 2.5 feet.	
10									
15									
20									
25									
30									
35									

GRANULAR SOILS		COHESIVE SOILS		REMARKS:	BURMISTER CLASSIFICATION
BLOWS/FT	DENSITY	BLOWS/FT	DENSITY		
0 - 4	V. LOOSE	<2	V. SOFT	1. Hand auger advanced with dutch closed bucket hand auger.	TRACE 0 - 10%
4 - 10	LOOSE	2 - 4	SOFT		LITTLE 10 - 20%
10 - 30	M. DENSE	4 - 8	M. STIFF		SOME 20 - 35%
30 - 50	DENSE	8 - 15	STIFF		AND 35 - 50%
>50	V. DENSE	15 - 30	V. STIFF		PERCENT BY WEIGHT
		>30	HARD		

NOTES: 1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES, TRANSITIONS MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

BORING NUMBER HA-2

PAGE 1 OF 1



Pare Corporation
10 Lincoln Road, Suite 210
Foxboro, MA 02035
T: 508-543-1755
F: 508-543-1881

CLIENT MassDOTPROJECT NAME Mill Pond DamPROJECT NUMBER 14199.14PROJECT LOCATION Wareham, MADATE STARTED 11/24/08COMPLETED 11/24/08

GROUND ELEVATION _____

HOLE SIZE _____ inches

DRILLING CONTRACTOR _____

GROUND WATER LEVELS:

DRILLING METHOD _____

AT TIME OF DRILLING ---LOGGED BY Varinia Arauco Vera

CHECKED BY _____

AT END OF DRILLING ---

BORING LOCATION _____

PARE BORING LOG - GINT STD US LAB GDT - 8/16/18 15:21 - Y:\JOBS\14 JOBS\14199.14 MASSDOT MILL POND DAM BORING LOG\14199.14 BORING LOGS.GPJ

DEPTH (ft)	CASING (bl/ft)	SAMPLE TYPE NUMBER	RECOVERY/PEN. (in)	DEPTH (FT)	BLOW COUNTS/6"	MIN/FT	GRAPHIC LOG	SAMPLE DESCRIPTION	STRATUM DESCRIPTION
0				0 - 0.4				Topsoil	
				0.4 - 0.6				Moist, tan, fine to medium SAND, little organic silt, trace fine gravel.	
				1 - 2				Moist, tan, fine to coarse SAND, little silt, trace gravel.	
5				2 - 3				Moist, light, fine to medium SAND, trace silt.	
				3 - 4				Moist, light tan/orange, fine to medium SAND, trace silt.	
				4 - 4				Moist, light tan/orange, fine to medium SAND, trace silt. Bottom of borehole at 4.0 feet.	
10									
15									
20									
25									
30									
35									

GRANULAR SOILS		COHESIVE SOILS		REMARKS:	BURMISTER CLASSIFICATION
BLOWS/FT	DENSITY	BLOWS/FT	DENSITY		
0 - 4	V. LOOSE	<2	V. SOFT	1. Hand auger advanced with dutch closed bucket hand auger.	TRACE 0 - 10%
4 - 10	LOOSE	2 - 4	SOFT		LITTLE 10 - 20%
10 - 30	M. DENSE	4 - 8	M. STIFF		SOME 20 - 35%
30 - 50	DENSE	8 - 15	STIFF		AND 35 - 50%
>50	V. DENSE	15 - 30	V. STIFF		PERCENT BY WEIGHT
		>30	HARD		

NOTES: 1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES, TRANSITIONS MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

BORING NUMBER HA-3

PAGE 1 OF 1

PARE CORPORATION
 Pare Corporation
 10 Lincoln Road, Suite 210
 Foxboro, MA 02035
 T: 508-543-1755
 F: 508-543-1881

CLIENT MassDOTPROJECT NAME Mill Pond DamPROJECT NUMBER 14199.14PROJECT LOCATION Wareham, MADATE STARTED 11/24/08COMPLETED 11/24/08

GROUND ELEVATION _____

HOLE SIZE inches

DRILLING CONTRACTOR _____

GROUND WATER LEVELS:

DRILLING METHOD _____

AT TIME OF DRILLING ---LOGGED BY Varinia Arauco Vera

CHECKED BY _____

AT END OF DRILLING ---

BORING LOCATION _____

SAMPLE DESCRIPTION

STRATUM
DESCRIPTIONDEPTH
(ft)

CASING (bl/ft)

SAMPLE TYPE
NUMBERRECOVERY/PEN.
(in)

DEPTH (FT)

BLOW
COUNTS/6"

MIN/FT

GRAPHIC
LOG

Wet, brown, fine to coarse SAND, little inorganic silt, trace fine gravel.

Bottom of borehole at 0.6 feet.

PARE BORING LOG - GINT STD US LAB GDT - 8/16/18 15:21 - Y:\JOBS\14 JOBS\14199.14 MASSDOT MILL POND DAM BORING LOG\14199.14 BORING LOGS.GPJ

GRANULAR SOILS	
BLOWS/FT	DENSITY
0 - 4	V. LOOSE
4 - 10	LOOSE
10 - 30	M. DENSE
30 - 50	DENSE
>50	V. DENSE

COHESIVE SOILS	
BLOWS/FT	DENSITY
<2	V. SOFT
2 - 4	SOFT
4 - 8	M. STIFF
8 - 15	STIFF
15 - 30	V. STIFF
>30	HARD

REMARKS:

1. Hand auger advanced with dutch closed bucket hang auger.

BURMISTER
CLASSIFICATION

TRACE	0 - 10%
LITTLE	10 - 20%
SOME	20 - 35%
AND	35 - 50%
PERCENT BY WEIGHT	

NOTES: 1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES, TRANSITIONS MAY BE GRADUAL.
 2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

BORING NUMBER HA-4

PAGE 1 OF 1



Pare Corporation
10 Lincoln Road, Suite 210
Foxboro, MA 02035
T: 508-543-1755
F: 508-543-1881

CLIENT MassDOTPROJECT NAME Mill Pond DamPROJECT NUMBER 14199.14PROJECT LOCATION Wareham, MADATE STARTED 11/24/08COMPLETED 11/24/08

GROUND ELEVATION _____

HOLE SIZE inches

DRILLING CONTRACTOR _____

GROUND WATER LEVELS:

DRILLING METHOD _____

AT TIME OF DRILLING ---LOGGED BY Varinia Arauco Vera

CHECKED BY _____

AT END OF DRILLING ---

BORING LOCATION _____

PARE BORING LOG - GINT STD US LAB GDT - 8/16/18 15:21 - Y:\JOBS\14 JOBS\14199.14 MASSDOT MILL POND DAM BORING LOG\14199.14 BORING LOGS.GPJ

DEPTH (ft)	CASING (bl/ft)	SAMPLE TYPE NUMBER	RECOVERY/PEN. (in)	DEPTH (FT)	BLOW COUNTS/6"	MIN/FT	GRAPHIC LOG	SAMPLE DESCRIPTION	STRATUM DESCRIPTION
0				0 - 0.8				Topsoil	
				0.8 - 2				Moist, tan, fine to medium SAND, some organic silt, trace roots.	
				2 - 2.34				Dry, dark brown, fine to medium SAND and organic SILT, little gravel, trace roots.	
5								Dry, dark brown, fine to medium SAND, some silt, trace fine gravel and roots.	
								Bottom of borehole at 2.4 feet.	
10									
15									
20									
25									
30									
35									

GRANULAR SOILS		COHESIVE SOILS		REMARKS:	BURMISTER CLASSIFICATION
BLOWS/FT	DENSITY	BLOWS/FT	DENSITY		
0 - 4	V. LOOSE	<2	V. SOFT	1. Hand auger advanced with dutch closed bucket hang auger.	TRACE 0 - 10%
4 - 10	LOOSE	2 - 4	SOFT		LITTLE 10 - 20%
10 - 30	M. DENSE	4 - 8	M. STIFF		SOME 20 - 35%
30 - 50	DENSE	8 - 15	STIFF		AND 35 - 50%
>50	V. DENSE	15 - 30	V. STIFF		PERCENT BY WEIGHT
		>30	HARD		

NOTES: 1) THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY BETWEEN SOIL TYPES, TRANSITIONS MAY BE GRADUAL.
2) WATER LEVEL READINGS HAVE BEEN MADE IN THE DRILL HOLES AT TIMES AND UNDER CONDITIONS STATED ON THE BORING LOGS. FLUCTUATIONS IN THE LEVEL OF GROUNDWATER MAY OCCUR DUE TO OTHER FACTORS THAN THOSE PRESENT AT THE TIME MEASUREMENTS WERE MADE.

Cranberry Highway/Mill Pond Dam over the Agawam River

APPENDIX B

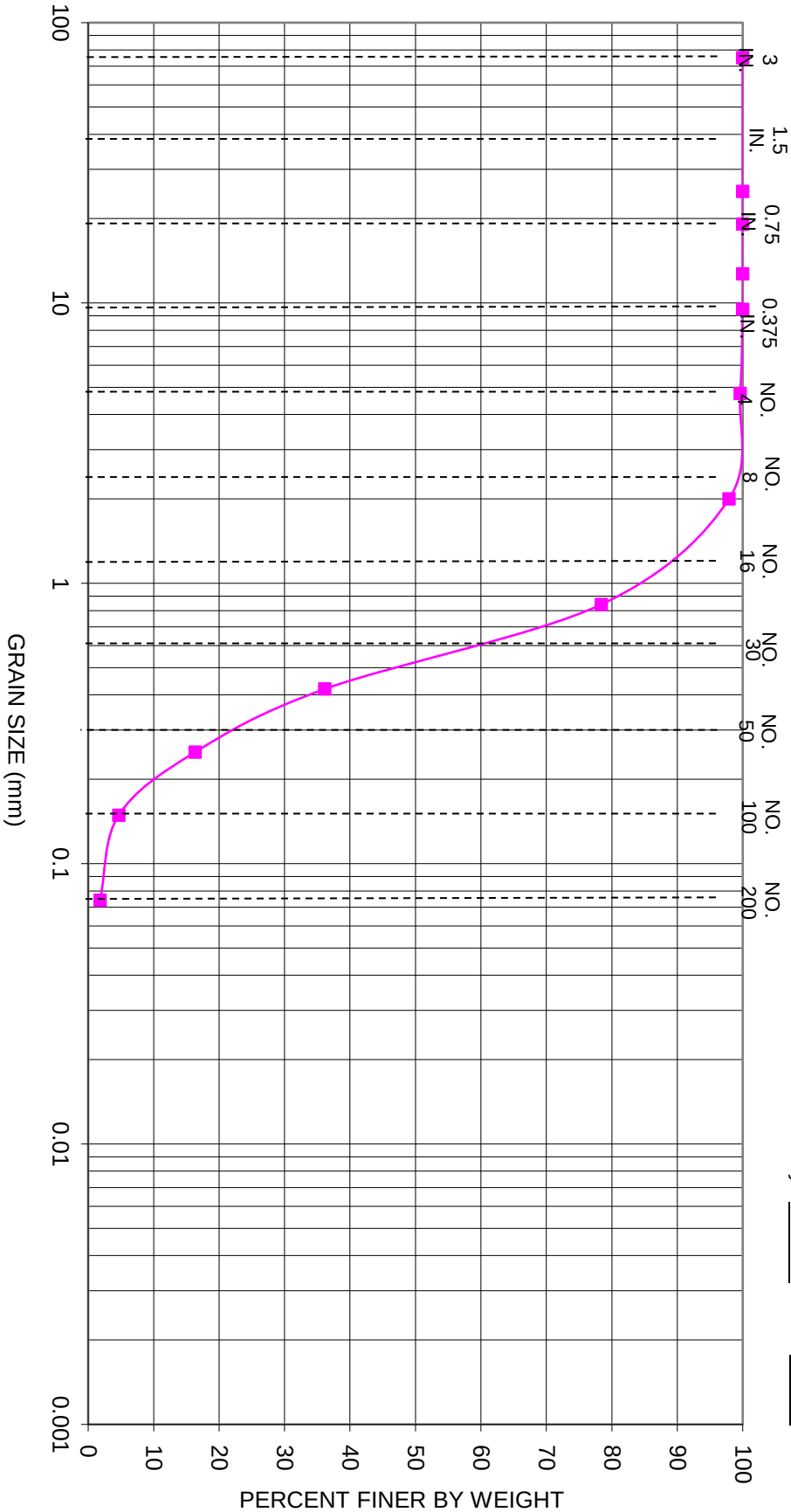
Laboratory Data

Cranberry Highway/Mill Pond Dam over the Agawam River



U.S. STANDARD SIEVE SIZE

Tested By: MLP Date: 4/18/18
Checked By: DMM Date: 7/19/18



GRAVEL			SAND		SILT	CLAY	
COARSE	FINE	COARSE	MEDIUM	FINE			
TEST		MATERIAL SOURCE		REMARKS			
1	Strata: Glacial Deposits Boring: BB17-1 Sample: S-10 Depth: 19'- 21'		Burmister - Medium SAND, Some fine sand and silt. Unified Soil Classification System - (SP) - Poorly-graded SAND				



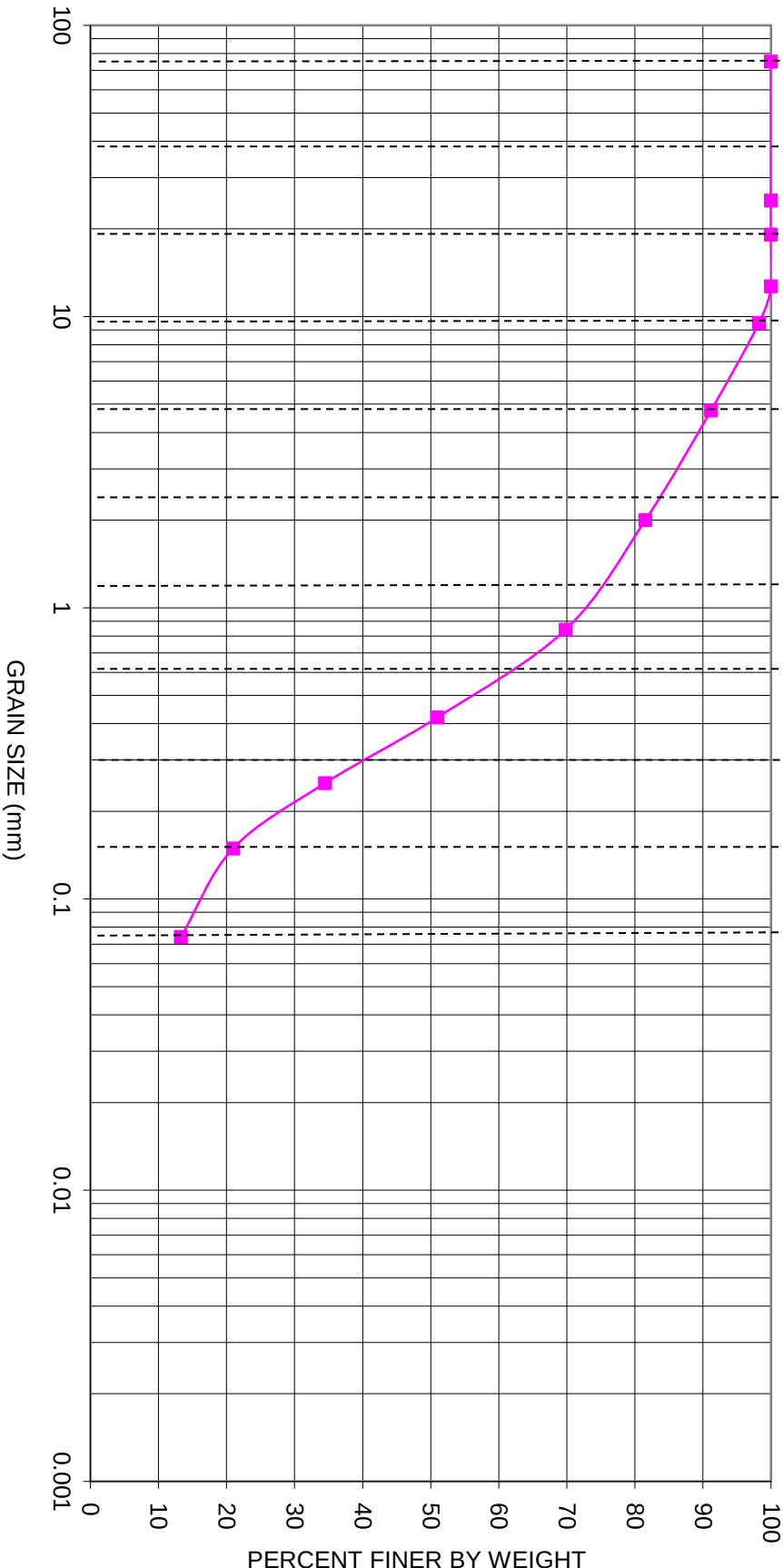
CP
PAARE
CORPORATION



U.S. STANDARD SIEVE SIZE

Tested By: MLP Date: 4/18/18

Checked By: DMM Date: 7/19/18



GRAVEL		SAND			SILT	CLAY
COARSE	FINE	COARSE	MEDIUM	FINE		
TEST	MATERIAL SOURCE				REMARKS	
2	Strata: Fill Boring: BB17-2 Sample: S-5 Depth: 9-11				Burmister - Fine to coarse SAND, little silt, trace fine gravel, and silt. Unified Soil Classification System - (SM) - Well-Graded Sand with silt	



PARE
CORPORATION



Cranberry Highway/Mill Pond Dam over the Agawam River

APPENDIX C

Geotechnical Limitations

Cranberry Highway/Mill Pond Dam over the Agawam River





GEOTECHNICAL LIMITATIONS

Explorations

1. The analyses and recommendations submitted in this report are based in part upon the data obtained from subsurface explorations. The nature and extent of variations between these explorations may not become evident until construction. If variations then appear evident, Pare Corporation (Pare) should be asked to reevaluate the recommendations of this report.
2. The generalized soil profile described in the text is intended to convey trends in the subsurface conditions. The boundaries between strata are approximate and idealized and have been developed by interpretations of widely spaced explorations and samples; actual soil transitions are probably more erratic. For specific information, refer to the boring logs.
3. Water level readings have been made in the drill holes at the times and under the conditions stated on the boring logs. These data have been reviewed and interpretations have been made in the text of this report. However, fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors occurring since the time the measurements were made.

Review

4. In the event that any changes in the nature or location of the proposed building are planned, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the conclusions of this report are verified in writing by Pare. Pare should also be provided with the opportunity for a general review of the final design and specifications in order that the earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications.

Construction

5. PARE should be retained to provide soil engineering services during construction of the excavation and foundation phases of work in order to observe compliance with the design concepts, specifications, and recommendations and to allow design changes in the event that subsurface conditions differ from those indicated prior to the start of construction.

Use of Report

6. This report has been prepared for the exclusive use of MassDOT for specific application for the design and construction of the proposed replacement of Bridge No. W-06-020 located in Wareham, Massachusetts in accordance with generally accepted engineering practices. No other warranty, expressed or implied, is made.
7. This engineering report has been prepared for this project by Pare. This report is for design purposes only and is not necessarily sufficient to prepare an accurate bid. Contractors wishing a copy of this report may secure it with the understanding that its scope is limited to design considerations only.



GENERAL INVESTIGATION NOTES

GENERAL

1. All depths are given in feet measured from the ground surface unless otherwise noted. Depth of angled borings is measured along the axis of the boring.
2. The identification and description of soils is based on visual inspection of the retrieved samples using the Burmister Classification System as modified by MassDOT. Descriptions of boring logs apply only at the specific boring locations and at the time the borings were made. They are not warranted to be representative of subsurface conditions at other locations or times.
3. Water levels are observed at the end of boring (E.O.B.) or/and on a long-term basis through the use of strategically placed observation wells. The indicated levels may not reflect the actual groundwater levels. Fluctuations in groundwater levels can occur due to variations in precipitation, season, tidal fluctuation, adjacent construction activity and construction dewatering systems, and other factors.

SOIL DESCRIPTION

1. The Standard Penetration (SPT) test is performed in general accordance with ASTM D-1586. The standard penetration resistance (N) is defined as the number of blows required to drive a 2-inch O.D., 1 3/8-inch I.D. split-spoon sampler by 12 inches by dropping a 140-lb hammer through a vertical distance of 30 inches. The sampler is normally driven 3 (for 18-inch long sampler) or 4 (for 24-inch long sampler) successive 6-inch increments. The first 6-inch is considered to be a seating drive, therefore the sum of the second and third increments are used in determining the N value.

2. Consistency/Condition

<u>Coarse-Grained Soils</u>	<u>Relative Density (%)</u>	<u>N (blows per foot)</u>
Very loose	0-15	0-4
Loose	15-35	4-10
Medium dense	35-65	10-30
Dense	65-85	30-50
Very dense	85-100	>50

<u>Fine-Grained Soils</u>	<u>Unconfined Compressive Strength, q_u (tsf)</u>	<u>N (blows per foot)</u>	<u>Field Identification</u>
Very Soft	<0.25	0-2	Exudes between fingers when squeezed in hand
Soft	0.25-0.50	2-4	Molded by light finger pressure
Medium	0.50-1.00	4-8	Molded by strong finger pressure
Stiff	1.00-2.00	8-15	Indented by thumb
Very Stiff	2.00-4.00	15-30	Indented by thumbnail
Hard	>4.00	>30	Difficult to indent by thumbnail

<u>Grain Size</u>	<u>Descriptive Adjective*</u>
Boulders – >12 in.	Trace 0-10% or less
Cobbles – 3 in. - 12 in.	Some 10-40%
Gravel – Coarse, 3/4 in. - 3 in.	And 40-50%
– Fine, 0.19 in. (#4) to 3/4 in.	Percent by Weight
Sand – Coarse, 0.079 in. (#10) to 0.19 in. (#4)	
– Medium, 0.017 in. (#40) to 0.079 in. (#10)	
– Fine, 0.0029 in. (#200) to 0.017 in. (#40)	
Silt – 0.0002 in. to 0.0029 in. (#200)	
Clay – <0.0002 in.	

*Burmister Classification System as modified by MassDOT

ROCK DESCRIPTION

1. Core recovery is the total length of rock core recovered from a core run divided by the length of the run, expressed as a percentage.
2. Rock Quality Designation (RQD) is the total length of hard, sound pieces of rock core greater than 4-inches from a core run divided by the length of the run, expressed as a percentage.

<u>RQD (%)</u>	<u>Description</u>	<u>Approximate Equivalent Fracture Spacing (feet)</u>
0-25	Very Poor	Very close (<0.2)
25-50	Poor	Close (0.2-1)
50-75	Fair	Moderately wide (1-3)
75-90	Good	Wide (3-10)
90-100	Excellent	Very wide (>10)

3. "Weathering" refers to the degree of alteration observed in the rock core, which is produced by chemical and/or mechanical processes.

<u>Grade</u>	<u>Symbol</u>	<u>Recognition</u>
Fresh	F	No visible sign of decomposition or discoloration. Rings under hammer impact.
Slightly Weathered	WS	Slight discoloration inwards from open fractures, otherwise similar to F.
Moderately Weathered	WM	Discoloration throughout. Weaker minerals such as feldspar decomposed. Strength somewhat

Highly Weathered	WH	less than fresh rock but cores cannot be broken by hand or scraped by knife. Texture preserved.
Completely Weathered	WC	Most minerals somewhat decomposed. Specimens can be broken by hand with effort or shaved with knife. Core stones present in rock mass. Texture becoming indistinct but fabric preserved.
Residual Soil	RS	Minerals decomposed to soil but fabric and structure preserved (Saprolite). Specimens easily crumbled or penetrated.
		Advance state of decomposition resulting in plastic soils. Rock fabric and structure completely destroyed. Large volume change.

4. "Hardness" is an estimate of the rock strength that is a function of lithology and the degree of weathering.

<u>Class</u>	<u>Hardness</u>	<u>Field Test</u>	<u>Approximate Range of Uniaxial Compression Strength kg/cm² (tons/ft²)</u>
I	Extremely Hard	Many blows with geologic hammer required to break intact specimen.	>2,000
II	Very Hard	Hand held specimen breaks with hammer end of pick under more than one blow.	2,000 – 1,000
III	Hard	Cannot be scraped or peeled with knife, hand held specimen can be broken with single moderate blow with pick.	1,000 – 500
IV	Soft	Can just be scraped or peeled with knife. Indentation 1 mm to 3 mm show in specimen with moderate blow with pick.	500 – 250
V	Very Soft	Material crumbles under moderate blow with sharp end of pick and can be peeled with a knife, but is too hard to hand-trim for triaxial test specimen.	250 – 10

5. Discontinuity Descriptions

Rock Continuity: Any break in a rock whether or not it has undergone relative displacement.

Extremely Fractured – Drill core stem less than 1 in.

Moderately Fractured – Drill core stem 1 in. to 4 in.

Slightly Fractured – Drill core stem 4 in. to 8 in.

Sound – Drill core stem greater than 8 in.

Texture: Terminology used to identify size, shape and arrangement of constituent elements.

Amorphous – Too small to be seen with naked eye.

Fine Grained – Barely seen with naked eye.

Medium Grained – Barely seen with naked eye to 1/8 in.

Coarse Grained – 1/8 in to 1/4 in.

Very Coarse Grained > 1/4 in.

Discontinuities: Surface representing breaks or fractures separating the rock mass into discrete units.

Crack – A partial or incomplete fracture.

Joint – A simple fracture along which no shear displacement has occurred. May form joint sets.

Shear – A fracture along which differential movement has taken place parallel to the surface sufficient to produce slickensides or polishing. May be accompanied by a zone of fractured rock up to a few inches wide.

Fault – A major fracture along which there has been appreciable displacement and accompanied by gouge and/or a severely fractured adjacent zone.

Shear or Fault Zone – A band or zone of parallel, closely spaced shears or faults.

Fractures, Bedding, and Foliation, Spacing and Attitude:

<u>Fractures</u>	<u>Bedding and Foliation</u>	<u>Spacing</u>	<u>Attitude</u>	<u>Dip Angle</u>
Very Close	Very Thin	Less than 2 in.	Horizontal	0 – 5
Close	Thin	2 in. – 1 ft.	Shallow or low angle	5 – 35
Moderate	Medium	1 ft. – 3 ft.	Moderately dipping	35 – 55
Wide	Thick	3 ft. – 10 ft.	Steep or high angle	55 – 85
Very Wide	Very Thick	More than 10 ft.	Vertical	85 – 90

DRILLING CODES

HSA	Hollow Stem Auger	SS	Split Spoon Sample
C/A	Casing Advancement	AS	Auger Sample
BX	Rock Cored with BX Core Barrel (Produces 1 5/8"-diameter core)	ST	Shelby Tube Sample
NX	Rock Cored with NX Core Barrel (Produces 2 1/8"-diameter core)	WS	Washed Sample
		NR	No Recovery

Cranberry Highway/Mill Pond Dam over the Agawam River

APPENDIX D
References

Cranberry Highway/Mill Pond Dam over the Agawam River



Cranberry Highway/Mill Pond Dam over the Agawam River

REFERENCES

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2. IBC 2012. “International Building Code 2012”, International Code Council, 2012.
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5. NAVY 1997. “Soil Dynamics and Special Design Aspects”, Department of Defense Handbook, NAVFAC DM 7.3, 11/15/97
6. DAS 1990. “Principles of Foundation Engineering, 7th Edition”, 1990.
7. NAVY 1986. “Foundation and Earth Structures, Design Manual 7.1 and 7.2”, Naval Facilities Engineering Command, 1982. Revalidated by change 9/1/86.
8. LAMB, WHITMAN 1969. “Soil Mechanics”, Lamb, Whitman, 1969.
9. YOUNG 2001. “Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils”, Young et. Al., 2001.
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14. STONE 2011-S. “Surficial Geologic Map of the Norton-Manomet-Westport-Sconticut Neck 23-Quadrangle Area in Southeast Massachusetts”, Stone, Bryon D., 2011.
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Cranberry Highway/Mill Pond Dam over the Agawam River

APPENDIX E
Calculations

Cranberry Highway/Mill Pond Dam over the Agawam River



Cranberry Highway/Mill Pond Dam over the Agawam River

Soil Parameters





Project: Mill Pond Dam
 Subject: Soil Parameter Table
 Computations By: MLP
 Checked By: DMM

Project No.: 14199.14
 Date: 4/18/2018
 Date: 7/20/2018

Soil Properties are for seepage and stability analyses

Stratum No.	Soil Type ⁶	(N ₁) ₆₀ (Blows/ft)	D _r ¹ (%)	γ_{dry}^2 (PCF)	e ²	ϕ^2 (degrees)	γ_{sat}^3 (PCF)	γ_{boy} (PCF)	k ⁵ (ft/sec)	Gravimetric Water Content, wc	Volumetric Water Content ² ft ³ /ft ³	Comments
1	Pavement	-	-	-	-	-	-	-	-	-	-	
2	Fill	8	25	102	0.65	31	126.6	64.2	-	-	-	
3	Timber Construction Debris	-	-	-	-	-	-	-	-	-	-	
4A Below Dam	Sand and Silt	61	90	112	0.48	38	132	70				Boring Nos. BB17-1 and BB17-2 are on the dam so pre-consolidation under 15 ft of fill is likely the cause of the higher N value for Stratum 4A compared to the soils encountered within the impoundment at BB17-3.
4A Below Impoundment	Sand and Silt	35	70	109	0.54	35	131	68				
4B	Glacial Till	100	95	127	0.32	37	142	80	-	-	-	Glacial Till and Weathered Rock are Assumed to have similar Properties
5	Weathered Rock	100	95	127	0.32	37	142	80	-	-	-	

1. Das, B., "Principles of Geotechnical Engineering", 6th edition, Table 17.5
2. Department of the Navy, "NAVFAC DM-7.1", 1982, Chapter 3, Figure 7
3. $\gamma_{sat} = \gamma_{dry} + (e/(1+e))\gamma_w$
4. $\gamma_{moist} = (\gamma_{dry} + \gamma_{sat})/2$
5. Assumed values based on Das, B., "Principles of Geotechnical Engineering", 6th Ed, Table 6.1, and available descriptions on boring logs
6. Soil classification is based on description on available boring logs

Stratum 4B:
Glacial Till

Stratum 5:
Weathered Rock

Stratum 4A (Below Dam):
Sand and Silt

Stratum 4A (Below Pond):
Sand and Silt

Stratum 2:
FILL

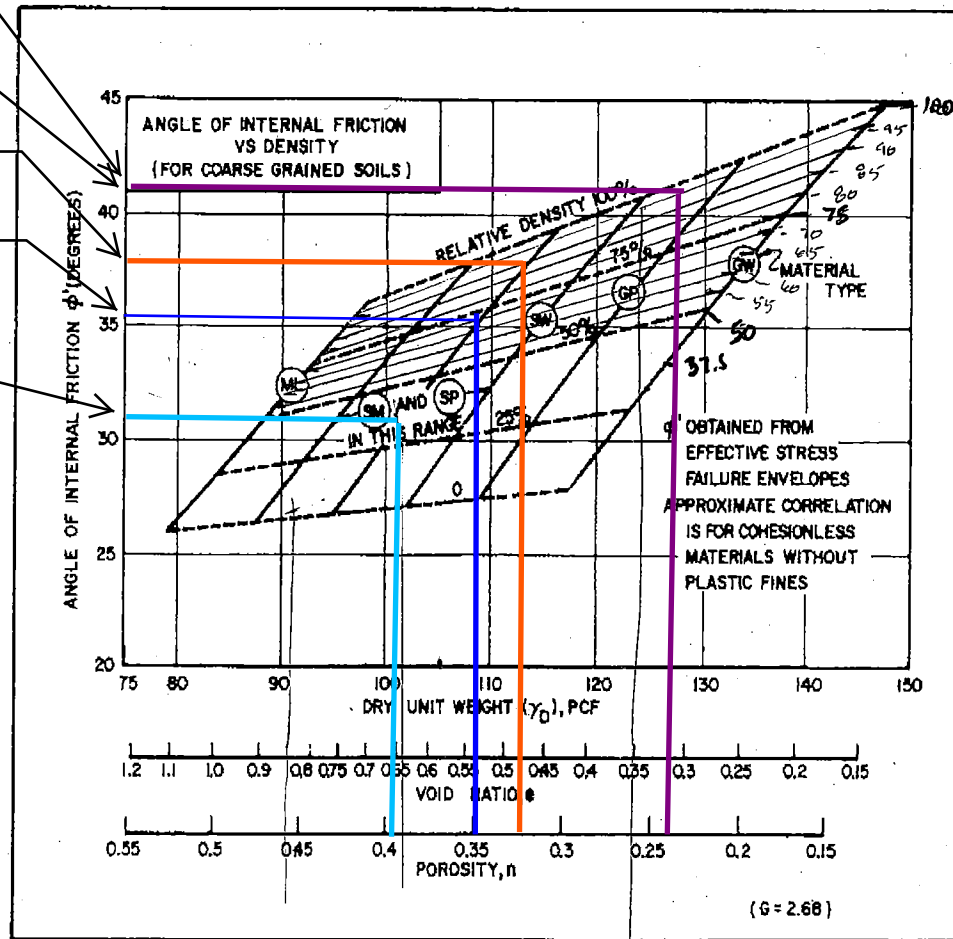


FIGURE 7
Correlations of Strength Characteristics for Granular Soils

$$w = \frac{n}{(1-n)G_s}$$

$$e = \frac{n}{1-n}$$

$$w = \frac{S_r(e)}{G_s}$$

$S_r = 1.0$ for fully saturated
 $G_s = 2.65$

*Eubrinovski and
'Ishikawa
(1999)*

(N1) ₆₀	D _r (%)
0	0
1	1
2	2
3	3
4	4
5	5
6	10
7	15
8	20
9	25
10	30
11	31.5
12	33
13	34.5
14	36
15	37.5
16	39
17	40.5
18	42
19	43.5
20	45
21	46.5
22	48
23	49.5
24	51
25	52.5
26	54
27	55.5
28	57
29	58.5
30	60
31	61.75
32	63.5
33	65.25
34	67
35	68.75
36	70.5
37	72.25
38	74
39	75.75
40	77.5
41	79.25
42	81
43	82.75
44	84.5
45	86.25
46	88
47	89.75
48	91.5
49	93.25
50	95



Project: Mass DOT Mill Pond Dam
 Subject: Modified SPT Development
 Computations By: MLP
 Checked By: DMM

Proj. No.: 14199.12

Date: December 2021
 Date: December 2021

N-Bar Calculation			Boring B17-1	
Depth	Di	Layer thickness	Raw SPT N	d/ni
0 - 3	3		100	0.03
3 - 5	2		50	0.04
5 - 7	2		27	0.07
7 - 9	2		18	0.11
9 - 11	2		15	0.13
11 - 13	2		2	1.00
13 - 13.25	0.25		100	0.00
13.25 - 16.25	3		100	0.03
16.25 - 19	2.75		13	0.21
19 - 21	2		24	0.08
21 - 24.3	3.3		100	0.03
24.3 - 30.7	6.4		100	0.06
30.7 - 36	5.3		41	0.13
36 - 41	5		58	0.09
41 - 46	5		53	0.09
46 - 50.9	4.9		100	0.05
50.9 - 54.8	3.9		100	0.04
54.8 - 59.3	4.5		100	0.05
59.3 - 64.3	5		100	0.05
64.3 - 69.7	5.4		100	0.05
69.7 - 74.3	4.6		100	0.05
74.3 - 78.2	3.9		100	0.04
78.2 - 100	21.8		100	0.22
Total	100.00			2.66

N Bar= 38 D 15<N<50

N-Bar Calculation			Boring B17-2	
Depth	Di	Layer thickness	Raw SPT N	d/ni
0 - 3	3	3	90	0.03
3 - 5	2	2	10	0.20
5 - 7	2	5	5	0.40
7 - 9	2	2	3	0.67
9 - 11	2	2	14	0.14
11 - 16	5	5	96	0.05
16 - 19	3	3	100	0.03
19 - 22	3	3	53	0.06
22 - 26	4	4	27	0.15
26 - 31	5	5	13	0.38
31 - 36	5	5	120	0.04
36 - 41	5	5	35	0.14
41 - 46	5	5	25	0.20
46 - 51	5	5	30	0.17
51 - 56	5	5	130	0.04
56 - 61	5	5	99	0.05
61 - 100	39	39	100	0.39
Total		100.00		3.14

N Bar= 32 D 15<N<50

N-Bar Calculation			Boring B17-3	
Depth	Di	Layer thickness	Raw SPT N	d/ni
0 - 2	2	2	21	0.10
2 - 4	2	2	11	0.18
4 - 6	2	2	27	0.07
6 - 8	2	2	43	0.05
8 - 10	2	2	58	0.03
10 - 12	2	2	21	0.10
12 - 14	2	6	6	0.33
14 - 16	2	2	40	0.05
16 - 18	2	2	59	0.03
18 - 20	2	2	31	0.06
20 - 26	6	6	82	0.07
26 - 31	5	5	62	0.08
31 - 36	5	5	13	0.38
36 - 40	4	4	86	0.05
40 - 40.6	0.6	0.6	83	0.01
40.6 - 44.5	3.9	3.9	100	0.04
44.5 - 48.1	3.6	3.6	50	0.07
48.1 - 100	51.9	51.9	100	0.52
Total		100.00		2.23

N Bar= 45 D 15<N<50

20.4.2 $\bar{N}_s$, Average Field Standard Penetration Resistance and $\bar{N}_{60}$, Average Standard Penetration Resistance for Cohesionless Soil Layers:
 $\bar{N}$ and $\bar{N}_{60}$ shall be determined in accordance with the following formulas:

$$\bar{N} = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n N_i} \quad (20.4-2)$$

where N_i and d_i in Eq. 20.4-2 are for cohesionless soil, cohesive soil, and rock layers.

$$\bar{N}_{60} = \frac{d_s}{\sum_{i=1}^n \frac{d_i}{N_i}} \quad (20.4-3)$$

where N_i and d_i in Eq. 20.4-3 are for cohesionless soil layers only and $\sum_{i=1}^n d_i = d_s$, where d_s is the total thickness of cohesionless soil layers in the top 100 ft (30 m). N_i is the standard penetration resistance (ASTM D1586) not to exceed 100 blows/ft (305 blows/m) as directly measured in the field without corrections. Where refusal is met for a rock layer, N_i shall be taken as 100 blows/ft (305 blows/m).

Table 20.3-1 Site Classification			
Site Class	$\bar{V}_s$	$\bar{N}$ or $\bar{N}_{60}$	$\bar{q}_s$
A. Hard rock	>5,000 ft/s	NA	NA
B. Rock	2,500 to 5,000 ft/s	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
Any profile with more than 10 ft of soil having the following characteristics: — Plasticity index $PI > 20$; — Moisture content $w \geq 40\%$; — Undrained shear strength $\bar{q}_s < 500$ psf			
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SE: 1 ft/s = 0.3048 m/s; 1 ft/ft² = 0.0479 kN/m².

Cranberry Highway/Mill Pond Dam over the Agawam River

Earthquake Loads



Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the [U.S. Seismic Design Maps web tools](#) (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

^ Input

Edition

Dynamic: Conterminous U.S. 2014 (v4.1. ▼

Spectral Period

Peak ground acceleration ▼

Latitude

Decimal degrees

41.763

Time Horizon

Return period in years

2475

Longitude

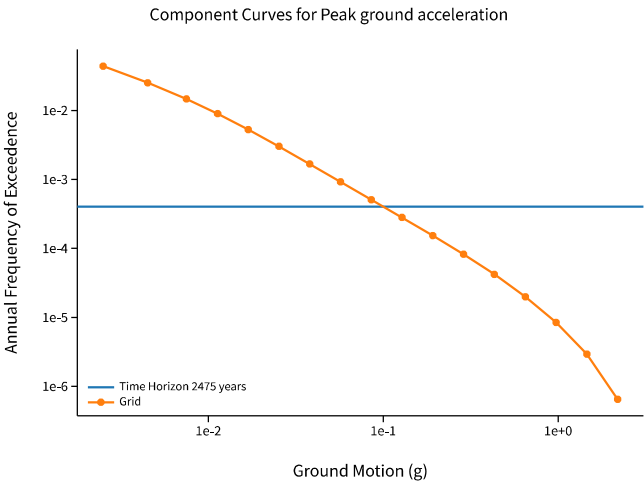
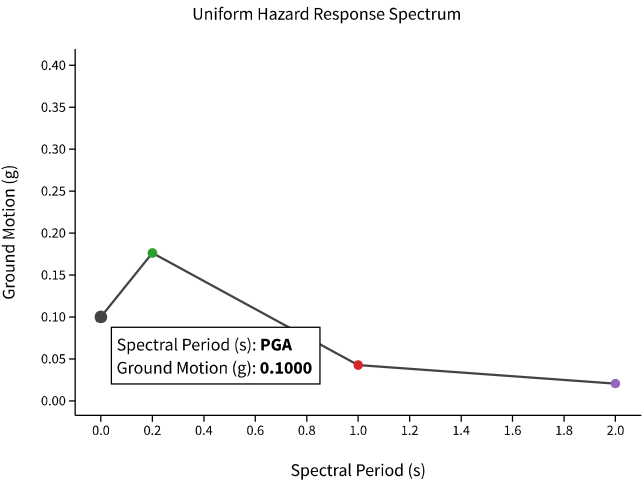
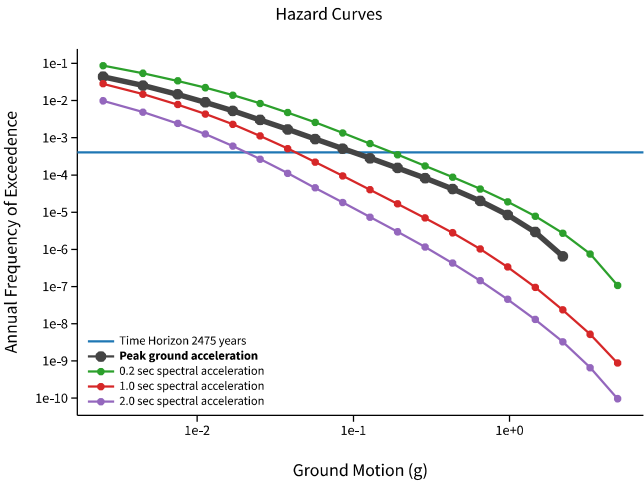
Decimal degrees, negative values for western longitudes

-70.676

Site Class

760 m/s (B/C boundary) ▼

^ Hazard Curve

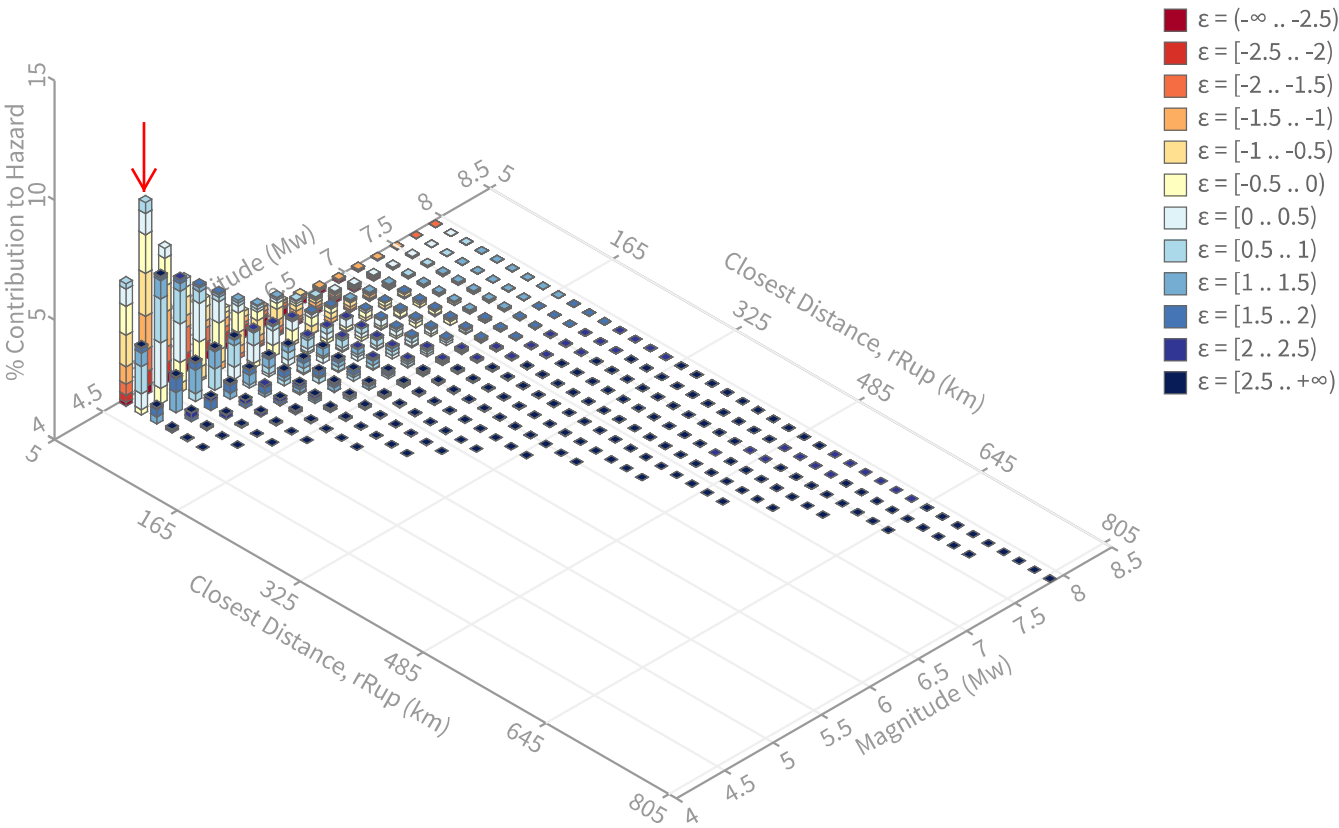


[View Raw Data](#)

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr⁻¹
PGA ground motion: 0.099981759 g

Recovered targets

Return period: 2474.5831 yrs
Exceedance rate: 0.00040410848 yr⁻¹

Totals

Binned: 100 %
Residual: 0 %
Trace: 1.5 %

Mean (for all sources)

r: 49.62 km
m: 5.61
ε₀: -0.07 σ

Mode (largest r-m bin)

r: 13.74 km
m: 4.9
ε₀: -0.89 σ
Contribution: 8.04 %

Mode (largest ε₀ bin)

r: 26.89 km
m: 4.9
ε₀: 0.26 σ
Contribution: 1.9 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set  Source	Type	r	m	ϵ_0	lon	lat	az	%
USGS Fixed Smoothing Zone 2 (opt)	Grid							29.52
PointSourceFinite: -70.676, 41.920		17.91	5.23	-0.73	70.676°W	41.920°N	0.00	4.86
PointSourceFinite: -70.676, 42.010		27.40	5.35	-0.12	70.676°W	42.010°N	0.00	2.84
PointSourceFinite: -70.676, 41.830		8.90	5.15	-1.84	70.676°W	41.830°N	0.00	2.33
PointSourceFinite: -70.676, 41.965		22.64	5.29	-0.38	70.676°W	41.965°N	0.00	2.13
PointSourceFinite: -70.676, 42.055		32.17	5.42	0.08	70.676°W	42.055°N	0.00	2.07
PointSourceFinite: -70.676, 42.100		36.92	5.50	0.24	70.676°W	42.100°N	0.00	1.89
PointSourceFinite: -70.676, 41.875		13.26	5.18	-1.21	70.676°W	41.875°N	0.00	1.51
PointSourceFinite: -70.676, 42.145		41.65	5.57	0.36	70.676°W	42.145°N	0.00	1.50
PointSourceFinite: -70.676, 42.235		51.06	5.73	0.54	70.676°W	42.235°N	0.00	1.14
PointSourceFinite: -70.676, 42.190		46.36	5.65	0.46	70.676°W	42.190°N	0.00	1.10
SSCn Fixed Smoothing Zone 4 (opt)	Grid							29.47
PointSourceFinite: -70.676, 41.920		17.91	5.23	-0.73	70.676°W	41.920°N	0.00	4.86
PointSourceFinite: -70.676, 42.010		27.40	5.35	-0.12	70.676°W	42.010°N	0.00	2.84
PointSourceFinite: -70.676, 41.830		8.90	5.15	-1.84	70.676°W	41.830°N	0.00	2.33
PointSourceFinite: -70.676, 41.965		22.64	5.29	-0.38	70.676°W	41.965°N	0.00	2.13
PointSourceFinite: -70.676, 42.055		32.17	5.42	0.08	70.676°W	42.055°N	0.00	2.07
PointSourceFinite: -70.676, 42.100		36.92	5.50	0.24	70.676°W	42.100°N	0.00	1.89
PointSourceFinite: -70.676, 41.875		13.26	5.18	-1.21	70.676°W	41.875°N	0.00	1.51
PointSourceFinite: -70.676, 42.145		41.65	5.57	0.36	70.676°W	42.145°N	0.00	1.50
PointSourceFinite: -70.676, 42.235		51.06	5.73	0.54	70.676°W	42.235°N	0.00	1.14
PointSourceFinite: -70.676, 42.190		46.36	5.65	0.46	70.676°W	42.190°N	0.00	1.10
USGS Adaptive Smoothing Zone 2 (opt)	Grid							19.43
PointSourceFinite: -70.676, 41.920		17.91	5.23	-0.73	70.676°W	41.920°N	0.00	2.72
PointSourceFinite: -70.676, 42.010		27.40	5.35	-0.12	70.676°W	42.010°N	0.00	1.63
PointSourceFinite: -70.676, 41.830		8.90	5.15	-1.84	70.676°W	41.830°N	0.00	1.36
PointSourceFinite: -70.676, 42.055		32.17	5.42	0.08	70.676°W	42.055°N	0.00	1.26
PointSourceFinite: -70.676, 41.965		22.64	5.29	-0.38	70.676°W	41.965°N	0.00	1.24
PointSourceFinite: -70.676, 42.100		36.92	5.50	0.24	70.676°W	42.100°N	0.00	1.05
SSCn Adaptive Smoothing Zone 4 (opt)	Grid							19.40
PointSourceFinite: -70.676, 41.920		17.91	5.23	-0.73	70.676°W	41.920°N	0.00	2.72
PointSourceFinite: -70.676, 42.010		27.40	5.35	-0.12	70.676°W	42.010°N	0.00	1.63
PointSourceFinite: -70.676, 41.830		8.90	5.15	-1.84	70.676°W	41.830°N	0.00	1.36
PointSourceFinite: -70.676, 42.055		32.17	5.42	0.08	70.676°W	42.055°N	0.00	1.26
PointSourceFinite: -70.676, 41.965		22.64	5.29	-0.38	70.676°W	41.965°N	0.00	1.24
PointSourceFinite: -70.676, 42.100		36.92	5.50	0.24	70.676°W	42.100°N	0.00	1.05

Cranberry Highway/Mill Pond Dam over the Agawam River

Liquefaction Analysis





Project: Mill Pond Dam
Subject: Modified SPT Development
Computations By: MLP
Checked By: DMM

Project No.: 14199.14
Date: 4/18/2018
Date: 7/20/2018

PARAMETERS

MOIST SOIL UNIT WEIGHT= 110 PCF
SATURATED SOIL UNIT WEIGHT= 125 PCF
NO SURCHARGE ASSUMED
ACCELERATION COEFFICIENT= 0.145 g
AUTO HAMMER BB17-1 and BB17-2
SAFETY HAMMER BB17-3
References Stress (Pa) tsf 1
E.O.E. IS USED

Results of Sieve Analysis (3)			
Fines Content (%)		13	Fill
α	$FC \leq 5\%$	$5\% \leq FC \leq 35\%$	$FC > 35\%$
β	0	1.8884	5
	1	0.943	1.2
Fines Content (%)		2	Glacial Deposits
α	$FC \leq 5\%$	$5\% \leq FC \leq 35\%$	$FC > 35\%$
β	0	0.00000	5
	1	0.00000	1.2
TanPhi (deg) (1)		ϕ	34
TanPhi (deg) (1)		ϕ	0.675
TanPhi (deg) (1)		ϕ	36
TanPhi (deg) (1)		ϕ	0.727
Earthquake Magnitude		4.9 g	Ref. 8, see input for selected area
Magnitude Scaling Factor		2.97341049	Ref. 5, p 112, Eq. 11, n=2.56
ϕ		34	

(Demand) (Capacity)

BORING & SAMPLE	DEPTH (FT)	DEPTH (m)	G.W. DEPTH (FT)	OVERBURDEN STRESS, TSF		RAW SPT #	N _r (2, Tbl 2.2)	N _H (3, Tbl 2.2)	N ₆₀ (4, eq 2.6)	Ave N ₆₀ LAYER _N	CN (10, eq 2.9)	(N) ₆₀ (4, eq 2.10)	N _{1,60cs} (7, pg 208, eq 3)	Stress Reduction Factor (Rd) (5, pg 3, eq 3)	CSR _{mr/5} (5, pg 55, eq 3)	CRR(9, pg 820, eq 4)	Magnitude Scaling Factor (Ref. 5 see above)	C _σ	K _σ Overburden Correction Factor (5)	Ts (Static Shear Stress, tsf)	α (Static Stress Ratio)	K _α (Correction factor for initial static shear stress, Inclined ground) (7)	F.S.	COMMENTS (Not Susceptible to Liquefaction if FS>1)
				TOTAL	EFFECT.																			
BB-1-S-1	2	0.61	18	0.1100	0.1100	100	0.75	80	100		1.7	170	162	1.00	0.09	1.189	2.973	-0.074	0.8	0.07	0.67	1.0	31.5	OK
BB-1-S-2	4	1.22	18	0.2200	0.2200	50	0.75	80	50		1.7	85	82	0.99	0.09	0.582	2.973	-0.238	0.6	0.15	0.67	1.0	11.8	OK
BB-1-S-3	6	1.83	18	0.3300	0.3300	27	0.75	80	27		1.7	46	45	0.99	0.09	0.240	2.973	0.300	1.1	0.22	0.67	1.0	8.4	OK
BB-1-S-4	8	2.44	18	0.4400	0.4400	18	0.75	80	18		1.5	27	27	0.98	0.09	0.352	2.973	0.181	1.1	0.30	0.67	1.0	12.4	OK
BB-1-S-5	10	3.05	18	0.5500	0.5500	15	0.75	80	15		1.3	20	21	0.98	0.09	0.228	2.973	0.138	1.1	0.37	0.67	1.0	7.9	OK
BB-1-S-6	12	3.66	18	0.6600	0.6600	2	0.75	80	2		1.2	2	4	0.97	0.09	0.066	2.973	0.073	1.0	0.45	0.67	1.0	2.2	OK
BB-1-S-7	13	3.97	18	0.7150	0.7150	100	0.85	80	113		1.2	134	128	0.97	0.09	0.935	2.973	-0.100	1.0	0.48	0.67	1.0	29.3	OK
BB-1-S-8	15	4.58	18	0.8250	0.8250	100	0.85	80	113		1.1	125	120	0.97	0.09	0.869	2.973	-0.111	1.0	0.56	0.67	1.0	27.7	OK
BB-1-S-9	18	5.49	18	0.9900	0.9900	13	0.85	80	15		1.0	15	15	0.96	0.09	0.158	2.973	0.110	1.0	0.72	0.73	1.0	5.2	OK
BB-1-S-10	20	6.10	18	1.1000	1.0526	24	0.85	80	27		1.0	27	27	0.96	0.09	0.325	2.973	0.173	1.0	0.76	0.73	1.0	10.2	OK
BB-1-S-11	24	7.32	18	1.3200	1.1778	100	0.95	80	127		0.9	117	117	0.94	0.10	0.847	2.973	-0.116	1.0	0.86	0.73	1.0	25.7	OK
BB-1-S-12	30	9.15	18	1.6500	1.3656	100	1	80	133		0.9	114	114	0.92	0.10	0.828	2.973	-0.120	1.0	0.99	0.73	1.0	24.4	OK
BB-1-S-13	35	10.68	18	1.9250	1.5221	41	1	80	55		0.8	44	44	0.89	0.11	0.226	2.973	0.300	0.9	1.11	0.73	1.0	5.6	OK
BB-1-S-14	40	12.20	18	2.2000	1.6786	58	1	80	77		0.8	60	60	0.85	0.10	0.398	2.973	-1.249	1.1	1.22	0.73	1.0	12.4	OK
BB-1-S-15	45	13.73	18	2.4750	1.8351	53	1	80	71		0.7	52	52	0.80	0.10	0.327	2.973	0.300	0.8	1.33	0.73	1.0	7.8	OK
BB-1-S-16	50	15.25	18	2.7500	1.9916	100	1	80	133		0.7	94	94	0.75	0.10	0.678	2.973	-0.170	1.1	1.45	0.73	1.0	22.7	OK
BB-1-S-17	54	16.47	18	2.9700	2.1168	100	1	80	133		0.7	92	92	0.71	0.09	0.657	2.973	-0.181	1.1	1.54	0.73	1.0	22.9	OK
BB-1-S-18	59	18.00	18	3.2450	2.2733	100	1	80	133		0.7	88	88	0.66	0.09	0.632	2.973	-0.197	1.1	1.65	0.73	1.0	23.1	OK
BB-1-S-19	64	19.52	18	3.5200	2.4298	100	1	80	133		0.6	86	86	0.63	0.09	0.609	2.973	-0.213	1.1	1.77	0.73	1.0	23.3	OK
BB-1-S-20	69	21.05	18	3.7950	2.5863	100	1	80	133		0.6	83	83	0.59	0.08	0.589	2.973	-0.232	1.1	1.88	0.73	1.0	23.4	OK
BB-1-S-21	74	22.57	18	4.0700	2.7428	100	1	80	133		0.6	81	81	0.57	0.08	0.570	2.973	-0.251	1.1	1.99	0.73	1.0	23.4	OK
BB-1-S-22	77	23.49	18	4.2350	2.8367	100	1	80	133		0.6	79	79	0.56	0.08	0.559	2.973	-0.264	1.1	2.06	0.73	1.0	23.3	OK
BB-2-S-1	2	0.61	10	0.1100	0.1100	90	0.75	80	90		1.7	153	146	1.00	0.09	1.069	2.973	-0.084	0.8	0.07	0.67	1.0	27.6	OK
BB-2-S-2	4	1.22	10	0.2200	0.2200	10	0.75	80	10		1.7	17	18	0.99	0.09	0.191	2.973	0.123	1.1	0.15	0.67	1.0	6.7	OK
BB-2-S-3	6	1.83	10	0.3300	0.3300	5	0.75	80	5		1.7	9	10	0.99	0.09	0.112	2.973	0.092	1.1	0.22	0.67	1.0	3.9	OK
BB-2-S-4	8	2.44	10	0.4400	0.4400	3	0.75	80	3		1.5	5	6	0.98	0.09	0.081	2.973	0.080	1.1	0.30	0.67	1.0	2.8	OK
BB-2-S-5	10	3.05	10	0.5500	0.5500	14	0.75	80	14		1.3	19	20	0.98	0.09	0.212	2.973	0.132	1.1	0.37	0.67	1.0	7.4	OK
BB-2-S-6	15	4.58	10	0.8250	0.7065	96	0.85	80	109		1.2	129	124	0.97	0.11	0.902	2.973	-0.105	1.0	0.48	0.67	1.0	24.2	OK
BB-2-S-7	19	5.80	10	1.0450	0.8317	100	0.85	80	113		1.1	124	124	0.96	0.11	0.904	2.973	-0.105	1.0	0.56	0.67	1.0	23.2	OK
BB-2-S-8	21	6.41	10	1.1550	0.8943	53	0.95	80	67		1.1	71	71	0.95	0.12	0.494	2.973	-0.387	1.0	0.60	0.67	1.0	12.1	OK
BB-2-S-9	25	7.63	10	1.3750	1.0195	27	0.95	80	34		1.0	34	34	0.94	0.12	8.019	2.973	0.246	1.0	0.74	0.73	1.0	198.3	OK
BB-2-S-10	30	9.15	10	1.6500	1.1760	13	1	80	17		0.9	16	16	0.92	0.12	0.810	2.973	0.115	1.0	0.85	0.73	1.0	4.1	OK
BB-2-S-11	35	10.68	10	1.9250	1.3325	100	1	80	133		0.9	116	116	0.89	0.12	0.838	2.973	-0.118	1.0	0.97	0.73	1.0	21.3	OK
BB-2-S-12	40	12.20	10	2.2000	1.4890	35	1	80	47		0.8	38	38	0.85	0.12	0.043	2.973	0.300	0.9	1.08	0.73	1.0	1.0	OK
BB-2-S-13	45	13.73	10	2.4750	1.6455	25	1	80	33		0.8	26	26	0.80	0.11	0.313	2.973	0.169	0.9	1.20	0.73	1.0	7.5	OK
BB-2-S-14	50	15.25	10	2.7500	1.8020	30	1	80	40		0.7	30	30	0.75	0.11	0.454	2.973	0.201	0.9	1.31	0.73	1.0	11.0	OK
BB-2-S-15	55	16.78	10	3.0250	1.9585	100	1	80	133		0.7	95	95	0.70	0.10	0.684	2.973	-0.167	1.1	1.42	0.73	1.0	22.0	OK
BB-2-S-16	60	18.30	10	3.3000	2.1150	99	1	80	132		0.7	91	91	0.66	0.10	0.650	2.973	-0.185	1.1	1.54	0.73	1.0	22.0	OK



Project: Mill Pond Dam
Subject: Modified SPT Development
Computations By: MLP
Checked By: DMM

Project No.: 14199.14
Date: 4/18/2018
Date: 7/20/2018

PARAMETERS									
MOIST SOIL UNIT WEIGHT=		110 PCF		ASSUMED					
SATURATED SOIL UNIT WEIGHT=		125 PCF							
NO SURCHARGE ASSUMED									
ACCELERATION COEFFICIENT=		0.145 g							
AUTO HAMMER BB17-1 and BB17-2									
SAFETY HAMMER BB17-3									
References Stress (Pa) tsf		1							
WHEN NO WATER LEVEL IS AVAILABLE, A VALUE DEEPER THAN THE E.O.E. IS USED									
		Fines Content (%)		13		Fill			
		FC ≤ 5%		5% ≤ FC ≤ 35%		FC>35%			
α		0		1.8884		5			
β		1		0.943		1.2			
		Fines Content (%)		2		Glacial Deposits			
FC ≤ 5%		5% ≤ FC ≤ 35%		FC>35%					
α		0		0.00000		5			
β		1		0.00000		1.2			

BORING & SAMPLE	DEPTH (FT)	DEPTH (m)	G.W. DEPTH (FT)	OVERBURDEN STRESS, TSF		RAW SPT #	N _r (2, Tbl 2.2)	N _H (3, Tbl 2.2)	N ₆₀ (4, eq 2.6)	Ave N ₆₀ LAYER _N	CN (10, eq 2.9)	(N) ₆₀ (4, eq 2.10)	N _{1,60cs} (7, pg 208, eq 3)	Stress Reduction Factor (Rd) (5, pg 3, eq 3)	CSR _{n7.5} (5, pg 55, eq 3)	CRR(9, pg 820, eq 4)	Magnitude Scaling Factor (Ref. 5 see above)	C _σ	K _σ Overburden Correction Factor (5)	Ts (Static Shear Stress, tsf)	α (Static Stress Ratio)	K _α (Correction factor for initial static shear stress, Inclined ground) (7)	F.S.	COMMENTS (Not Susceptible to Liquefaction if FS≥1)
				TOTAL	EFFECT.																			
BB-3-S-1	1	0.31	0	0.0550	0.0313	21	0.75	60	16	Pond Muck	1.7	26.775	26.775	1.00	0.17	0.332	2.973	0.175	1.1	0.02	0.73	1.0	6.6	OK
BB-3-S-2	3	0.92	0	0.1650	0.0939	11	0.75	60	8		1.7	14.025	14.025	0.99	0.16	0.150	2.973	0.107	1.1	0.07	0.73	1.0	3.0	OK
BB-3-S-3	5	1.53	0	0.2750	0.1565	27	0.75	60	20		1.7	34.425	34.425	0.99	0.16	-2.103	2.973	0.254	1.1	0.11	0.73	1.0	-41.9	OK
BB-3-S-4	7	2.14	0	0.3850	0.2191	43	0.75	60	32		1.7	54.825	54.825	0.99	0.16	0.353	2.973	0.300	1.1	0.16	0.73	1.0	7.1	OK
BB-3-S-5	9	2.75	0	0.4950	0.2817	58	0.75	60	44		1.7	73.950	73.950	0.98	0.16	0.518	2.973	-0.330	0.6	0.20	0.73	1.0	5.5	OK
BB-3-S-6	11	3.36	0	0.6050	0.3443	21	0.75	60	16		1.7	26.775	26.775	0.98	0.16	0.332	2.973	0.175	1.1	0.25	0.73	1.0	6.7	OK
BB-3-S-7	13	3.97	0	0.7150	0.4069	6	0.85	60	5		1.6	7.995	7.995	0.97	0.16	0.096	2.973	0.086	1.1	0.30	0.73	1.0	1.9	OK
BB-3-S-8	15	4.58	0	0.8250	0.4695	40	0.85	60	34	Silty Sand Ave. 35	1.5	49.620	49.620	0.97	0.16	0.299	2.973	0.300	1.1	0.34	0.73	1.0	6.1	OK
BB-3-S-9	17	5.19	0	0.9350	0.5321	59	0.85	60	50		1.4	68.750	68.750	0.96	0.16	0.476	2.973	-0.446	0.7	0.39	0.73	1.0	6.4	OK
BB-3-S-10	19	5.80	0	1.0450	0.5947	31	0.85	60	26		1.3	34.169	34.169	0.96	0.16	-5.670	2.973	0.250	1.1	0.43	0.73	1.0	-116.8	OK
BB-3-S-11	25	7.63	0	1.3750	0.7825	82	0.95	60	78		1.1	88.063	88.063	0.94	0.16	0.629	2.973	-0.199	1.0	0.57	0.73	1.0	11.4	OK
BB-3-S-12	30	9.15	0	1.6500	0.9390	62	1	60	62		1.0	63.982	63.982	0.92	0.15	0.436	2.973	-0.668	1.0	0.68	0.73	1.0	8.1	OK
BB-3-S-13	35	10.68	0	1.9250	1.0955	13	1	60	13		1.0	12.420	12.420	0.89	0.15	0.135	2.973	0.101	1.0	0.80	0.73	1.0	2.7	OK
BB-3-S-14	39	11.90	0	2.1450	1.2207	100	1	60	100		0.9	90.510	90.510	0.86	0.14	0.648	2.973	-0.187	1.0	0.89	0.73	1.0	14.1	OK
BB-3-S-15	40	12.20	0	2.2000	1.2520	100	1	60	100	Glacial Till	0.9	89.371	89.371	0.85	0.14	0.639	2.973	-0.192	1.0	0.91	0.73	1.0	14.1	OK
BB-3-S-16	44	13.42	0	2.4200	1.3772	100	1	60	100	Ave. 100	0.9	85.212	85.212	0.81	0.13	0.607	2.973	-0.216	1.1	1.00	0.73	1.0	14.4	OK
BB-3-S-17	48	14.64	0	2.6400	1.5024	100	1	60	100		0.8	81.584	81.584	0.77	0.13	0.578	2.973	-0.242	1.1	1.09	0.73	1.0	14.8	OK

References

- (1) "An Introduction to Geotechnical Engineering Second Edition" Holtz, Robert D., Kovacs, William D., Sheahan, Thomas G., 2011 Not Used
- (2), (3), (4) Das Principles of Foundation Engineering, 5th Edition.
- (5) NCEER 1997 Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils
- (6) Youd et al 2001
- (7) Tech Report NCEER 97-0022
- (8) <https://earthquake.usgs.gov/hazards/interactive/>
- (9) Liquefaction Resistance of Soils: Summary Report from 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation... DOI: 10.1061/(ASCE)1090-0241(2001)127:10(817)
- (10) Report No. UCD/CGM-10/02 SPT-Based Liquefaction Triggering Procedures

Cranberry Highway/Mill Pond Dam over the Agawam River

Seismic Parameters





Proposal No. 606352-136278
Project: Mill Pond Dam Project No.: 14199.14
Subject: Seismic Parameter Development
Computations By: MLP Date: 12/2021
Checked By: _____ Date: _____

DESCRIPTION:

To determine Seismic design criteria for the determination of maximum considered earthquake and design spectral response accelerations.

CALCULATIONS:

PGA = 0.079	USGS Seismic Hazard Map for the 2500-yr event
S _s = 0.131	USGS Seismic Hazard Map for the 2500-yr event
S ₁ = 0.047	USGS Seismic Hazard Map for the 2500-yr event

For this site, N<50; Therefore, Site Class Profile = D (Stiff Soil)

Adjusted Maximum considered earthquake response accelerations considering site Class D

For S _s = 0.131; for Site Class D, F _a = 1.6	AASHTO, Table 3.4.2.3-1 (1)
For S ₁ = 0.047; for Site Class D, F _v = 2.4	AASHTO, Table 3.4.2.3-2 (2)

Design Spectral Response Parameters

S _{DS} = F _a S _s = 1.6 x 0.131 = 0.21	AASHTO, Table 3.4.2.3-1
S _{D1} = F _v S ₁ = 2.4 x 0.047 = 0.113	AASHTO, Table 3.4.2.3-1

Peak Ground Acceleration for Liquefaction Potential

A _s = F _{PGA} x PGA = 1.6 x 0.079 = 0.126	Equation 3.4.1-1
---	------------------

Seismic zone 1

REFERENCES:

1. 2020 AASHTO Guide Specifications for LRFD Bridge Design.
2. DAS 1990. "Principles of Foundation Engineering, 7th Edition". 1990.
3. MassDOT LRFD Bridge Manual



Project: Mill Pond Dam

Subject: Seismic Earth Pressure Coefficient (Kae)

Computations By: MLP

Checked By:

Prof. No.: 14199.12

Date: 5/31/2018

Date:

Description: Estimate the seismic active earth pressure coefficient (Kae) to use in seismic stabilities analyses for retaining walls in accordance with the Mononobe-Okabe

Parameters:

ϕ	34 deg	Soil internal friction angle
β	10 deg	Slope of wall back face from vertical
δ	20 deg	Wall friction angle for formed concrete against Silty Sand or Clayey Fine to Medium Sand, between 19 to 24°, use 20°
i	0 deg	Slope of backfill from horizontal

A	0.079	Effective peak acceleration
Kh	0.0395	A/2 Horizontal acceleration
Kv	0	Vertical acceleration

$$\theta = \text{atan}\left(\frac{K_h}{1 - K_v}\right)$$

θ 2.26 deg

$$K_{ae} = \frac{\cos(\phi - \theta - \beta)^2}{\cos(\phi)^2 * \cos(\beta)^2 * \cos(\delta - \beta + \theta) * \left[1 + \sqrt{\left(\frac{\sin(\phi - \theta - i)}{\cos(\delta + \beta + \theta) * \cos(i - \beta)}\right)^2}\right]}$$

Kae	0.862831	0.829038	0.969846	0.977187	0.694658	0.526036	0.845616	0.984808
	0.40							

Cranberry Highway/Mill Pond Dam over the Agawam River

Bearing Capacity





Calculation Cover Sheet

Project #: 14199.14
 Project: **Mill Pond Dam**
 Subject/Task: Bearing Capacity of Footing, Case 1: Mat Foundation under Roadway/Bridge
 Status: Draft
 Date: 7/23/2018

Objective: Calculate bearing capacity for soil using the AASHTO (2014) manual

FOUNDATION DIMENSIONS

$B := 15$ ft width of foundation (left to right)
 $L := 60$ ft length of foundation, abutment width
 $D_f := 5$ ft depth of foundation measured from the ground surface

LOADING

$P_v := 200$ kip Vertical line load (assumed)
 $\theta := 0$ deg kip Projection direction of load relative to length L
 $\beta := 10$ deg Directional load relative to vertical
 $P_h := P_v \cdot \tan(\beta) = 35.3$ kip Horizontal load

SOIL PROPERTIES

$\gamma_w := 62.4$ pcf Density of water
 $D_w := 0$ ft Depth to ground water from the ground surface, assume saturated condition
 $\gamma_1 := 132$ pcf Unit weight of soil below foundation base
 $\gamma_{sat1} := \gamma_1 - \gamma_w = 69.6$ pcf Effective unit weight of Sand and Silt
 $H_1 := 16$ ft Height of soil layer above the foundation
 $\phi_1 := 38$ deg Internal angle of friction for Sand and Silt
 $c_1 := 0.00$ Cohesion of soil
 $N_1 := 61$ Assume N value of bearing soils based on boring logs
 $\phi_q := 35$ deg Internal angle of friction for Structural Fill above the bearing soils
 $\gamma_q := 130$ pcf Saturated unit weight of Structural Fill above bearing soil.
 $\gamma_{bouyq} := \gamma_q - \gamma_w = 67.6$ pcf Effective unit weight of Structural Fill

CHECK BEARING CAPACITY AASHTO LRFD Bridge Design Specifications

Bearing Capacity Factors

$$N_{qaashto} := \tan \left(45 \text{ deg} + \frac{\phi_q}{2} \right)^2 \cdot e^{\pi \cdot \tan(\phi_q)} \quad N_{qaashto} = 33.3$$

$$N_{caashto} := (N_{qaashto} - 1) \cdot \cot(\phi_1) \quad N_{caashto} = 41.3$$

$$N_{\gamma aashto} := 2 \cdot (N_{qaashto} + 1) \cdot \tan(\phi_1) \quad N_{\gamma aashto} = 53.6$$

Shape factors:

Table 10.6.3.1.2a-3—Shape Correction Factors s_c , s_γ , s_q

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

$$S_{qsaashto} := 1 + \left(\frac{B}{L} \cdot \tan(\phi_q) \right) \quad S_{qsaashto} = 1.2$$

$$S_{csaashto} := 1 + \left(\frac{B}{L} \cdot \frac{N_{qaashto}}{N_{caashto}} \right) \quad S_{csaashto} = 1.2$$

$$S_{\gamma saashto} := 1 - 0.4 \cdot \frac{B}{L} \quad S_{\gamma saashto} = 0.9$$

Depth factors: Depth factor should only be used when the soil above the footing elevation are as competent as the soils beneath the footing level; otherwise 1 is used.

$$\phi_1 = 38 \text{ deg}$$

$$D_{qdaashto} := 1$$

$$D_{cdaashto} := 1$$

$$D_{\gamma daashto} := 1$$

Water factors:Assume saturated condition $D_w = 0$ **Table 10.6.3.1.2a-2—Coefficients C_{wq} and C_{wy} for Various Groundwater Depths**

D_w	C_{wq}	C_{wy}
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

$$C_{wqaashto} := 0.5$$

$$C_{w\gamma aashto} := 0.5$$

$$C_{wcaashto} := \text{if} \left((D_f + B) < D_w, 1.0, 1.0 \right)$$

$$C_{wcaashto} = 1$$

Inclination factors:

$$n := \frac{\left(2 + \frac{L}{B}\right)}{\left(1 + \frac{L}{B}\right)} \cdot \cos(\theta)^2 + \frac{\left(2 + \frac{B}{L}\right)}{\left(1 + \frac{B}{L}\right)} \cdot \sin(\theta)^2 = 1.2$$

$$i_{\gamma aashto} := \left(1 - \frac{P_h}{P_v + c_1 \cdot B \cdot L \cdot \left(\frac{1}{\tan(\phi_1)} \right)} \right)^{n+1} = 0.65$$

$$i_{qaashto} := \left(1 - \frac{P_h}{(P_v + c_1 \cdot B \cdot L \cdot \cot(\phi_q))} \right)^n = 0.79$$

$$i_{caashto} := i_{qaashto} - \frac{(1 - i_{qaashto})}{(N_{qaashto} - 1)} = 0.8$$

Correction factors:

$$N_{caashto} := N_{caashto} \cdot S_{csaashto} \cdot D_{cdaashto} \cdot i_{caashto} \cdot C_{wcaashto} = 39.03$$

$$N_{qaashto} := N_{qaashto} \cdot S_{qsaashto} \cdot D_{qdaashto} \cdot i_{qaashto} \cdot C_{wqaashto} = 15.5$$

$$N_{\gamma aashto} := N_{\gamma aashto} \cdot S_{\gamma saashto} \cdot D_{\gamma daashto} \cdot i_{\gamma aashto} \cdot C_{w\gamma aashto} = 15.7$$

Calculate Ultimate Bearing Capacity:

$$q_{ult.asshto} := c_1 \cdot N_{caashto} + \gamma_{bouyq} \cdot D_f \cdot N_{qaashto} + \frac{1}{2} \cdot \gamma_{sat1} \cdot B \cdot N_{\gamma aashto}$$

$$q_{ult.asshto} = 13454 \quad psf$$

$$resistance.factor := 0.45 \quad \text{AASHTO Table 10.5.5.2.2-1 Page 1244}$$

$$q_{all.asshto} := q_{ult.asshto} \cdot resistance.factor = 6054 \quad psf$$



Calculation Cover Sheet

Project #: 14199.14
 Project: **Mill Pond Dam**
 Subject/Task: Bearing Capacity of Footing, Case 1: Mat Foundation under Roadway/Bridge
 Status: Draft
 Date: 7/23/2018

Objective: Calculate bearing capacity for soil using the AASHTO (2014) manual

FOUNDATION DIMENSIONS

$B := 20$ ft width of foundation (left to right)
 $L := 60$ ft length of foundation, abutment width
 $D_f := 5$ ft depth of foundation measured from the ground surface

LOADING

$P_v := 200$ kip Vertical line load (assumed)
 $\theta := 0$ deg kip Projection direction of load relative to length L
 $\beta := 10$ deg Directional load relative to vertical
 $P_h := P_v \cdot \tan(\beta) = 35.3$ kip Horizontal load

SOIL PROPERTIES

$\gamma_w := 62.4$ pcf Density of water
 $D_w := 0$ ft Depth to ground water from the ground surface, assume saturated condition
 $\gamma_1 := 132$ pcf Unit weight of soil below foundation base
 $\gamma_{sat1} := \gamma_1 - \gamma_w = 69.6$ pcf Effective unit weight of Sand and Silt
 $H_1 := 16$ ft Height of soil layer above the foundation
 $\phi_1 := 38$ deg Internal angle of friction for Sand and Silt
 $c_1 := 0.00$ Cohesion of soil
 $N_1 := 61$ Assume N value of bearing soils based on boring logs
 $\phi_q := 35$ deg Internal angle of friction for Structural Fill above the bearing soils
 $\gamma_q := 130$ pcf Saturated unit weight of Structural Fill above bearing soil.
 $\gamma_{bouyq} := \gamma_q - \gamma_w = 67.6$ pcf Effective unit weight of Structural Fill

CHECK BEARING CAPACITY AASHTO LRFD Bridge Design Specifications

Bearing Capacity Factors

$$N_{qaashto} := \tan \left(45 \text{ deg} + \frac{\phi_q}{2} \right)^2 \cdot e^{\pi \cdot \tan(\phi_q)} \quad N_{qaashto} = 33.3$$

$$N_{caashto} := (N_{qaashto} - 1) \cdot \cot(\phi_1) \quad N_{caashto} = 41.3$$

$$N_{\gamma aashto} := 2 \cdot (N_{qaashto} + 1) \cdot \tan(\phi_1) \quad N_{\gamma aashto} = 53.6$$

Shape factors:

Table 10.6.3.1.2a-3—Shape Correction Factors s_c , s_γ , s_q

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

$$S_{qsaashto} := 1 + \left(\frac{B}{L} \cdot \tan(\phi_q) \right) \quad S_{qsaashto} = 1.2$$

$$S_{csaashto} := 1 + \left(\frac{B}{L} \cdot \frac{N_{qaashto}}{N_{caashto}} \right) \quad S_{csaashto} = 1.3$$

$$S_{\gamma saashto} := 1 - 0.4 \cdot \frac{B}{L} \quad S_{\gamma saashto} = 0.9$$

Depth factors: Depth factor should only be used when the soil above the footing elevation are as competent as the soils beneath the footing level; otherwise 1 is used.

$$\phi_1 = 38 \text{ deg}$$

$$D_{qdaashto} := 1$$

$$D_{cdaashto} := 1$$

$$D_{\gamma daashto} := 1$$

Water factors:Assume saturated condition $D_w = 0$ **Table 10.6.3.1.2a-2—Coefficients C_{wq} and C_{wy} for Various Groundwater Depths**

D_w	C_{wq}	C_{wy}
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

$$C_{wqaashto} := 0.5$$

$$C_{w\gamma aashto} := 0.5$$

$$C_{wcaashto} := \text{if} \left((D_f + B) < D_w, 1.0, 1.0 \right)$$

$$C_{wcaashto} = 1$$

Inclination factors:

$$n := \frac{\left(2 + \frac{L}{B}\right)}{\left(1 + \frac{L}{B}\right)} \cdot \cos(\theta)^2 + \left(\frac{\left(2 + \frac{B}{L}\right)}{\left(1 + \frac{B}{L}\right)} \cdot \sin(\theta)^2 \right) = 1.25$$

$$i_{\gamma aashto} := \left(1 - \frac{P_h}{P_v + c_1 \cdot B \cdot L \cdot \left(\frac{1}{\tan(\phi_1)} \right)} \right)^{n+1} = 0.65$$

$$i_{qaashto} := \left(1 - \frac{P_h}{(P_v + c_1 \cdot B \cdot L \cdot \cot(\phi_q))} \right)^n = 0.78$$

$$i_{caashto} := i_{qaashto} - \frac{(1 - i_{qaashto})}{(N_{qaashto} - 1)} = 0.8$$

Correction factors:

$$N_{caashto} := N_{caashto} \cdot S_{csaashto} \cdot D_{cdaashto} \cdot i_{caashto} \cdot C_{wcaashto} = 40.8$$

$$N_{qaashto} := N_{qaashto} \cdot S_{qsaashto} \cdot D_{qdaashto} \cdot i_{qaashto} \cdot C_{wqaashto} = 16.1$$

$$N_{\gamma aashto} := N_{\gamma aashto} \cdot S_{\gamma saashto} \cdot D_{\gamma daashto} \cdot i_{\gamma aashto} \cdot C_{w\gamma aashto} = 15$$

Calculate Ultimate Bearing Capacity:

$$q_{ult.asshto} := c_1 \cdot N_{caashto} + \gamma_{bouyq} \cdot D_f \cdot N_{qaashto} + \frac{1}{2} \cdot \gamma_{sat1} \cdot B \cdot N_{\gamma aashto}$$

$$q_{ult.asshto} = 15892 \quad psf$$

$$resistance.factor := 0.45 \quad \text{AASHTO Table 10.5.5.2.2-1 Page 1244}$$

$$q_{all.asshto} := q_{ult.asshto} \cdot resistance.factor = 7152 \quad psf$$



Calculation Cover Sheet

Project #: 14199.14
 Project: **Mill Pond Dam**
 Subject/Task: Bearing Capacity of Footing, Case 1: Mat Foundation under Roadway/Bridge
 Status: Draft
 Date: 7/23/2018

Objective: Calculate bearing capacity for soil using the AASHTO (2014) manual

FOUNDATION DIMENSIONS

$B := 25$ ft width of foundation (left to right)
 $L := 60$ ft length of foundation, abutment width
 $D_f := 5$ ft depth of foundation measured from the ground surface

LOADING

$P_v := 200$ kip Vertical line load (assumed)
 $\theta := 0$ deg kip Projection direction of load relative to length L
 $\beta := 10$ deg Directional load relative to vertical
 $P_h := P_v \cdot \tan(\beta) = 35.3$ kip Horizontal load

SOIL PROPERTIES

$\gamma_w := 62.4$ pcf Density of water
 $D_w := 0$ ft Depth to ground water from the ground surface, assume saturated condition
 $\gamma_1 := 132$ pcf Unit weight of soil below foundation base
 $\gamma_{sat1} := \gamma_1 - \gamma_w = 69.6$ pcf Effective unit weight of Sand and Silt
 $H_1 := 16$ ft Height of soil layer above the foundation
 $\phi_1 := 38$ deg Internal angle of friction for Sand and Silt
 $c_1 := 0.00$ Cohesion of soil
 $N_1 := 61$ Assume N value of bearing soils based on boring logs
 $\phi_q := 35$ deg Internal angle of friction for Structural Fill above the bearing soils
 $\gamma_q := 130$ pcf Saturated unit weight of Structural Fill above bearing soil.
 $\gamma_{bouyq} := \gamma_q - \gamma_w = 67.6$ pcf Effective unit weight of Structural Fill

CHECK BEARING CAPACITY AASHTO LRFD Bridge Design Specifications

Bearing Capacity Factors

$$N_{qaashto} := \tan \left(45 \text{ deg} + \frac{\phi_q}{2} \right)^2 \cdot e^{\pi \cdot \tan(\phi_q)} \quad N_{qaashto} = 33.3$$

$$N_{caashto} := (N_{qaashto} - 1) \cdot \cot(\phi_1) \quad N_{caashto} = 41.3$$

$$N_{\gamma aashto} := 2 \cdot (N_{qaashto} + 1) \cdot \tan(\phi_1) \quad N_{\gamma aashto} = 53.6$$

Shape factors:

Table 10.6.3.1.2a-3—Shape Correction Factors s_c , s_γ , s_q

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

$$S_{qsaashto} := 1 + \left(\frac{B}{L} \cdot \tan(\phi_q) \right) \quad S_{qsaashto} = 1.3$$

$$S_{csaashto} := 1 + \left(\frac{B}{L} \cdot \frac{N_{qaashto}}{N_{caashto}} \right) \quad S_{csaashto} = 1.3$$

$$S_{\gamma saashto} := 1 - 0.4 \cdot \frac{B}{L} \quad S_{\gamma saashto} = 0.8$$

Depth factors: Depth factor should only be used when the soil above the footing elevation are as competent as the soils beneath the footing level; otherwise 1 is used.

$$\phi_1 = 38 \text{ deg}$$

$$D_{qdaashto} := 1$$

$$D_{cdaashto} := 1$$

$$D_{\gamma daashto} := 1$$

Water factors:Assume saturated condition $D_w = 0$ **Table 10.6.3.1.2a-2—Coefficients C_{wq} and C_{wy} for Various Groundwater Depths**

D_w	C_{wq}	C_{wy}
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

$$C_{wqaashto} := 0.5$$

$$C_{w\gamma aashto} := 0.5$$

$$C_{wcaashto} := \text{if} \left((D_f + B) < D_w, 1.0, 1.0 \right)$$

$$C_{wcaashto} = 1$$

Inclination factors:

$$n := \frac{\left(2 + \frac{L}{B}\right)}{\left(1 + \frac{L}{B}\right)} \cdot \cos(\theta)^2 + \left(\frac{\left(2 + \frac{B}{L}\right)}{\left(1 + \frac{B}{L}\right)} \cdot \sin(\theta)^2 \right) = 1.29$$

$$i_{\gamma aashto} := \left(1 - \frac{P_h}{P_v + c_1 \cdot B \cdot L \cdot \left(\frac{1}{\tan(\phi_1)} \right)} \right)^{n+1} = 0.64$$

$$i_{qaashto} := \left(1 - \frac{P_h}{(P_v + c_1 \cdot B \cdot L \cdot \cot(\phi_q))} \right)^n = 0.78$$

$$i_{caashto} := i_{qaashto} - \frac{(1 - i_{qaashto})}{(N_{qaashto} - 1)} = 0.8$$

Correction factors:

$$N_{caashto} := N_{caashto} \cdot S_{csaashto} \cdot D_{cdaashto} \cdot i_{caashto} \cdot C_{wcaashto} = 42.57$$

$$N_{qaashto} := N_{qaashto} \cdot S_{qsaashto} \cdot D_{qdaashto} \cdot i_{qaashto} \cdot C_{wqaashto} = 16.7$$

$$N_{\gamma aashto} := N_{\gamma aashto} \cdot S_{\gamma saashto} \cdot D_{\gamma daashto} \cdot i_{\gamma aashto} \cdot C_{w\gamma aashto} = 14.3$$

Calculate Ultimate Bearing Capacity:

$$q_{ult.asshto} := c_1 \cdot N_{caashto} + \gamma_{bouyq} \cdot D_f \cdot N_{qaashto} + \frac{1}{2} \cdot \gamma_{sat1} \cdot B \cdot N_{\gamma aashto}$$

$$q_{ult.asshto} = 18104 \quad psf$$

$$resistance.factor := 0.45 \quad \text{AASHTO Table 10.5.5.2.2-1 Page 1244}$$

$$q_{all.asshto} := q_{ult.asshto} \cdot resistance.factor = 8147 \quad psf$$



Calculation Cover Sheet

Project #: 14199.14
 Project: Mill Pond Dam
 Subject/Task: Bearing Capacity of Footing, Case 2: Mat Foundation under Spillway
 Status: Draft
 Date: 7/25/2018

Objective: Calculate bearing capacity for soil using the Meyerhof (1963), AASHTO (2014)

FOUNDATION DIMENSIONS

$B := 20$ ft width of foundation (left to right)
 $L := 110$ ft length of foundation, abutment width
 $D_f := 5$ ft depth of foundation measured from the ground surface

LOADING

$P_v := 200$ kip Vertical line load (assumed)
 $\theta := 0$ deg kip Projection direction of load relative to length L
 $\beta := 5$ deg Directional load relative to vertical
 $P_h := P_v \cdot \tan(\beta) = 17.5$ kip Horizontal load

SOIL PROPERTIES

$\gamma_w := 62.4$ pcf Density of water
 $D_w := 0$ ft Depth to ground water from the ground surface, assume saturated condition
 $\gamma_1 := 131$ pcf Unit weight of soil below foundation base
 $\gamma_{sat1} := \gamma_1 - \gamma_w = 68.6$ pcf Effective unit weight of Sand and Silt
 $H_1 := 16$ ft Height of soil layer above the foundation
 $\phi_1 := 38$ deg Internal angle of friction for Sand and Silt
 $c_1 := 0.00$ Cohesion of soil
 $N_1 := 61$ Assume N value of bearing soils based on boring logs
 $\phi_q := 35$ deg Internal angle of friction for Structural Fill above the bearing soil.
 $\gamma_q := 130$ pcf Saturated unit weight of Structural Fill above the bearing soil.
 $\gamma_{bouyq} := \gamma_q - \gamma_w = 67.6$ pcf Effective unit weight of Structural Fill

CHECK BEARING CAPACITY AASHTO LRFD Bridge Design Specifications

Bearing Capacity Factors

$$N_{qaashto} := \tan \left(45 \text{ deg} + \frac{\phi_q}{2} \right)^2 \cdot e^{\pi \cdot \tan(\phi_q)} \quad N_{qaashto} = 33.3$$

$$N_{caashto} := (N_{qaashto} - 1) \cdot \cot(\phi_1) \quad N_{caashto} = 41.3$$

$$N_{\gamma aashto} := 2 \cdot (N_{qaashto} + 1) \cdot \tan(\phi_1) \quad N_{\gamma aashto} = 53.6$$

Shape factors:

Table 10.6.3.1.2a-3—Shape Correction Factors s_c , s_γ , s_q

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

$$S_{qsaashto} := 1 + \left(\frac{B}{L} \cdot \tan(\phi_q) \right) \quad S_{qsaashto} = 1.1$$

$$S_{csaashto} := 1 + \left(\frac{B}{L} \cdot \frac{N_{qaashto}}{N_{caashto}} \right) \quad S_{csaashto} = 1.1$$

$$S_{\gamma saashto} := 1 - 0.4 \cdot \frac{B}{L} \quad S_{\gamma saashto} = 0.9$$

Depth factors: Depth factor should only be used when the soil above the footing elevation are as competent as the soils beneath the footing level; otherwise 1 is used.

$$\phi_1 = 38 \text{ deg}$$

$$D_{qdaashto} := 1$$

$$D_{cdaashto} := 1$$

$$D_{\gamma daashto} := 1$$

Water factors:Assume saturated condition $D_w = 0$ **Table 10.6.3.1.2a-2—Coefficients C_{wq} and C_{wy} for Various Groundwater Depths**

D_w	C_{wq}	C_{wy}
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

$$C_{wqaashto} := 0.5$$

$$C_{w\gamma aashto} := 0.5$$

$$C_{wcaashto} := \text{if} \left((D_f + B) < D_w, 1.0, 1.0 \right)$$

$$C_{wcaashto} = 1$$

Inclination factors:

$$n := \frac{\left(2 + \frac{L}{B}\right)}{\left(1 + \frac{L}{B}\right)} \cdot \cos(\theta)^2 + \left(\frac{\left(2 + \frac{B}{L}\right)}{\left(1 + \frac{B}{L}\right)} \cdot \sin(\theta)^2 \right) = 1.15$$

$$i_{\gamma aashto} := \left(1 - \frac{P_h}{P_v + c_1 \cdot B \cdot L \cdot \left(\frac{1}{\tan(\phi_1)} \right)} \right)^{n+1} = 0.82$$

$$i_{qaashto} := \left(1 - \frac{P_h}{(P_v + c_1 \cdot B \cdot L \cdot \cot(\phi_q))} \right)^n = 0.9$$

$$i_{caashto} := i_{qaashto} - \frac{(1 - i_{qaashto})}{(N_{qaashto} - 1)} = 0.9$$

Correction factors:

$$N_{caashto} := N_{caashto} \cdot S_{csaashto} \cdot D_{cdaashto} \cdot i_{caashto} \cdot C_{wcaashto} = 42.49$$

$$N_{qaashto} := N_{qaashto} \cdot S_{qsaashto} \cdot D_{qdaashto} \cdot i_{qaashto} \cdot C_{wqaashto} = 16.9$$

$$N_{\gamma aashto} := N_{\gamma aashto} \cdot S_{\gamma saashto} \cdot D_{\gamma daashto} \cdot i_{\gamma aashto} \cdot C_{w\gamma aashto} = 20.4$$

Calculate Ultimate Bearing Capacity:

$$q_{ult.asshto} := c_1 \cdot N_{caashto} + \gamma_{bouyq} \cdot D_f \cdot N_{qaashto} + \frac{1}{2} \cdot \gamma_{sat1} \cdot B \cdot N_{\gamma aashto}$$

$$q_{ult.asshto} = 19702 \quad psf$$

$$resistance.factor := 0.45 \quad \text{AASHTO Table 10.5.5.2.2-1 Page 1244}$$

$$q_{all.asshto} := q_{ult.asshto} \cdot resistance.factor = 8866 \quad psf$$



Calculation Cover Sheet

Project #: 14199.14
 Project: Mill Pond Dam
 Subject/Task: Bearing Capacity of Footing, Case 2: Mat Foundation under Spillway
 Status: Draft
 Date: 7/25/2018

Objective: Calculate bearing capacity for soil using the Meyerhof (1963), AASHTO (2014)

FOUNDATION DIMENSIONS

$B := 25$ ft width of foundation (left to right)
 $L := 110$ ft length of foundation, abutment width
 $D_f := 5$ ft depth of foundation measured from the ground surface

LOADING

$P_v := 200$ kip Vertical line load (assumed)
 $\theta := 0$ deg kip Projection direction of load relative to length L
 $\beta := 5$ deg Directional load relative to vertical
 $P_h := P_v \cdot \tan(\beta) = 17.5$ kip Horizontal load

SOIL PROPERTIES

$\gamma_w := 62.4$ pcf Density of water
 $D_w := 0$ ft Depth to ground water from the ground surface, assume saturated condition
 $\gamma_1 := 131$ pcf Unit weight of soil below foundation base
 $\gamma_{sat1} := \gamma_1 - \gamma_w = 68.6$ pcf Effective unit weight of Sand and Silt
 $H_1 := 16$ ft Height of soil layer above the foundation
 $\phi_1 := 38$ deg Internal angle of friction for Sand and Silt
 $c_1 := 0.00$ Cohesion of soil
 $N_1 := 61$ Assume N value of bearing soils based on boring logs
 $\phi_q := 35$ deg Internal angle of friction for Structural Fill above the bearing soil.
 $\gamma_q := 130$ pcf Saturated unit weight of Structural Fill above the bearing soil.
 $\gamma_{bouyq} := \gamma_q - \gamma_w = 67.6$ pcf Effective unit weight of Structural Fill

CHECK BEARING CAPACITY AASHTO LRFD Bridge Design Specifications

Bearing Capacity Factors

$$N_{qaashto} := \tan \left(45 \text{ deg} + \frac{\phi_q}{2} \right)^2 \cdot e^{\pi \cdot \tan(\phi_q)} \quad N_{qaashto} = 33.3$$

$$N_{caashto} := (N_{qaashto} - 1) \cdot \cot(\phi_1) \quad N_{caashto} = 41.3$$

$$N_{\gamma aashto} := 2 \cdot (N_{qaashto} + 1) \cdot \tan(\phi_1) \quad N_{\gamma aashto} = 53.6$$

Shape factors:

Table 10.6.3.1.2a-3—Shape Correction Factors s_c , s_γ , s_q

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

$$S_{qsaashto} := 1 + \left(\frac{B}{L} \cdot \tan(\phi_q) \right) \quad S_{qsaashto} = 1.2$$

$$S_{csaashto} := 1 + \left(\frac{B}{L} \cdot \frac{N_{qaashto}}{N_{caashto}} \right) \quad S_{csaashto} = 1.2$$

$$S_{\gamma saashto} := 1 - 0.4 \cdot \frac{B}{L} \quad S_{\gamma saashto} = 0.9$$

Depth factors: Depth factor should only be used when the soil above the footing elevation are as competent as the soils beneath the footing level; otherwise 1 is used.

$$\phi_1 = 38 \text{ deg}$$

$$D_{qdaashto} := 1$$

$$D_{cdaashto} := 1$$

$$D_{\gamma daashto} := 1$$

Water factors:Assume saturated condition $D_w = 0$ **Table 10.6.3.1.2a-2—Coefficients C_{wq} and C_{wy} for Various Groundwater Depths**

D_w	C_{wq}	C_{wy}
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

$$C_{wqaashto} := 0.5$$

$$C_{w\gamma aashto} := 0.5$$

$$C_{wcaashto} := \text{if} \left((D_f + B) < D_w, 1.0, 1.0 \right)$$

$$C_{wcaashto} = 1$$

Inclination factors:

$$n := \frac{\left(2 + \frac{L}{B}\right)}{\left(1 + \frac{L}{B}\right)} \cdot \cos(\theta)^2 + \left(\frac{\left(2 + \frac{B}{L}\right)}{\left(1 + \frac{B}{L}\right)} \cdot \sin(\theta)^2 \right) = 1.19$$

$$i_{\gamma aashto} := \left(1 - \frac{P_h}{P_v + c_1 \cdot B \cdot L \cdot \left(\frac{1}{\tan(\phi_1)} \right)} \right)^{n+1} = 0.82$$

$$i_{qaashto} := \left(1 - \frac{P_h}{(P_v + c_1 \cdot B \cdot L \cdot \cot(\phi_q))} \right)^n = 0.9$$

$$i_{caashto} := i_{qaashto} - \frac{(1 - i_{qaashto})}{(N_{qaashto} - 1)} = 0.9$$

Correction factors:

$$N_{caashto} := N_{caashto} \cdot S_{csaashto} \cdot D_{cdaashto} \cdot i_{caashto} \cdot C_{wcaashto} = 43.72$$

$$N_{qaashto} := N_{qaashto} \cdot S_{qsaashto} \cdot D_{qdaashto} \cdot i_{qaashto} \cdot C_{wqaashto} = 17.3$$

$$N_{\gamma aashto} := N_{\gamma aashto} \cdot S_{\gamma saashto} \cdot D_{\gamma daashto} \cdot i_{\gamma aashto} \cdot C_{w\gamma aashto} = 19.9$$

Calculate Ultimate Bearing Capacity:

$$q_{ult.asshto} := c_1 \cdot N_{caashto} + \gamma_{bouyq} \cdot D_f \cdot N_{qaashto} + \frac{1}{2} \cdot \gamma_{sat1} \cdot B \cdot N_{\gamma aashto}$$

$$q_{ult.asshto} = 22952 \quad psf$$

$$resistance.factor := 0.45 \quad \text{AASHTO Table 10.5.5.2.2-1 Page 1244}$$

$$q_{all.asshto} := q_{ult.asshto} \cdot resistance.factor = 10329 \quad psf$$



Calculation Cover Sheet

Project #: 14199.14
 Project: **Mill Pond Dam**
 Subject/Task: Bearing Capacity of Footing, Case 2: Mat Foundation under Spillway
 Status: Draft
 Date: 7/25/2018

Objective: Calculate bearing capacity for soil using the Meyerhof (1963), AASHTO (2014)

FOUNDATION DIMENSIONS

$B := 30$ ft width of foundation (left to right)
 $L := 110$ ft length of foundation, abutment width
 $D_f := 5$ ft depth of foundation measured from the ground surface

LOADING

$P_v := 200$ kip Vertical line load (assumed)
 $\theta := 0$ deg kip Projection direction of load relative to length L
 $\beta := 5$ deg Directional load relative to vertical
 $P_h := P_v \cdot \tan(\beta) = 17.5$ kip Horizontal load

SOIL PROPERTIES

$\gamma_w := 62.4$ pcf Density of water
 $D_w := 0$ ft Depth to ground water from the ground surface, assume saturated condition
 $\gamma_1 := 131$ pcf Unit weight of soil below foundation base
 $\gamma_{sat1} := \gamma_1 - \gamma_w = 68.6$ pcf Effective unit weight of Sand and Silt
 $H_1 := 16$ ft Height of soil layer above the foundation
 $\phi_1 := 38$ deg Internal angle of friction for Sand and Silt
 $c_1 := 0.00$ Cohesion of soil
 $N_1 := 61$ Assume N value of bearing soils based on boring logs
 $\phi_q := 35$ deg Internal angle of friction for Structural Fill above the bearing soil.
 $\gamma_q := 130$ pcf Saturated unit weight of Structural Fill above the bearing soil.
 $\gamma_{bouyq} := \gamma_q - \gamma_w = 67.6$ pcf Effective unit weight of Structural Fill

CHECK BEARING CAPACITY AASHTO LRFD Bridge Design Specifications

Bearing Capacity Factors

$$N_{qaashto} := \tan \left(45 \text{ deg} + \frac{\phi_q}{2} \right)^2 \cdot e^{\pi \cdot \tan(\phi_q)} \quad N_{qaashto} = 33.3$$

$$N_{caashto} := (N_{qaashto} - 1) \cdot \cot(\phi_1) \quad N_{caashto} = 41.3$$

$$N_{\gamma aashto} := 2 \cdot (N_{qaashto} + 1) \cdot \tan(\phi_1) \quad N_{\gamma aashto} = 53.6$$

Shape factors:

Table 10.6.3.1.2a-3—Shape Correction Factors s_c , s_γ , s_q

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

$$S_{qsaashto} := 1 + \left(\frac{B}{L} \cdot \tan(\phi_q) \right) \quad S_{qsaashto} = 1.2$$

$$S_{csaashto} := 1 + \left(\frac{B}{L} \cdot \frac{N_{qaashto}}{N_{caashto}} \right) \quad S_{csaashto} = 1.2$$

$$S_{\gamma saashto} := 1 - 0.4 \cdot \frac{B}{L} \quad S_{\gamma saashto} = 0.9$$

Depth factors: Depth factor should only be used when the soil above the footing elevation are as competent as the soils beneath the footing level; otherwise 1 is used.

$$\phi_1 = 38 \text{ deg}$$

$$D_{qdaashto} := 1$$

$$D_{cdaashto} := 1$$

$$D_{\gamma daashto} := 1$$

Water factors:Assume saturated condition $D_w = 0$ **Table 10.6.3.1.2a-2—Coefficients C_{wq} and C_{wy} for Various Groundwater Depths**

D_w	C_{wq}	C_{wy}
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

$$C_{wqaashto} := 0.5$$

$$C_{w\gamma aashto} := 0.5$$

$$C_{wcaashto} := \text{if} \left((D_f + B) < D_w, 1.0, 1.0 \right)$$

$$C_{wcaashto} = 1$$

Inclination factors:

$$n := \frac{\left(2 + \frac{L}{B}\right)}{\left(1 + \frac{L}{B}\right)} \cdot \cos(\theta)^2 + \left(\frac{\left(2 + \frac{B}{L}\right)}{\left(1 + \frac{B}{L}\right)} \cdot \sin(\theta)^2 \right) = 1.21$$

$$i_{\gamma aashto} := \left(1 - \frac{P_h}{P_v + c_1 \cdot B \cdot L \cdot \left(\frac{1}{\tan(\phi_1)} \right)} \right)^{n+1} = 0.82$$

$$i_{qaashto} := \left(1 - \frac{P_h}{(P_v + c_1 \cdot B \cdot L \cdot \cot(\phi_q))} \right)^n = 0.89$$

$$i_{caashto} := i_{qaashto} - \frac{(1 - i_{qaashto})}{(N_{qaashto} - 1)} = 0.9$$

Correction factors:

$$N_{caashto} := N_{caashto} \cdot S_{csaashto} \cdot D_{cdaashto} \cdot i_{caashto} \cdot C_{wcaashto} = 44.95$$

$$N_{qaashto} := N_{qaashto} \cdot S_{qsaashto} \cdot D_{qdaashto} \cdot i_{qaashto} \cdot C_{wqaashto} = 17.7$$

$$N_{\gamma aashto} := N_{\gamma aashto} \cdot S_{\gamma saashto} \cdot D_{\gamma daashto} \cdot i_{\gamma aashto} \cdot C_{w\gamma aashto} = 19.5$$

Calculate Ultimate Bearing Capacity:

$$q_{ult.asshto} := c_1 \cdot N_{caashto} + \gamma_{bouyq} \cdot D_f \cdot N_{qaashto} + \frac{1}{2} \cdot \gamma_{sat1} \cdot B \cdot N_{\gamma aashto}$$

$$q_{ult.asshto} = 26053 \quad psf$$

$$resistance.factor := 0.45 \quad \text{AASHTO Table 10.5.5.2.2-1 Page 1244}$$

$$q_{all.asshto} := q_{ult.asshto} \cdot resistance.factor = 11724 \quad psf$$



Calculation Cover Sheet

Project #: 14199.14
Project: Mill Pond Dam
Subject/Task: Bearing Capacity of Footing, Case 3: Wall Footings Downstream of Bridge
Status: Draft
Date: 7/25/2018

Objective: Calculate bearing capacity for soil using the AASHTO (2014) manual.

FOUNDATION DIMENSIONS

$B := 10$ ft width of foundation (left to right)
 $L := 30$ ft length of foundation, abutment width
 $D_f := 4$ ft depth of foundation measured from the ground surface

LOADING

$P_v := 200$ kip Vertical line load (assumed)
 $\theta := 0$ deg kip Projection direction of load relative to length L
 $\beta := 10$ deg Directional load relative to vertical

$P_h := P_v \cdot \tan(\beta) = 35.3$ kip Horizontal load

SOIL PROPERTIES

$\gamma_w := 62.4$ pcf Density of water
 $D_w := 0$ ft Depth to ground water from the ground surface, assume saturated condition

$\gamma_1 := 132$ pcf Unit weight of soil below foundation base

$\gamma_{sat1} := \gamma_1 - \gamma_w = 69.6$ pcf Effective unit weight of Sand and Silt

$H_1 := 16$ ft Height of soil layer above the foundation

$\phi_1 := 38$ deg Internal angle of friction for Sand and Silt

$c_1 := 0.00$ Cohesion of soil

$N_1 := 61$ Assume N value based on boring logs

$\phi_q := 35$ deg Internal angle of friction for Structural Fill

$\gamma_q := 130$ pcf Unit weight of Structural Fill

$\gamma_{bouyq} := \gamma_q - \gamma_w = 67.6$ pcf Effective unit weight of Structural Fill

CHECK BEARING CAPACITY AASHTO LRFD Bridge Design Specifications

Bearing Capacity Factors

$$N_{qaashto} := \tan \left(45 \text{ deg} + \frac{\phi_q}{2} \right)^2 \cdot e^{\pi \cdot \tan(\phi_q)} \quad N_{qaashto} = 33.3$$

$$N_{caashto} := (N_{qaashto} - 1) \cdot \cot(\phi_1) \quad N_{caashto} = 41.3$$

$$N_{\gamma aashto} := 2 \cdot (N_{qaashto} + 1) \cdot \tan(\phi_1) \quad N_{\gamma aashto} = 53.6$$

Shape factors:

Table 10.6.3.1.2a-3—Shape Correction Factors s_c , s_γ , s_q

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

$$S_{qsaashto} := 1 + \left(\frac{B}{L} \cdot \tan(\phi_q) \right) \quad S_{qsaashto} = 1.2$$

$$S_{csaashto} := 1 + \left(\frac{B}{L} \cdot \frac{N_{qaashto}}{N_{caashto}} \right) \quad S_{csaashto} = 1.3$$

$$S_{\gamma saashto} := 1 - 0.4 \cdot \frac{B}{L} \quad S_{\gamma saashto} = 0.9$$

Depth factors: Depth factor should only be used when the soil above the footing elevation are as competent as the soils beneath the footing level; otherwise 1 is used.

$$\phi_1 = 38 \text{ deg}$$

$$D_{qdaashto} := 1$$

$$D_{cdaashto} := 1$$

$$D_{\gamma daashto} := 1$$

Water factors:

Assume saturated condition $D_w = 0$

Table 10.6.3.1.2a-2—Coefficients C_{wq} and C_{wy} for Various Groundwater Depths

D_w	C_{wq}	C_{wy}
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

$$C_{wqaashto} := 0.5$$

$$C_{wyashto} := 0.5$$

$$C_{wcaashto} := \text{if} \left((D_f + B) < D_w, 1.0, 1.0 \right)$$

$$C_{wcaashto} = 1$$

Inclination factors:

$$n := \frac{\left(2 + \frac{L}{B}\right)}{\left(1 + \frac{L}{B}\right)} \cdot \cos(\theta)^2 + \left(\frac{\left(2 + \frac{B}{L}\right)}{\left(1 + \frac{B}{L}\right)} \cdot \sin(\theta)^2 \right) = 1.25$$

$$i_{\gamma ashto} := \left(1 - \frac{P_h}{P_v + c_1 \cdot B \cdot L \cdot \left(\frac{1}{\tan(\phi_1)} \right)} \right)^{n+1} = 0.65$$

$$i_{qaashto} := \left(1 - \frac{P_h}{(P_v + c_1 \cdot B \cdot L \cdot \cot(\phi_q))} \right)^n = 0.78$$

$$i_{caashto} := i_{qaashto} - \frac{(1 - i_{qaashto})}{(N_{qaashto} - 1)} = 0.8$$

Correction factors:

$$N_{caashto} := N_{caashto} \cdot S_{csaashto} \cdot D_{cdaashto} \cdot i_{caashto} \cdot C_{wcaashto} = 40.8$$

$$N_{qaashto} := N_{qaashto} \cdot S_{qsaashto} \cdot D_{qdaashto} \cdot i_{qaashto} \cdot C_{wqaashto} = 16.1$$

$$N_{\gamma aashto} := N_{\gamma aashto} \cdot S_{\gamma saashto} \cdot D_{\gamma daashto} \cdot i_{\gamma aashto} \cdot C_{w\gamma aashto} = 15$$

Calculate Ultimate Bearing Capacity:

$$q_{ult.asshto} := c_1 \cdot N_{caashto} + \gamma_{bouyq} \cdot D_f \cdot N_{qaashto} + \frac{1}{2} \cdot \gamma_{sat1} \cdot B \cdot N_{\gamma aashto}$$

$$q_{ult.asshto} = 9580$$

psf

$$resistance.factor := 0.45$$

AASHTO Table 10.5.5.2.2-1 Page 1244

$$q_{all.asshto} := q_{ult.asshto} \cdot resistance.factor = 4311$$

psf



Calculation Cover Sheet

Project #: 14199.14
 Project: Mill Pond Dam
 Subject/Task: Bearing Capacity of Footing, Case 3: Wall Footings Downstream of Bridge
 Status: Draft
 Date: 7/25/2018

Objective: Calculate bearing capacity for soil using the AASHTO (2014) manual.

FOUNDATION DIMENSIONS

$B := 15$ ft width of foundation (left to right)
 $L := 30$ ft length of foundation, abutment width
 $D_f := 4$ ft depth of foundation measured from the ground surface

LOADING

$P_v := 200$ kip Vertical line load (assumed)
 $\theta := 0$ deg kip Projection direction of load relative to length L
 $\beta := 10$ deg Directional load relative to vertical

$P_h := P_v \cdot \tan(\beta) = 35.3$ kip Horizontal load

SOIL PROPERTIES

$\gamma_w := 62.4$ pcf Density of water
 $D_w := 0$ ft Depth to ground water from the ground surface, assume saturated condition

$\gamma_1 := 132$ pcf Unit weight of soil below foundation base

$\gamma_{sat1} := \gamma_1 - \gamma_w = 69.6$ pcf Effective unit weight of Sand and Silt

$H_1 := 16$ ft Height of soil layer above the foundation

$\phi_1 := 38$ deg Internal angle of friction for Sand and Silt

$c_1 := 0.00$ Cohesion of soil

$N_1 := 61$ Assume N value based on boring logs

$\phi_q := 35$ deg Internal angle of friction for Structural Fill

$\gamma_q := 130$ pcf Unit weight of Structural Fill

$\gamma_{bouyq} := \gamma_q - \gamma_w = 67.6$ pcf Effective unit weight of Structural Fill

CHECK BEARING CAPACITY AASHTO LRFD Bridge Design Specifications

Bearing Capacity Factors

$$N_{qaashto} := \tan \left(45 \text{ deg} + \frac{\phi_q}{2} \right)^2 \cdot e^{\pi \cdot \tan(\phi_q)} \quad N_{qaashto} = 33.3$$

$$N_{caashto} := (N_{qaashto} - 1) \cdot \cot(\phi_1) \quad N_{caashto} = 41.3$$

$$N_{\gamma aashto} := 2 \cdot (N_{qaashto} + 1) \cdot \tan(\phi_1) \quad N_{\gamma aashto} = 53.6$$

Shape factors:

Table 10.6.3.1.2a-3—Shape Correction Factors s_c , s_γ , s_q

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

$$S_{qsaashto} := 1 + \left(\frac{B}{L} \cdot \tan(\phi_q) \right) \quad S_{qsaashto} = 1.4$$

$$S_{csaashto} := 1 + \left(\frac{B}{L} \cdot \frac{N_{qaashto}}{N_{caashto}} \right) \quad S_{csaashto} = 1.4$$

$$S_{\gamma saashto} := 1 - 0.4 \cdot \frac{B}{L} \quad S_{\gamma saashto} = 0.8$$

Depth factors: Depth factor should only be used when the soil above the footing elevation are as competent as the soils beneath the footing level; otherwise 1 is used.

$$\phi_1 = 38 \text{ deg}$$

$$D_{qdaashto} := 1$$

$$D_{cdaashto} := 1$$

$$D_{\gamma daashto} := 1$$

Water factors:Assume saturated condition $D_w = 0$ **Table 10.6.3.1.2a-2—Coefficients C_{wq} and C_{wy} for Various Groundwater Depths**

D_w	C_{wq}	C_{wy}
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

$$C_{wqaashto} := 0.5$$

$$C_{w\gamma aashto} := 0.5$$

$$C_{wcaashto} := \text{if} \left((D_f + B) < D_w, 1.0, 1.0 \right)$$

$$C_{wcaashto} = 1$$

Inclination factors:

$$n := \frac{\left(2 + \frac{L}{B}\right)}{\left(1 + \frac{L}{B}\right)} \cdot \cos(\theta)^2 + \left(\frac{\left(2 + \frac{B}{L}\right)}{\left(1 + \frac{B}{L}\right)} \cdot \sin(\theta)^2 \right) = 1.33$$

$$i_{\gamma aashto} := \left(1 - \frac{P_h}{P_v + c_1 \cdot B \cdot L \cdot \left(\frac{1}{\tan(\phi_1)} \right)} \right)^{n+1} = 0.64$$

$$i_{qaashto} := \left(1 - \frac{P_h}{(P_v + c_1 \cdot B \cdot L \cdot \cot(\phi_q))} \right)^n = 0.77$$

$$i_{caashto} := i_{qaashto} - \frac{(1 - i_{qaashto})}{(N_{qaashto} - 1)} = 0.8$$

Correction factors:

$$N_{caashto} := N_{caashto} \cdot S_{csaashto} \cdot D_{cdaashto} \cdot i_{caashto} \cdot C_{wcaashto} = 44.36$$

$$N_{qaashto} := N_{qaashto} \cdot S_{qsaashto} \cdot D_{qdaashto} \cdot i_{qaashto} \cdot C_{wqaashto} = 17.4$$

$$N_{\gamma aashto} := N_{\gamma aashto} \cdot S_{\gamma saashto} \cdot D_{\gamma daashto} \cdot i_{\gamma aashto} \cdot C_{w\gamma aashto} = 13.6$$

Calculate Ultimate Bearing Capacity:

$$q_{ult.asshto} := c_1 \cdot N_{caashto} + \gamma_{bouyq} \cdot D_f \cdot N_{qaashto} + \frac{1}{2} \cdot \gamma_{sat1} \cdot B \cdot N_{\gamma aashto}$$

$$q_{ult.asshto} = 11809 \quad psf$$

$$resistance.factor := 0.45 \quad \text{AASHTO Table 10.5.5.2.2-1 Page 1244}$$

$$q_{all.asshto} := q_{ult.asshto} \cdot resistance.factor = 5314 \quad psf$$



Calculation Cover Sheet

Project #: 14199.14
Project: Mill Pond Dam
Subject/Task: Bearing Capacity of Footing, Case 3: Wall Footings Downstream of Bridge
Status: Draft
Date: 7/25/2018

Objective: Calculate bearing capacity for soil using the AASHTO (2014) manual.

FOUNDATION DIMENSIONS

$B := 20$ ft width of foundation (left to right)
 $L := 30$ ft length of foundation, abutment width
 $D_f := 4$ ft depth of foundation measured from the ground surface

LOADING

$P_v := 200$ kip Vertical line load (assumed)
 $\theta := 0$ deg kip Projection direction of load relative to length L
 $\beta := 10$ deg Directional load relative to vertical
 $P_h := P_v \cdot \tan(\beta) = 35.3$ kip Horizontal load

SOIL PROPERTIES

$\gamma_w := 62.4$ pcf Density of water
 $D_w := 0$ ft Depth to ground water from the ground surface, assume saturated condition
 $\gamma_1 := 132$ pcf Unit weight of soil below foundation base
 $\gamma_{sat1} := \gamma_1 - \gamma_w = 69.6$ pcf Effective unit weight of Sand and Silt
 $H_1 := 16$ ft Height of soil layer above the foundation
 $\phi_1 := 38$ deg Internal angle of friction for Sand and Silt
 $c_1 := 0.00$ Cohesion of soil
 $N_1 := 61$ Assume N value based on boring logs
 $\phi_q := 35$ deg Internal angle of friction for Structural Fill
 $\gamma_q := 130$ pcf Unit weight of Structural Fill
 $\gamma_{bouyq} := \gamma_q - \gamma_w = 67.6$ pcf Effective unit weight of Structural Fill

CHECK BEARING CAPACITY AASHTO LRFD Bridge Design Specifications

Bearing Capacity Factors

$$N_{qaashto} := \tan \left(45 \text{ deg} + \frac{\phi_q}{2} \right)^2 \cdot e^{\pi \cdot \tan(\phi_q)} \quad N_{qaashto} = 33.3$$

$$N_{caashto} := (N_{qaashto} - 1) \cdot \cot(\phi_1) \quad N_{caashto} = 41.3$$

$$N_{\gamma aashto} := 2 \cdot (N_{qaashto} + 1) \cdot \tan(\phi_1) \quad N_{\gamma aashto} = 53.6$$

Shape factors:

Table 10.6.3.1.2a-3—Shape Correction Factors s_c , s_γ , s_q

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

$$S_{qsaashto} := 1 + \left(\frac{B}{L} \cdot \tan(\phi_q) \right) \quad S_{qsaashto} = 1.5$$

$$S_{csaashto} := 1 + \left(\frac{B}{L} \cdot \frac{N_{qaashto}}{N_{caashto}} \right) \quad S_{csaashto} = 1.5$$

$$S_{\gamma saashto} := 1 - 0.4 \cdot \frac{B}{L} \quad S_{\gamma saashto} = 0.7$$

Depth factors: Depth factor should only be used when the soil above the footing elevation are as competent as the soils beneath the footing level; otherwise 1 is used.

$$\phi_1 = 38 \text{ deg}$$

$$D_{qdaashto} := 1$$

$$D_{cdaashto} := 1$$

$$D_{\gamma daashto} := 1$$

Water factors:Assume saturated condition $D_w = 0$ **Table 10.6.3.1.2a-2—Coefficients C_{wq} and C_{wy} for Various Groundwater Depths**

D_w	C_{wq}	C_{wy}
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

$$C_{wqaashto} := 0.5$$

$$C_{w\gamma aashto} := 0.5$$

$$C_{wcaashto} := \text{if} \left((D_f + B) < D_w, 1.0, 1.0 \right)$$

$$C_{wcaashto} = 1$$

Inclination factors:

$$n := \frac{\left(2 + \frac{L}{B}\right)}{\left(1 + \frac{L}{B}\right)} \cdot \cos(\theta)^2 + \frac{\left(2 + \frac{B}{L}\right)}{\left(1 + \frac{B}{L}\right)} \cdot \sin(\theta)^2 = 1.4$$

$$i_{\gamma aashto} := \left(1 - \frac{P_h}{P_v + c_1 \cdot B \cdot L \cdot \left(\frac{1}{\tan(\phi_1)} \right)} \right)^{n+1} = 0.63$$

$$i_{qaashto} := \left(1 - \frac{P_h}{(P_v + c_1 \cdot B \cdot L \cdot \cot(\phi_q))} \right)^n = 0.76$$

$$i_{caashto} := i_{qaashto} - \frac{(1 - i_{qaashto})}{(N_{qaashto} - 1)} = 0.8$$

Correction factors:

$$N_{caashto} := N_{caashto} \cdot S_{csaashto} \cdot D_{cdaashto} \cdot i_{caashto} \cdot C_{wcaashto} = 47.96$$

$$N_{qaashto} := N_{qaashto} \cdot S_{qsaashto} \cdot D_{qdaashto} \cdot i_{qaashto} \cdot C_{wqaashto} = 18.6$$

$$N_{\gamma aashto} := N_{\gamma aashto} \cdot S_{\gamma saashto} \cdot D_{\gamma daashto} \cdot i_{\gamma aashto} \cdot C_{w\gamma aashto} = 12.3$$

Calculate Ultimate Bearing Capacity:

$$q_{ult.asshto} := c_1 \cdot N_{caashto} + \gamma_{bouyq} \cdot D_f \cdot N_{qaashto} + \frac{1}{2} \cdot \gamma_{sat1} \cdot B \cdot N_{\gamma aashto}$$

$$q_{ult.asshto} = 13618 \quad psf$$

$$resistance.factor := 0.45 \quad \text{AASHTO Table 10.5.5.2.2-1 Page 1244}$$

$$q_{all.asshto} := q_{ult.asshto} \cdot resistance.factor = 6128 \quad psf$$



Calculation Cover Sheet

Project #: 14199.14
 Project: **Mill Pond Dam**
 Subject/Task: Bearing Capacity of Footing, Case 4: Proposed Spillway as a Whole Section
 Status: Draft
 Date: 7/25/2018

Objective: Calculate bearing capacity for soil using the Meyerhof (1963), AASHTO (2014)

FOUNDATION DIMENSIONS

$B := 65$ ft width of foundation (left to right)
 $L := 100$ ft length of foundation, abutment width
 $D_f := 5$ ft depth of foundation measured from the ground surface

LOADING

$P_v := 200$ kip Vertical line load (assumed)
 $\theta := 0$ deg kip Projection direction of load relative to length L
 $\beta := 10$ deg Directional load relative to vertical
 $P_h := P_v \cdot \tan(\beta) = 35.3$ kip Horizontal load

SOIL PROPERTIES

$\gamma_w := 62.4$ pcf Density of water
 $D_w := 0$ ft Depth to ground water from the ground surface, assume saturated condition
 $\gamma_1 := 131$ pcf Unit weight of soil below foundation base
 $\gamma_{sat1} := \gamma_1 - \gamma_w = 68.6$ pcf Effective unit weight of Sand and Silt
 $H_1 := 16$ ft Height of soil layer above the foundation
 $\phi_1 := 38$ deg Internal angle of friction for Sand and Silt
 $c_1 := 0.00$ Cohesion of soil
 $N_1 := 61$ Assume N value based on boring logs
 $\phi_q := 35$ deg Internal angle of friction for Structural Fill
 $\gamma_q := 130$ pcf Unit weight of Structural Fill
 $\gamma_{bouyq} := \gamma_q - \gamma_w = 67.6$ pcf Effective unit weight of Structural Fill

CHECK BEARING CAPACITY AASHTO LRFD Bridge Design Specifications

Bearing Capacity Factors

$$N_{qaashto} := \tan \left(45 \text{ deg} + \frac{\phi_q}{2} \right)^2 \cdot e^{\pi \cdot \tan(\phi_q)} \quad N_{qaashto} = 33.3$$

$$N_{caashto} := (N_{qaashto} - 1) \cdot \cot(\phi_1) \quad N_{caashto} = 41.3$$

$$N_{\gamma aashto} := 2 \cdot (N_{qaashto} + 1) \cdot \tan(\phi_1) \quad N_{\gamma aashto} = 53.6$$

Shape factors:

Table 10.6.3.1.2a-3—Shape Correction Factors s_c , s_γ , s_q

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

$$S_{qsaashto} := 1 + \left(\frac{B}{L} \cdot \tan(\phi_q) \right) \quad S_{qsaashto} = 1.5$$

$$S_{csaashto} := 1 + \left(\frac{B}{L} \cdot \frac{N_{qaashto}}{N_{caashto}} \right) \quad S_{csaashto} = 1.5$$

$$S_{\gamma saashto} := 1 - 0.4 \cdot \frac{B}{L} \quad S_{\gamma saashto} = 0.7$$

Depth factors: Depth factor should only be used when the soil above the footing elevation are as competent as the soils beneath the footing level; otherwise 1 is used.

$$\phi_1 = 38 \text{ deg}$$

$$D_{qdaashto} := 1$$

$$D_{cdaashto} := 1$$

$$D_{\gamma daashto} := 1$$

Water factors:Assume saturated condition $D_w = 0$ **Table 10.6.3.1.2a-2—Coefficients C_{wq} and C_{wy} for Various Groundwater Depths**

D_w	C_{wq}	C_{wy}
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

$$C_{wqaashto} := 0.5$$

$$C_{w\gamma aashto} := 0.5$$

$$C_{wcaashto} := \text{if} \left((D_f + B) < D_w, 1.0, 1.0 \right)$$

$$C_{wcaashto} = 1$$

Inclination factors:

$$n := \frac{\left(2 + \frac{L}{B}\right)}{\left(1 + \frac{L}{B}\right)} \cdot \cos(\theta)^2 + \left(\frac{\left(2 + \frac{B}{L}\right)}{\left(1 + \frac{B}{L}\right)} \cdot \sin(\theta)^2 \right) = 1.39$$

$$i_{\gamma aashto} := \left(1 - \frac{P_h}{P_v + c_1 \cdot B \cdot L \cdot \left(\frac{1}{\tan(\phi_1)} \right)} \right)^{n+1} = 0.63$$

$$i_{qaashto} := \left(1 - \frac{P_h}{(P_v + c_1 \cdot B \cdot L \cdot \cot(\phi_1))} \right)^n = 0.76$$

$$i_{caashto} := i_{qaashto} - \frac{(1 - i_{qaashto})}{(N_{qaashto} - 1)} = 0.8$$

Correction factors:

$$N_{caashto} := N_{caashto} \cdot S_{csaashto} \cdot D_{cdaashto} \cdot i_{caashto} \cdot C_{wcaashto} = 47.6$$

$$N_{qaashto} := N_{qaashto} \cdot S_{qsaashto} \cdot D_{qdaashto} \cdot i_{qaashto} \cdot C_{wqaashto} = 18.5$$

$$N_{\gamma aashto} := N_{\gamma aashto} \cdot S_{\gamma saashto} \cdot D_{\gamma daashto} \cdot i_{\gamma aashto} \cdot C_{w\gamma aashto} = 12.5$$

Calculate Ultimate Bearing Capacity:

$$q_{ult.asshto} := c_1 \cdot N_{caashto} + \gamma_{bouyq} \cdot D_f \cdot N_{qaashto} + \frac{1}{2} \cdot \gamma_{sat1} \cdot B \cdot N_{\gamma aashto}$$

$$q_{ult.asshto} = 34033 \quad psf$$

$$resistance.factor := 0.45 \quad \text{AASHTO Table 10.5.5.2.2-1 Page 1244}$$

$$q_{all.asshto} := q_{ult.asshto} \cdot resistance.factor = 15315 \quad psf$$



Calculation Cover Sheet

Project #: 14199.14
 Project: **Mill Pond Dam**
 Subject/Task: Bearing Capacity of Footing, Case 5: Mat Foundation as a Whole Section
 Status: Draft
 Date: 7/25/2018

Objective: Calculate bearing capacity for soil using the AASHTO (2014) manual

FOUNDATION DIMENSIONS

$B := 60$ ft width of foundation (left to right)
 $L := 60$ ft length of foundation, abutment width
 $D_f := 5$ ft depth of foundation measured from the ground surface

LOADING

$P_v := 200$ kip Vertical line load (assumed)
 $\theta := 0$ deg kip Projection direction of load relative to length L
 $\beta := 5$ deg Directional load relative to vertical
 $P_h := P_v \cdot \tan(\beta) = 17.5$ kip Horizontal load

SOIL PROPERTIES

$\gamma_w := 62.4$ pcf Density of water
 $D_w := 0$ ft Depth to ground water from the ground surface, assume saturated condition
 $\gamma_1 := 132$ pcf Unit weight of soil below foundation base
 $\gamma_{sat1} := \gamma_1 - \gamma_w = 69.6$ pcf Effective unit weight of Sand and Silt
 $H_1 := 16$ ft Height of soil layer above the foundation
 $\phi_1 := 38$ deg Internal angle of friction for Sand and Silt
 $c_1 := 0.00$ Cohesion of soil
 $N_1 := 61$ Assume N value of bearing soils based on boring logs
 $\phi_q := 35$ deg Internal angle of friction for Structural Fill above the bearing soil.
 $\gamma_q := 130$ pcf Saturated unit weight of Structural Fill above the bearing soil.
 $\gamma_{bouyq} := \gamma_q - \gamma_w = 67.6$ pcf Effective unit weight of Structural Fill

CHECK BEARING CAPACITY AASHTO LRFD Bridge Design Specifications

Bearing Capacity Factors

$$N_{qaashto} := \tan \left(45 \text{ deg} + \frac{\phi_q}{2} \right)^2 \cdot e^{\pi \cdot \tan(\phi_q)} \quad N_{qaashto} = 33.3$$

$$N_{caashto} := (N_{qaashto} - 1) \cdot \cot(\phi_1) \quad N_{caashto} = 41.3$$

$$N_{\gamma aashto} := 2 \cdot (N_{qaashto} + 1) \cdot \tan(\phi_1) \quad N_{\gamma aashto} = 53.6$$

Shape factors:

Table 10.6.3.1.2a-3—Shape Correction Factors s_c , s_γ , s_q

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

$$S_{qsaashto} := 1 + \left(\frac{B}{L} \cdot \tan(\phi_q) \right) \quad S_{qsaashto} = 1.7$$

$$S_{csaashto} := 1 + \left(\frac{B}{L} \cdot \frac{N_{qaashto}}{N_{caashto}} \right) \quad S_{csaashto} = 1.8$$

$$S_{\gamma saashto} := 1 - 0.4 \cdot \frac{B}{L} \quad S_{\gamma saashto} = 0.6$$

Depth factors: Depth factor should only be used when the soil above the footing elevation are as competent as the soils beneath the footing level; otherwise 1 is used.

$$\phi_1 = 38 \text{ deg}$$

$$D_{qdaashto} := 1$$

$$D_{cdaashto} := 1$$

$$D_{\gamma daashto} := 1$$

Water factors:Assume saturated condition $D_w = 0$ **Table 10.6.3.1.2a-2—Coefficients C_{wq} and C_{wy} for Various Groundwater Depths**

D_w	C_{wq}	C_{wy}
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

$$C_{wqaashto} := 0.5$$

$$C_{w\gamma aashto} := 0.5$$

$$C_{wcaashto} := \text{if} \left((D_f + B) < D_w, 1.0, 1.0 \right)$$

$$C_{wcaashto} = 1$$

Inclination factors:

$$n := \frac{\left(2 + \frac{L}{B}\right)}{\left(1 + \frac{L}{B}\right)} \cdot \cos(\theta)^2 + \frac{\left(2 + \frac{B}{L}\right)}{\left(1 + \frac{B}{L}\right)} \cdot \sin(\theta)^2 = 1.5$$

$$i_{\gamma aashto} := \left(1 - \frac{P_h}{P_v + c_1 \cdot B \cdot L \cdot \left(\frac{1}{\tan(\phi_1)} \right)} \right)^{n+1} = 0.8$$

$$i_{qaashto} := \left(1 - \frac{P_h}{(P_v + c_1 \cdot B \cdot L \cdot \cot(\phi_q))} \right)^n = 0.87$$

$$i_{caashto} := i_{qaashto} - \frac{(1 - i_{qaashto})}{(N_{qaashto} - 1)} = 0.9$$

Correction factors:

$$N_{caashto} := N_{caashto} \cdot S_{csaashto} \cdot D_{cdaashto} \cdot i_{caashto} \cdot C_{wcaashto} = 64.76$$

$$N_{qaashto} := N_{qaashto} \cdot S_{qsaashto} \cdot D_{qdaashto} \cdot i_{qaashto} \cdot C_{wqaashto} = 24.7$$

$$N_{\gamma aashto} := N_{\gamma aashto} \cdot S_{\gamma saashto} \cdot D_{\gamma daashto} \cdot i_{\gamma aashto} \cdot C_{w\gamma aashto} = 12.8$$

Calculate Ultimate Bearing Capacity:

$$q_{ult.asshto} := c_1 \cdot N_{caashto} + \gamma_{bouyq} \cdot D_f \cdot N_{qaashto} + \frac{1}{2} \cdot \gamma_{sat1} \cdot B \cdot N_{\gamma aashto}$$

$$q_{ult.asshto} = 35041 \quad psf$$

$$resistance.factor := 0.45 \quad \text{AASHTO Table 10.5.5.2.2-1 Page 1244}$$

$$q_{all.asshto} := q_{ult.asshto} \cdot resistance.factor = 15768 \quad psf$$



Calculation Cover Sheet

Project #: 14199.14
 Project: Mill Pond Dam
 Subject/Task: Bearing Capacity of Footing, Temporary Bridge Foundation
 Status: Draft
 Date: 6/18/2023

Objective: Calculate bearing capacity for soil using the AASHTO (2020) manual

FOUNDATION DIMENSIONS

$B := 10$ ft width of foundation (left to right)
 $L := 13$ ft length of foundation, abutment width
 $D_f := 5$ ft depth of foundation measured from the shallowest ground surface

LOADING

$P_v := 20$ kip Vertical line load (assumed)
 $\theta := 0$ deg Projection direction of load relative to length L
 $\beta := 0$ deg Directional load relative to vertical
 $P_h := P_v \cdot \tan(\beta) = 0$ kip Horizontal load

SOIL PROPERTIES

$\gamma_w := 62.4$ pcf Density of water
 $D_w := 0$ ft Depth to ground water from the ground surface, assume saturated condition
 $\gamma_1 := 127$ pcf Unit weight of bearing soil
 $\gamma_{bouy1} := \gamma_1 - \gamma_w = 64.6$ pcf Effective unit weight of bearing soil
 $\phi_1 := 31$ deg Internal angle of friction for bearing soil
 $c_1 := 0.00$ Cohesion of bearing soil
 $\phi_q := 35$ deg Internal angle of friction for Structural Fill above the bearing soil.
 $\gamma_q := 130$ pcf Saturated unit weight of Structural Fill above the bearing soil.
 $\gamma_{bouyq} := \gamma_q - \gamma_w = 67.6$ pcf Effective unit weight of Structural Fill above the bearing soil

CHECK BEARING CAPACITY AASHTO LRFD Bridge Design Specifications

Bearing Capacity Factors

$$N_{qaashto} := \tan \left(45 \text{ deg} + \frac{\phi_q}{2} \right)^2 \cdot e^{\pi \cdot \tan(\phi_q)} \quad N_{qaashto} = 33.3$$

$$N_{caashto} := (N_{qaashto} - 1) \cdot \cot(\phi_1) \quad N_{caashto} = 53.7$$

$$N_{\gamma aashto} := 2 \cdot (N_{qaashto} + 1) \cdot \tan(\phi_1) \quad N_{\gamma aashto} = 41.2$$

Shape factors:

Table 10.6.3.1.2a-3—Shape Correction Factors s_c , s_γ , s_q

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

$$S_{qsaashto} := 1 + \left(\frac{B}{L} \cdot \tan(\phi_q) \right) \quad S_{qsaashto} = 1.5$$

$$S_{csaashto} := 1 + \left(\frac{B}{L} \cdot \frac{N_{qaashto}}{N_{caashto}} \right) \quad S_{csaashto} = 1.5$$

$$S_{\gamma saashto} := 1 - 0.4 \cdot \frac{B}{L} \quad S_{\gamma saashto} = 0.7$$

Depth factors: Depth factor should only be used when the soil above the footing elevation are as competent as the soils beneath the footing level; otherwise 1 is used.

$$\phi_1 = 31 \text{ deg}$$

$$D_{qdaashto} := 1$$

$$D_{cdaashto} := 1$$

$$D_{\gamma daashto} := 1$$

Water factors:

Assume saturated condition $D_w = 0$

Table 10.6.3.1.2a-2—Coefficients C_{wq} and C_{wy} for Various Groundwater Depths

D_w	C_{wq}	C_{wy}
0.0	0.5	0.5
D_f	1.0	0.5
$>1.5B + D_f$	1.0	1.0

$$C_{wqaashto} := 0.5$$

$$C_{w\gamma aashto} := 0.5$$

$$C_{wcaashto} := \text{if} \left((D_f + B) < D_w, 1.0, 1.0 \right)$$

$$C_{wcaashto} = 1$$

Inclination factors:

$$n := \frac{\left(2 + \frac{L}{B}\right)}{\left(1 + \frac{L}{B}\right)} \cdot \cos(\theta)^2 + \frac{\left(2 + \frac{B}{L}\right)}{\left(1 + \frac{B}{L}\right)} \cdot \sin(\theta)^2 = 1.43$$

$$i_{\gamma aashto} := \left(1 - \frac{P_h}{P_v + c_1 \cdot B \cdot L \cdot \left(\frac{1}{\tan(\phi_1)} \right)} \right)^{n+1} = 1$$

$$i_{qaashto} := \left(1 - \frac{P_h}{(P_v + c_1 \cdot B \cdot L \cdot \cot(\phi_q))} \right)^n = 1$$

$$i_{caashto} := i_{qaashto} - \frac{(1 - i_{qaashto})}{(N_{qaashto} - 1)} = 1$$

Correction factors:

$$N_{caashto} := N_{caashto} \cdot S_{csaashto} \cdot D_{cdaashto} \cdot i_{caashto} \cdot C_{wcaashto} = 79.36$$

$$N_{qaashto} := N_{qaashto} \cdot S_{qsaashto} \cdot D_{qdaashto} \cdot i_{qaashto} \cdot C_{wqaashto} = 25.6$$

$$N_{\gamma aashto} := N_{\gamma aashto} \cdot S_{\gamma saashto} \cdot D_{\gamma daashto} \cdot i_{\gamma aashto} \cdot C_{w\gamma aashto} = 14.3$$

Calculate Ultimate Bearing Capacity:

$$q_{ult.asshto} := c_1 \cdot N_{caashto} + \gamma_{bouyq} \cdot D_f \cdot N_{qaashto} + \frac{1}{2} \cdot \gamma_{bouy1} \cdot B \cdot N_{\gamma aashto}$$

$$q_{ult.asshto} = 13266 \quad psf$$

$$resistance.factor := 0.45 \quad \text{AASHTO Table 10.5.5.2.2-1 Page 1244}$$

$$q_{all.asshto} := q_{ult.asshto} \cdot resistance.factor = 5970 \quad psf$$

Cranberry Highway/Mill Pond Dam over the Agawam River

Settlement



Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 \cdot C_i \cdot \Delta p \cdot \Delta z_i \sum I_{zi} / E_{si}$$

Short Term

0.0531 ft

0.64 inches <1

OK

Long Term

0.0744 ft

0.89 inches <1

OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5 \sigma'_{od} / \Delta p \geq 0.5$

0.97

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf

$0.17 = \gamma' D$

where, γ_{moist} unit weight (pcf)

120

γ_{sat} unit weight (pcf) =

132.00

C_i = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$

1.0 (short-term) 1.4 (long-term)

t = time, years (end of constructic

0.1 10 (long term)

Δp = net applied footing pressure, q - σ'_{od} , tsf

2.85

Δz_i = depth increment i, 0.1B, ft

1.5

I_{zi} = influence factor of soil layer i, fig. 3-4

E_{si} = elastic modulus of soil layer i, tsf

481.25

q = applied pressure at foundation, tsf

3.03

6054

B =

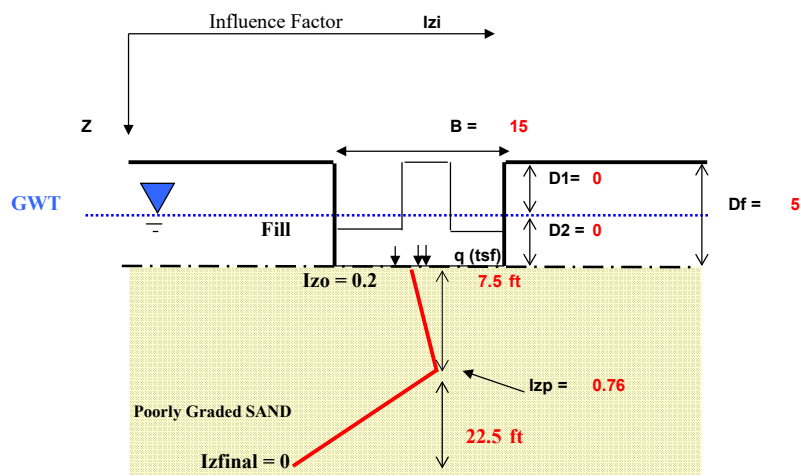
15

Df =

5

GWT depth (ft) =

0



$$I_{zo} = 0.10$$

$$I_{zp} = 0.76$$

$$I_{zp} - I_{zo} = 0.66$$

$$Z_{final (ft)} = 30$$

$$Z_{zp (ft)} = 7.5$$

$$Z_{final} - Z_{zp} = 22.5$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi
0.00	0.00	0.10			
1.50	0.10	0.23	610	0.166	0.0003
3.00	0.20	0.36	610	0.297	0.0005
4.50	0.30	0.49	610	0.428	0.0007
6.00	0.40	0.62	610	0.559	0.0009
7.50	0.50	0.76	610	0.690	0.0011
9.00	0.60	0.71	610	0.731	0.0012
10.50	0.70	0.66	610	0.680	0.0011
12.00	0.80	0.60	610	0.630	0.0010
13.50	0.90	0.55	610	0.580	0.0010
15.00	1.00	0.50	610	0.529	0.0009
16.50	1.10	0.45	610	0.479	0.0008
18.00	1.20	0.40	610	0.428	0.0007
19.50	1.30	0.35	610	0.378	0.0006
21.00	1.40	0.30	610	0.328	0.0005
22.50	1.50	0.25	610	0.277	0.0005
24.00	1.60	0.20	610	0.227	0.0004
25.50	1.70	0.15	610	0.176	0.0003
27.00	1.80	0.10	610	0.126	0.0002
28.50	1.90	0.05	610	0.076	0.0001
30.00	2.00	0.00	610	0.025	0.0000

Sand and Silt

$$\sum Izi/E_{si} = 0.0128$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1 |\Delta p / \sigma'_{izp}|^{1/2} = 0.756$

where, γ' (pcf) = effective unit weight 0.035
0.06 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{izp} = 0.5 \times B \times \gamma' + D \times \gamma' = 0.435$

2) **Elastic Modulus, Esi (ksf)**

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N \text{ (blows/ft)} = 61$$

Soil Type

ts, sandy silts, slightly cohesive silt-sand mixtu
Clean, fine to medium sands and slightly silty sands
Coarse sands and sands with little gravel
Sandy gravel and gravel

L/B = 1

10N = 610
17.5N = 1067.5
25N = 1525
30N = 1830

$$Esi \text{ (Ksf)} = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 * C_2 * \Delta p * A_z \sum \frac{I_z I_{z_i}}{L_{\text{Long Term}}}$$

Short Term

Long Term

0.0816 ft

0.1143 ft

0.28 inches <1

1.37 inches <1

OK

NOT OK

where,

C₁ = correction to account for strain relief from embedment, 1-0.5σ'_{vd}/Δp>0.5

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D, tsf

where, γ_{moist} unit weight (pcf)

γ_{moist} unit weight (pcf) = 132.00

C₂ = correction for time dependent increase in settlement, 1+0.2 log₁₀ (t / 0.1)

t = time, years (end of constructi

Δp = net applied footing pressure, q-σ'_{vd}, tsf

A_z = depth increment i, 0.1B, ft

I_z = influence factor of soil layer i, fig. 3-4

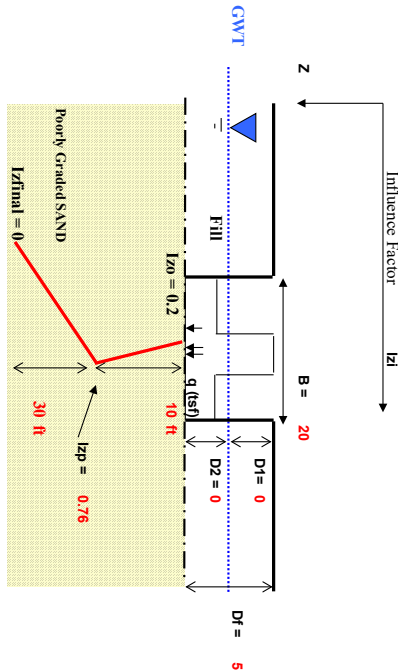
E_s = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

B = 20

D_f = 5

GWT depth (ft) = 0



7152 psf

$$I_{zo} = 0.10$$
$$I_{zp} = 0.76$$
$$I_{zp}I_{zo} = 0.66$$
$$Z_{max(0)} = 40$$
$$Z_{ap(0)} = 10$$
$$Z_{max} - Z_{ap} = 30$$

z, ft	z/B	I _{zi}	E _{si} , tsf	I _{za}	I _{za} E _{si}
0.00	0.00	0.10			
2.00	0.10	0.23	610	0.166	0.0003
4.00	0.20	0.36	610	0.297	0.0005
6.00	0.30	0.49	610	0.428	0.0007
8.00	0.40	0.62	610	0.559	0.0009
10.00	0.50	0.76	610	0.690	0.0011
12.00	0.60	0.70	610	0.730	0.0012
14.00	0.70	0.65	610	0.680	0.0011
16.00	0.80	0.60	610	0.629	0.0010
18.00	0.90	0.55	610	0.579	0.0009
20.00	1.00	0.50	610	0.529	0.0009
22.00	1.10	0.45	610	0.478	0.0008
24.00	1.20	0.40	610	0.428	0.0007
26.00	1.30	0.35	610	0.378	0.0006
28.00	1.40	0.30	610	0.327	0.0005
30.00	1.50	0.25	610	0.277	0.0005
32.00	1.60	0.20	610	0.227	0.0004
34.00	1.70	0.15	2250	0.176	0.0001
36.00	1.80	0.10	2250	0.126	0.0001
38.00	1.90	0.05	2250	0.076	0.0000
40.00	2.00	0.00	2250	0.025	0.0000

Sand and Silt

Glacial Till

$$\Sigma I_{zi}/E_{si} = 0.01231$$

1) $I_{zp} = \text{peak influence factor of soil layer 1, Fig. 3-4} = 0.5 + 0.11 \Delta p / \sigma'_{vm} \cdot 1/2 = 0.755$

where, γ' (pcf) = effective unit weight
 0.035
 0.06

Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing
 $\sigma'_{vp} = 0.5 \Delta B \times \gamma' + D \times \gamma'$
 0.522

-effective overburden pressure at depth I_{zp}, tsf

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf), After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N \text{ (blows/ft)} = 61$$
$$\frac{L}{B} = 1$$

Soil Type
s, sandy silts, slightly cohesive silt-sand mixt
Clean, fine to medium sands and slightly silty sands
Coarse sands and sands with little gravel
Sandy gravel and gravel

10N = 610
17.5N = 1067.5
25N = 1525
30N = 1830

E_{si} (ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$P_i = C_i \cdot C_1 \cdot A_p \cdot \Delta z_i \cdot \sum \frac{I_{zi}}{E_{si}}$$

Short Term

Long Term

0.1076 ft

0.1506 ft

1.22 inches < 1

1.81 inches < 1

NOT OK

NOT OK

where,

C₁ = correction to account for strain relief from embedment, 1-0.5σ'_{ad}/Δp>=0.5

σ'_{ad} = effective vertical overburden pressure at bottom of footing or depth D, tsf

where, γ_{soil} unit weight (pcf) = 120

γ_{sat} unit weight (pcf) = 132.00

C_i = correction for time dependent increase in settlement, 1+0.2 log₁₀(t / 0.1)

t = time, years (end of constructi

Δp = net applied footing pressure, q-σ'_{ad}, tsf

Δz_i = depth increment i, 0.1B, ft

I_{zi} = influence factor of soil layer i, fig. 3-4

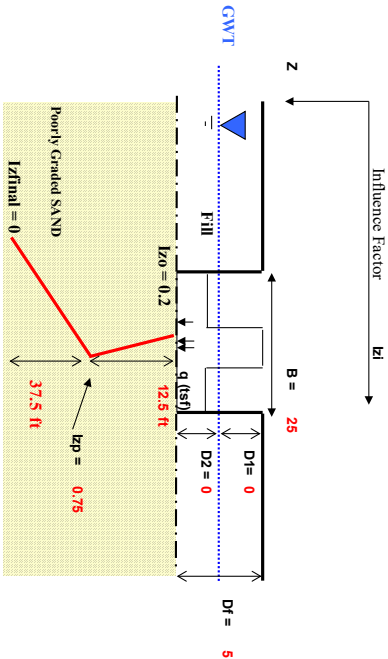
E_{si} = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

B = 25

Df = 5

GWT depth (ft) = 0



$l_{zo} = 0.10$
 $l_{zp} = 0.75$
 $l_{zp}-l_{zo} = 0.65$

$Z_{final (t)} = 50$
 $Z_{sp (t)} = 12.5$
 $Z_{final} - Z_{sp} = 37.5$

z, ft	z/B	l_{zi}	E_{si}, tsf	l_{za}	l_{za}/E_{si}	
0.00	0.00	0.10				Sand and Silt
2.50	0.10	0.23	610	0.165	0.0003	
5.00	0.20	0.36	610	0.296	0.0005	
7.50	0.30	0.49	610	0.427	0.0007	
10.00	0.40	0.62	610	0.557	0.0009	
12.50	0.50	0.75	610	0.688	0.0011	
15.00	0.60	0.70	610	0.728	0.0012	
17.50	0.70	0.65	610	0.678	0.0011	
20.00	0.80	0.60	610	0.628	0.0010	
22.50	0.90	0.55	610	0.577	0.0009	
25.00	1.00	0.50	610	0.527	0.0009	Glacial Till
27.50	1.10	0.45	610	0.477	0.0008	
30.00	1.20	0.40	610	0.427	0.0007	
32.50	1.30	0.35	610	0.377	0.0006	
35.00	1.40	0.30	2250	0.326	0.0001	
37.50	1.50	0.25	2250	0.276	0.0001	
40.00	1.60	0.20	2250	0.226	0.0001	
42.50	1.70	0.15	2250	0.176	0.0001	
45.00	1.80	0.10	2250	0.126	0.0001	
47.50	1.90	0.05	2250	0.075	0.0000	
50.00	2.00	0.00	2250	0.025	0.0000	

$\Sigma l_{zi}/E_{si} = 0.01129$

1) l_{zp} = peak influence factor of soil layer i, Fig. 3-4 = $0.5+0.1(\Delta p/\sigma'_{vm})^{1/2} = 0.753$

where, $\gamma' (cf)$ = effective unit weig U sing $\gamma' = \gamma \text{ sat} - \gamma \text{ water}$ (If soil is below GWT)
Using $\gamma' = \gamma \text{ moist}$ (If soil is above GWT)

Axisymmetric ($L/B=1$) - Spread/column footing -effective overburden pressure at depth l_{zp} , tsf
 $\sigma'_{vm} = 0.5\alpha B \times \gamma' + D \times \gamma' \quad 0.609$

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf) .After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978).

N (blows/ft) = 100 $L/B = 1$

Soil Type $10N = 1000$
s, sandy silts, slightly cohesive silt-sand mixt
 $17.5N = 1750$
Clean, fine to medium sands and slightly silty sands $25N = 2500$
Coarse sands and sands with little gravel $30N = 3000$
Sandy gravel and gravel $E_{si} (ksf) = 962.5$

Settlement - Schmertmann Approximation for Spread Footing

**Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.*

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.1855 ft	2.23 inches <1	NOT OK
Long Term	0.2597 ft	3.12 inches <1	NOT OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5 \sigma'_{od} / \Delta p \geq 0.5$ 0.98

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf 0.17 = $\gamma' D$

where, γ_{moist} unit weight (pcf) 120

γ_{sat} unit weight (pcf) = 131.00

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ 1.0 (short-term) 1.4 (long-term)

t = time, years (end of constructic 0.1 10 (long term)

Δp = net applied footing pressure, q - σ'_{od} , tsf 4.26

Δz_i = depth increment i, 0.1B, ft 2.0

I_{zi} = influence factor of soil layer i, fig. 3-4

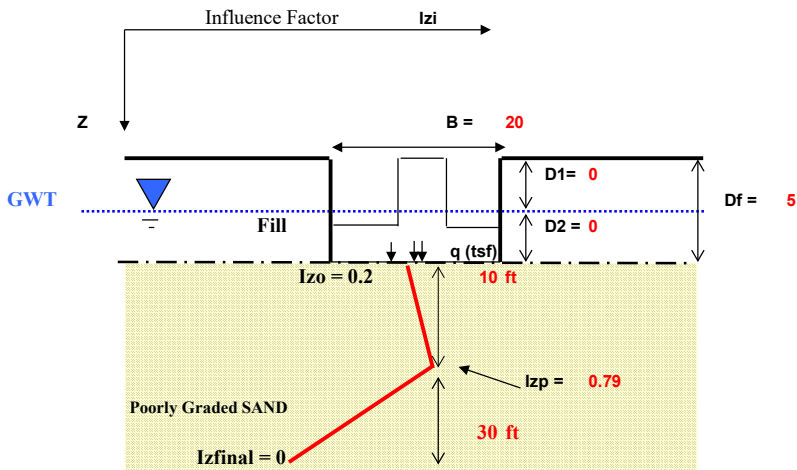
E_{si} = elastic modulus of soil layer i, tsf 481.25

q = applied pressure at foundation, tsf 4.43 8866

B = 20

Df = 5

GWT depth (ft) = 0



$$I_{zo} = 0.10$$

$$I_{zp} = 0.79$$

$$I_{zp} - I_{zo} = 0.69$$

$$Z_{final (ft)} = 40$$

$$Z_{zp (ft)} = 10$$

$$Z_{final} - Z_{zp} = 30$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Sand and Silt
2.00	0.10	0.24	350	0.169	0.0005	
4.00	0.20	0.38	350	0.306	0.0009	
6.00	0.30	0.51	350	0.444	0.0013	
8.00	0.40	0.65	350	0.581	0.0017	
10.00	0.50	0.79	350	0.719	0.0021	
12.00	0.60	0.74	350	0.762	0.0022	
14.00	0.70	0.68	350	0.709	0.0020	
16.00	0.80	0.63	350	0.656	0.0019	
18.00	0.90	0.58	350	0.604	0.0017	
20.00	1.00	0.53	350	0.551	0.0016	
22.00	1.10	0.47	350	0.499	0.0014	
24.00	1.20	0.42	350	0.446	0.0013	
26.00	1.30	0.37	350	0.394	0.0011	
28.00	1.40	0.32	350	0.341	0.0010	
30.00	1.50	0.26	350	0.289	0.0008	Glacial Till
32.00	1.60	0.21	350	0.236	0.0007	
34.00	1.70	0.16	2250	0.184	0.0001	
36.00	1.80	0.11	2250	0.131	0.0001	
38.00	1.90	0.05	2250	0.079	0.0000	
40.00	2.00	0.00	2250	0.026	0.0000	

$$\sum Izi/E_{si} = 0.02221$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1 |\Delta p / \sigma'_{izp}|^{1/2} = 0.788$

where, γ' (pcf) = effective unit weight 0.034 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{izp} = 0.5 \times B \times \gamma' + D \times \gamma' = 0.515$

2) **Elastic Modulus, Esi (ksf)**

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N \text{ (blows/ft)} = 100$$

Soil Type

ts, sandy silts, slightly cohesive silt-sand mixtu
 Clean, fine to medium sands and slightly silty sands
 Coarse sands and sands with little gravel
 Sandy gravel and gravel

L/B = 1

10N = 1000
 17.5N = 1750
 25N = 2500
 30N = 3000

$$Esi \text{ (Ksf)} = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 * C_2 * \Delta p * A_z \sum \frac{I_z I_{z_i}}{L}$$

Short Term

Long Term

0.2470 ft

0.3458 ft

2.96 inches

4.15 inches

<1

<1

NOT OK

NOT OK

where,

C₁ = correction to account for strain relief from embedment, 1-0.5σ'_{vd}/Δp>0.5

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D, tsf

where,

γ_{moist} unit weight (pcf) = 120

C₂ = correction for time dependent increase in settlement, 1+0.2 log₁₀ (t / 0.1)

t = time, years (end of constructi

Δp = net applied footing pressure, q-σ'_{vd}, tsf

A_z = depth increment i, 0.1B, ft

I_{zi} = influence factor of soil layer i, fig. 3-4

E_{si} = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

B = 25

Df = 5

GWT depth (ft) = 0

The diagram illustrates a cross-section of a spread footing on sand. The footing has a width B = 25 and is embedded at a depth Df = 5. The soil is labeled 'Poorly Graded SAND'. The groundwater table (GWT) is shown at a depth of 0 from the ground surface. The footing is subjected to a net applied pressure q (tsf). The diagram also shows the influence factor Izi and the depth increment Az. The settlement at the center of the footing is indicated as 37.5 ft. The diagram includes a vertical axis Z and a horizontal axis X. The diagram also shows the influence factor Izi and the depth increment Az. The settlement at the center of the footing is indicated as 37.5 ft. The diagram includes a vertical axis Z and a horizontal axis X.

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$$I_{zo} = 0.10$$
$$I_{zp} = 0.79$$
$$I_{zp} I_{zo} = 0.69$$
$$Z_{max(0)} = 50$$
$$Z_{ap(0)} = 12.5$$
$$Z_{max} - Z_{ap} = 37.5$$

z, ft	z/B	I _{zi}	Esi, tsf	I _{za}	I _{za} Esi
0.00	0.00	0.10			
2.50	0.10	0.24	350	0.169	0.0005
5.00	0.20	0.38	350	0.307	0.0009
7.50	0.30	0.51	350	0.444	0.0013
10.00	0.40	0.65	350	0.582	0.0017
12.50	0.50	0.79	350	0.720	0.0021
15.00	0.60	0.74	350	0.762	0.0022
17.50	0.70	0.68	350	0.710	0.0020
20.00	0.80	0.63	350	0.657	0.0019
22.50	0.90	0.58	350	0.604	0.0017
25.00	1.00	0.53	350	0.552	0.0016
27.50	1.10	0.47	350	0.499	0.0014
30.00	1.20	0.42	350	0.447	0.0013
32.50	1.30	0.37	350	0.394	0.0011
35.00	1.40	0.32	2250	0.342	0.0002
37.50	1.50	0.26	2250	0.289	0.0001
40.00	1.60	0.21	2250	0.237	0.0001
42.50	1.70	0.16	2250	0.184	0.0001
45.00	1.80	0.11	2250	0.131	0.0001
47.50	1.90	0.05	2250	0.079	0.0000
50.00	2.00	0.00	2250	0.026	0.0000

Sand and Silt

Glacial Till

$$\sum I_{zi}/E_{si} = 0.02013$$

1) I_{zp} = peak influence factor of soil layer i, fig. 3-4 = $0.5+0.1I_{ap}/\sigma'_{vm}$ $1/2 = 0.788$

where, γ' (pcf)= effective unit weight
Using $\gamma' = \gamma$ sat - γ water (If soil is below GWT)
Using $\gamma' = \gamma$ moist (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing
 $\sigma'_{vm} = 0.5\gamma B \times \gamma' + D \times \gamma'$ -effective overburden pressure at depth I_{zp} , tsf
 0.600

2) Elastic Modulus, Esi (ksf)

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

Soil Type	N (blows/ft) = 100	$L/B = 1$
s, sandy silts, slightly cohesive silt-sand mixtu		10N = 1000
Clean, fine to medium sands and slightly silty sands		17.5N = 1750
Coarse sands and sands with little gravel		25N = 2500
Sandy gravel and gravel		30N = 3000
		Esi (Ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$P_i = C_i \cdot C_1 \cdot A_p \cdot \Delta z_i \cdot \sum \frac{I_{zi}}{E_{si}}$$

Short Term

Long Term

0.3043 ft

0.4260 ft

3.65 inches < 1

5.11 inches < 1

NOT OK

NOT OK

where,

C₁ = correction to account for strain relief from embedment, 1-0.5σ'_{ad}/Δp>=0.5

σ'_{ad} = effective vertical overburden pressure at bottom of footing or depth D, tsf

where, γ_{soil} unit weight (pcf)

γ_{soil} unit weight (pcf) = 131.00

C_i = correction for time dependent increase in settlement, 1+0.2 log₁₀(t / 0.1)

t = time, years (end of constructi

Δp = net applied footing pressure, q-σ'_{ad}, tsf

Δz_i = depth increment i, 0.1B, ft

I_{zi} = influence factor of soil layer i, fig. 3-4

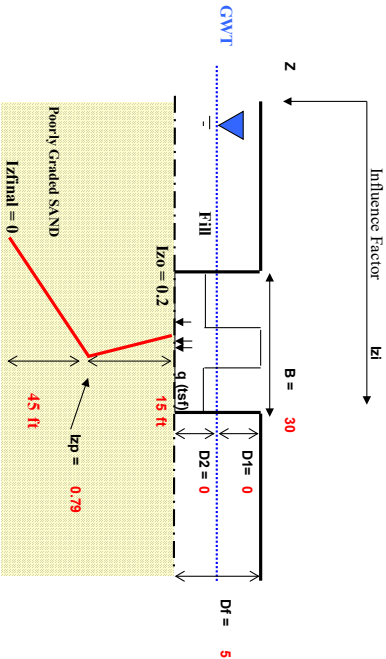
E_{si} = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

B = 30

Df = 5

GWT depth (ft) = 0



$$I_{zo} = 0.10$$
$$I_{zp} = 0.79$$
$$I_{zp} - I_{zo} = 0.69$$
$$Z_{final (ft)} = 60$$
$$Z_{sp (ft)} = 15$$
$$Z_{final} - Z_{sp} = 45$$

z, ft	z/B	I_{zi}	E_{si}, tsf	I_{za}	I_{za}/E_{si}	
0.00	0.00	0.10				Sand and Silt Glacial Till
3.00	0.10	0.24	350	0.169	0.0005	
6.00	0.20	0.38	350	0.306	0.0009	
9.00	0.30	0.51	350	0.444	0.0013	
12.00	0.40	0.65	350	0.582	0.0017	
15.00	0.50	0.79	350	0.719	0.0021	
18.00	0.60	0.74	350	0.762	0.0022	
21.00	0.70	0.68	350	0.709	0.0020	
24.00	0.80	0.63	350	0.657	0.0019	
27.00	0.90	0.58	350	0.604	0.0017	
30.00	1.00	0.53	350	0.552	0.0016	
33.00	1.10	0.47	350	0.499	0.0014	
36.00	1.20	0.42	2250	0.447	0.0002	
39.00	1.30	0.37	2250	0.394	0.0002	
42.00	1.40	0.32	2250	0.341	0.0002	
45.00	1.50	0.26	2250	0.289	0.0001	
48.00	1.60	0.21	2250	0.236	0.0001	
51.00	1.70	0.16	2250	0.184	0.0001	
54.00	1.80	0.11	2250	0.131	0.0001	
57.00	1.90	0.05	2250	0.079	0.0000	
60.00	2.00	0.00	2250	0.026	0.0000	

$$\Sigma I_{zi}/E_{si} = 0.0181$$

1) I_{zp} = peak influence factor of soil layer i, Fig. 3-4 = $0.5 + 0.1 \Delta p / \sigma'_{vm}$ $I_{zp}^{1/2} = 0.788$

where, $\gamma' (cf)$ = effective unit weig 0.034 Using $\gamma' = \gamma \text{ sat} - \gamma \text{ water}$ (If soil is below GWT)
 0.06 Using $\gamma' = \gamma \text{ moist}$ (If soil is above GWT)

Axisymmetric ($L/B=1$) - Spread/column footing $\sigma'_{vm} = 0.5 \alpha B \times \gamma' + D \times \gamma' \quad 0.686$ -effective overburden pressure at depth I_{zp}, tsf

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf) .After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978).

$$N \text{ (blows/ft)} = 100 \quad \frac{L/B = 1}{10N = 1000}$$

Soil Type
s, sandy silts, slightly cohesive silt-sand mixt $17.5N = 1750$
Clean, fine to medium sands and slightly silty sands $25N = 2500$
Coarse sands and sands with little gravel $30N = 3000$
Sandy gravel and gravel

$$E_{si} \text{ (ksf)} = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 \cdot C_2 \cdot \Delta p \cdot A_z \cdot \sum \frac{I_z I_{z_i}}{L}$$

Short Term

Long Term

0.0249 ft

0.0348 ft

0.30 inches

0.42 inches

<1

<1

OK

OK

where,

C₁ = correction to account for strain relief from embedment, 1-0.5σ'_{vd}/Δp>0.5

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D, tsf

where,

γ_{moist} unit weight (pcf) = 120

C₂ = correction for time dependent increase in settlement, 1+0.2 log₁₀ (t / 0.1)

t = time, years (end of constructi

Δp = net applied footing pressure, q-σ'_{vd}, tsf

A_z = depth increment i, 0.1B, ft

I_z = influence factor of soil layer i, fig. 3-4

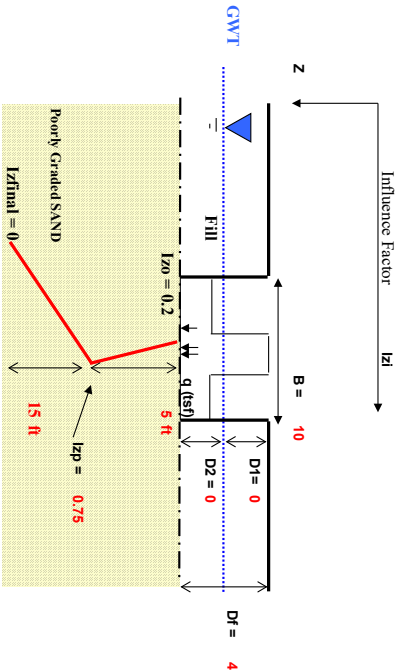
E_s = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

B = 10

D_f = 4

GWT depth (ft) = 0



$$I_{zo} = 0.10$$
$$I_{zp} = 0.75$$
$$I_{zp} - I_{zo} = 0.65$$
$$Z_{max(0)} = 20$$
$$Z_{ap(0)} = 5$$
$$Z_{max} - Z_{ap} = 15$$

z, ft	z/B	I _{zi}	E _{si} , tsf	I _{za}	I _{za} /E _{si}
0.00	0.00	0.10			
1.00	0.10	0.23	610	0.165	0.0003
2.00	0.20	0.36	610	0.296	0.0005
3.00	0.30	0.49	610	0.427	0.0007
4.00	0.40	0.62	610	0.558	0.0009
5.00	0.50	0.75	610	0.688	0.0011
6.00	0.60	0.70	610	0.729	0.0012
7.00	0.70	0.65	610	0.678	0.0011
8.00	0.80	0.60	610	0.628	0.0010
9.00	0.90	0.55	610	0.578	0.0009
10.00	1.00	0.50	610	0.528	0.0009
11.00	1.10	0.45	610	0.477	0.0008
12.00	1.20	0.40	610	0.427	0.0007
13.00	1.30	0.35	610	0.377	0.0006
14.00	1.40	0.30	610	0.327	0.0005
15.00	1.50	0.25	610	0.276	0.0004
16.00	1.60	0.20	610	0.226	0.0004
17.00	1.70	0.15	610	0.176	0.0003
18.00	1.80	0.10	610	0.126	0.0002
19.00	1.90	0.05	610	0.075	0.0001
20.00	2.00	0.00	610	0.025	0.0000

Sand and Silt

1) I_{zp} = peak influence factor of soil layer 1, fig. 3-4 = $0.5 + 0.1 \Delta p / \sigma'_{vm}$, $1/2 = 0.754$

$\Sigma I_{zi} / E_{si} = 0.01277$

where, γ' (pcf) = effective unit weight
 0.035
 0.06

Using $\gamma' = \gamma$ sat - γ water (If soil is below GWT)
Using $\gamma' = \gamma$ moist (If soil is above GWT)

Axisymmetric ($L/B=1$) - Spread/column footing
 $\sigma'_{vm} = 0.5 \gamma B \times \gamma' + D \times \gamma'$ 0.313

-effective overburden pressure at depth I_{zp} , tsf

2) Elastic Modulus, Esi (ksf)

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

N (blows/ft) = 61

$L/B = 1$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixtu

Clean, fine to medium sands and slightly silty sands

Coarse sands and sands with little gravel

Sandy gravel and gravel

$10N = 610$
 $17.5N = 1067.5$
 $25N = 1525$
 $30N = 1830$

Esi (Ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$P_i = C_1 \cdot C_2 \cdot A_p \cdot \Delta z \cdot \sum \frac{I_z I}{E_{si}}$$

Short Term

Long Term

0.0467 ft

0.0654 ft

0.56 inches

0.78 inches

<1

<1

OK

OK

where,

C₁ = correction to account for strain relief from embedment, 1-0.5σ'_{ad}/Δp>=0.5

σ'_{ad} = effective vertical overburden pressure at bottom of footing or depth D, tsf

where, γ_{soil} unit weight (pcf)

γ_{soil} unit weight (pcf) = 132.00

C₂ = correction for time dependent increase in settlement, 1+0.2 log₁₀(t / 0.1)

t = time, years (end of constructi

Δp = net applied footing pressure, q-σ'_{ad}, tsf

Δz_i = depth increment i, 0.1B, ft

I_{zi} = influence factor of soil layer i, fig. 3-4

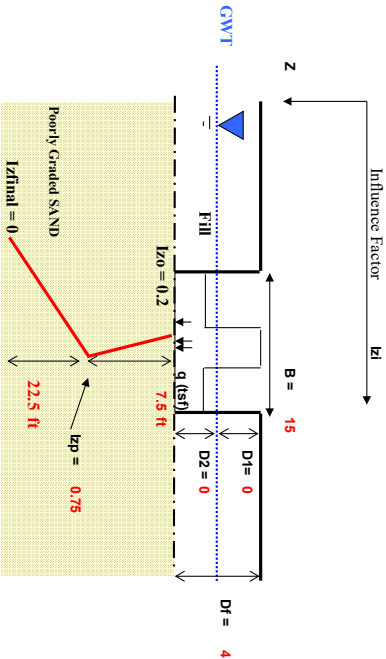
E_{si} = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

B = 15

Df = 4

GWT depth (ft) = 0



$I_{zo} = 0.10$
 $I_{zp} = 0.75$
 $I_{zp} - I_{zo} = 0.65$

$Z_{final (ft)} = 30$
 $Z_{sp (ft)} = 7.5$
 $Z_{final} - Z_{sp} = 22.5$

z, ft	z/B	I_{zi}	E_{si}, tsf	I_{za}	I_{za}/E_{si}	Sand and Silt
0.00	0.00	0.10	610	0.165	0.0003	
1.50	0.10	0.23	610	0.165	0.0003	
3.00	0.20	0.36	610	0.295	0.0005	
4.50	0.30	0.49	610	0.425	0.0007	
6.00	0.40	0.62	610	0.556	0.0009	
7.50	0.50	0.75	610	0.686	0.0011	
9.00	0.60	0.70	610	0.726	0.0012	
10.50	0.70	0.65	610	0.676	0.0011	
12.00	0.80	0.60	610	0.626	0.0010	
13.50	0.90	0.55	610	0.576	0.0009	
15.00	1.00	0.50	610	0.526	0.0009	
16.50	1.10	0.45	610	0.476	0.0008	
18.00	1.20	0.40	610	0.425	0.0007	
19.50	1.30	0.35	610	0.375	0.0006	
21.00	1.40	0.30	610	0.325	0.0005	
22.50	1.50	0.25	610	0.275	0.0004	
24.00	1.60	0.20	610	0.225	0.0003	
25.50	1.70	0.15	610	0.175	0.0003	
27.00	1.80	0.10	610	0.125	0.0002	
28.50	1.90	0.05	610	0.075	0.0001	
30.00	2.00	0.00	610	0.025	0.0000	

$\Sigma I_{zi}/E_{si} = 0.01272$

1) I_{zp} = peak influence factor of soil layer i, Fig. 3-4 = $0.5 + 0.1 \Delta p / \sigma'_{vm}$ $^{1/2}$ = 0.751

where, $\gamma' (cf)$ = effective unit weig 0.035 Using $\gamma' = \gamma \text{ sat} - \gamma \text{ water}$ (If soil is below GWT)
0.06 Using $\gamma' = \gamma \text{ moist}$ (If soil is above GWT)

Axisymmetric ($L/B=1$) - Spread/column footing -effective overburden pressure at depth I_{zp}, tsf
 $\sigma'_{vm} = 0.5 \Delta B \times \gamma' + D \times \gamma' = 0.400$

2) Elastic Modulus, $E_{si} (ksf)$

Convert SPT (blows/ft) to $E_{si} (ksf)$ -After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978).

$N \text{ (blows/ft)} = 61$ $L/B = 1$
Soil Type 40N = 610
s, sandy silts, slightly cohesive silt-sand mixt 17.5N = 1067.5
Clean, fine to medium sands and slightly silty sands 25N = 1525
Coarse sands and sands with little gravel 30N = 1830
Sandy gravel and gravel

$E_{si} (ksf) = 962.5$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$P_i = C_i \cdot C_1 \cdot A_p \cdot \Delta z_i \cdot \sum \frac{I_z i}{E_{s_i}}$$

Short Term

Long Term

0.0721 ft

0.1009 ft

0.87 inches < 1

1.21 inches < 1

OK

NOT OK

where,

C_1 = correction to account for strain relief from embedment, $1-0.5\sigma'_{ad}/\Delta p \geq 0.5$

σ'_{ad} = effective vertical overburden pressure at bottom of footing or depth D, tsf

Δp = net applied footing pressure, q- σ'_{ad} , tsf

0.98

0.14 = γ D

where, γ_{sat} unit weight (pcf) = 132.00

γ_{moist} unit weight (pcf) = 120

C_i = correction for time dependent increase in settlement, $1+0.2 \log_{10}(t / 0.1)$

t = time, years (end of construction) 0.1 10 (long term)

Δp = net applied footing pressure, q- σ'_{ad} , tsf

Δz_i = depth increment i, 0.1B, ft

$I_z i$ = influence factor of soil layer i, fig. 3-4

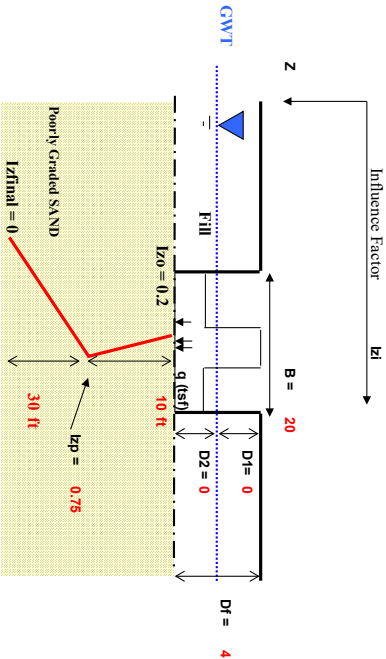
E_{s_i} = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

B = 20

Df = 4

GWT depth (ft) = 0



$$\begin{aligned} I_{zo} &= 0.10 \\ I_{zp} &= 0.75 \\ I_{zp} - I_{zo} &= 0.65 \end{aligned}$$

$$\begin{aligned} Z_{\text{final}}(\text{ft}) &= 40 \\ Z_{zp}(\text{ft}) &= 10 \\ Z_{\text{final}} - Z_{zp} &= 30 \end{aligned}$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi
0.00	0.00	0.10			
2.00	0.10	0.23	610	0.165	0.0003
4.00	0.20	0.36	610	0.294	0.0005
6.00	0.30	0.49	610	0.423	0.0007
8.00	0.40	0.62	610	0.552	0.0009
10.00	0.50	0.75	610	0.681	0.0011
12.00	0.60	0.70	610	0.720	0.0012
14.00	0.70	0.65	610	0.671	0.0011
16.00	0.80	0.60	610	0.621	0.0010
18.00	0.90	0.55	610	0.571	0.0009
20.00	1.00	0.50	610	0.522	0.0009
22.00	1.10	0.45	610	0.472	0.0008
24.00	1.20	0.40	610	0.422	0.0007
26.00	1.30	0.35	610	0.373	0.0006
28.00	1.40	0.30	610	0.323	0.0005
30.00	1.50	0.25	610	0.273	0.0004
32.00	1.60	0.20	610	0.224	0.0004
34.00	1.70	0.15	610	0.174	0.0003
36.00	1.80	0.10	610	0.124	0.0002
38.00	1.90	0.05	610	0.075	0.0001
40.00	2.00	0.00	610	0.025	0.0000

Sand and Silt

$$\sum Izi/E_{si} = 0.01262$$

1) $I_{zp} = \text{peak influence factor of soil layer } i, \text{ fig. 3-4} = 0.5 + 0.1[\Delta p/\sigma'_{zp}]^{1/2} = 0.745$

where, γ' (pcf) = effective unit weight 0.035 Using $\gamma' = \gamma_{\text{sat}} - \gamma_{\text{water}}$ (If soil is below GWT)
0.06 Using $\gamma' = \gamma_{\text{moist}}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing -effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{zp} = 0.5 \times B \times \gamma' + D \times \gamma' = 0.487$

2) **Elastic Modulus, Esi (ksf)**

Convert SPT (blows/ft) to Esi (ksf), After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N \text{ (blows/ft)} = 61$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt
 Clean, fine to medium sands and slightly silty sands
 Coarse sands and sands with little gravel
 Sandy gravel and gravel

L/B = 1

$$\begin{aligned} 10N &= 610 \\ 17.5N &= 1067.5 \\ 25N &= 1525 \\ 30N &= 1830 \end{aligned}$$

$$Esi \text{ (Ksf)} = 962.5$$

Case 4

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.4026 ft	4.83 inches < 1	NOT OK
Long Term	0.5636 ft	6.76 inches < 1	NOT OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5 \sigma'_{od} / \Delta p \geq 0.5$ 0.99

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf 0.17 = $\gamma' D$

where, γ_{moist} unit weight (pcf) 120

γ_{sat} unit weight (pcf) = 131.00

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ 1.0 (short-term) 1.4 (long-term)

t = time, years (end of constructic 0.1 10 (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf 7.49

Δz_i = depth increment i , 0.1B, ft 6.5

I_{zi} = influence factor of soil layer i , fig. 3-4

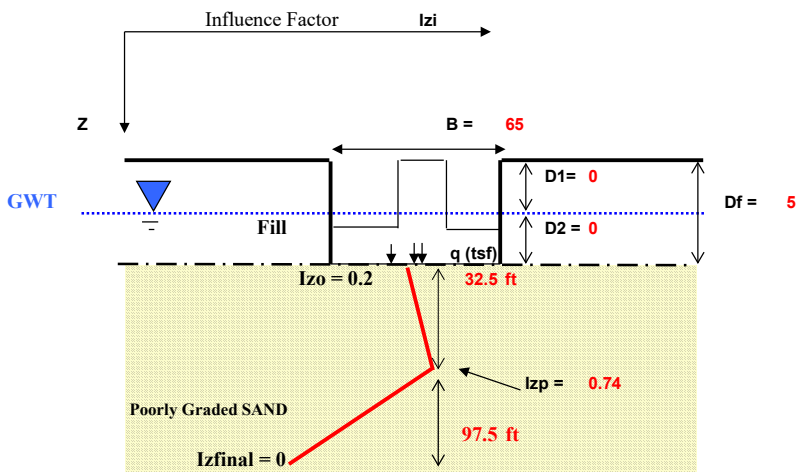
E_{si} = elastic modulus of soil layer i , tsf 481.25

q = applied pressure at foundation, tsf 7.66 15315

B = 65

D_f = 5

GWT depth (ft) = 0



$$I_{zo} = 0.10$$

$$I_{zp} = 0.74$$

$$I_{zp} - I_{zo} = 0.64$$

$$Z_{final (ft)} = 130$$

$$Z_{zp (ft)} = 32.5$$

$$Z_{final} - Z_{zp} = 97.5$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Sand and Silt
6.50	0.10	0.23	350	0.164	0.0005	
13.00	0.20	0.36	350	0.292	0.0008	
19.50	0.30	0.48	350	0.421	0.0012	
26.00	0.40	0.61	350	0.549	0.0016	
32.50	0.50	0.74	350	0.677	0.0019	Gravel Till Weather
39.00	0.60	0.69	2250	0.717	0.0003	
45.50	0.70	0.64	2250	0.667	0.0003	
52.00	0.80	0.59	2250	0.618	0.0003	
58.50	0.90	0.54	2250	0.568	0.0003	
65.00	1.00	0.49	2250	0.519	0.0002	Bedrock
71.50	1.10	0.44	2500	0.469	0.0002	
78.00	1.20	0.40	2500	0.420	0.0002	
84.50	1.30	0.35	2500	0.371	0.0001	
91.00	1.40	0.30	2500	0.321	0.0001	
97.50	1.50	0.25	2500	0.272	0.0001	
104.00	1.60	0.20	2500	0.222	0.0001	
110.50	1.70	0.15	2500	0.173	0.0001	
117.00	1.80	0.10	2500	0.124	0.0000	
123.50	1.90	0.05	2500	0.074	0.0000	
130.00	2.00	0.00	2500	0.025	0.0000	

$$\sum Izi/E_{si} = 0.00837$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1 |\Delta p / \sigma'_{izp}|^{1/2} = 0.741$

where, γ' (pcf) = effective unit weight 0.034 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing -effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{izp} = 0.5 \times B \times \gamma' + D \times \gamma' = 1.286$

2) **Elastic Modulus, Esi (ksf)**

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N \text{ (blows/ft)} = 100$$

Soil Type

ts, sandy silts, slightly cohesive silt-sand mixtu
 Clean, fine to medium sands and slightly silty sands
 Coarse sands and sands with little gravel
 Sandy gravel and gravel

L/B = 1

10N = 1000
 17.5N = 1750
 25N = 2500
 30N = 3000

$$Esi \text{ (Ksf)} = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.2640 ft	3.17 inches < 1	NOT OK
Long Term	0.3696 ft	4.43 inches < 1	NOT OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od}/\Delta p \geq 0.5$ **0.99**

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf **0.17** $= \gamma' D$

where, γ_{moist} unit weight (pcf) **120**

γ_{sat} unit weight (pcf) = **132.00**

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ **1.0** (short-term) **1.4** (long-term)

t = time, years (end of constructic **0.1** **10** (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf **7.71**

Δz_i = depth increment i, 0.1B, ft **6.0**

I_{zi} = influence factor of soil layer i, fig. 3-4

E_{si} = elastic modulus of soil layer i, tsf **481.25**

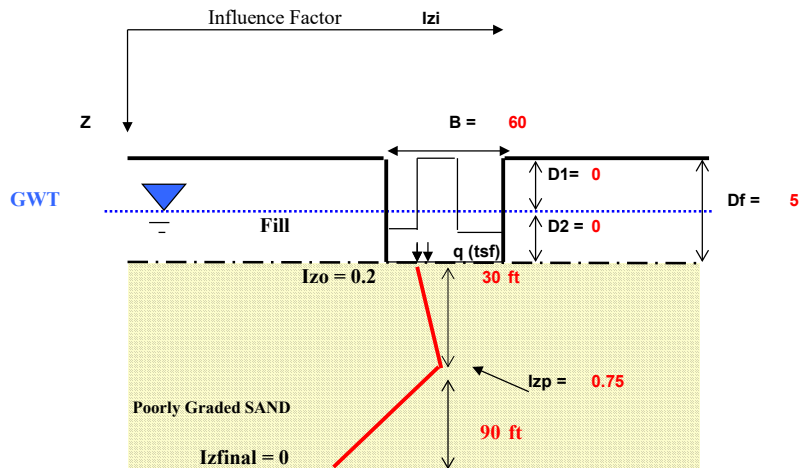
q = applied pressure at foundation, tsf **7.88**

B = **60**

Df = **5**

GWT depth (ft) = **0**

15768 psf



$$I_{zo} = 0.10$$

$$I_{zp} = 0.75$$

$$I_{zp} - I_{zo} = 0.65$$

$$Z_{\text{final (ft)}} = 120$$

$$Z_{\text{zp (ft)}} = 30$$

$$Z_{\text{final}} - Z_{\text{zp}} = 90$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Sand and Silt
6.00	0.10	0.23	610	0.165	0.0003	
12.00	0.20	0.36	610	0.295	0.0005	
18.00	0.30	0.49	610	0.426	0.0007	
24.00	0.40	0.62	610	0.556	0.0009	
30.00	0.50	0.75	610	0.686	0.0011	
36.00	0.60	0.70	2250	0.727	0.0003	
42.00	0.70	0.65	2250	0.676	0.0003	
48.00	0.80	0.60	2250	0.626	0.0003	
54.00	0.90	0.55	2250	0.576	0.0003	
60.00	1.00	0.50	2250	0.526	0.0002	Glacial Till
66.00	1.10	0.45	2250	0.476	0.0002	
72.00	1.20	0.40	3000	0.426	0.0001	
78.00	1.30	0.35	3000	0.376	0.0001	
84.00	1.40	0.30	3000	0.326	0.0001	Weathered Bedrock
90.00	1.50	0.25	3000	0.276	0.0001	
96.00	1.60	0.20	3000	0.225	0.0001	
102.00	1.70	0.15	3000	0.175	0.0001	
108.00	1.80	0.10	3000	0.125	0.0000	
114.00	1.90	0.05	3000	0.075	0.0000	
120.00	2.00	0.00	2500	0.025	0.0000	

$$\sum Izi/E_{si} = 0.00577$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1[\Delta p/\sigma'_{i, zp}]^{1/2} = 0.752$

where, γ' (pcf) = effective unit weight 0.035 Using $\gamma' = \gamma_{\text{sat}} - \gamma_{\text{water}}$ (If soil is below GWT)
0.06 Using $\gamma' = \gamma_{\text{moist}}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing -effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{i, zp} = 0.5 \times B \times \gamma' + D \times \gamma' \quad 1.218$

2) **Elastic Modulus, Esi (ksf)**

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N \text{ (blows/ft)} = 100$$

Soil Type

ts, sandy silts, slightly cohesive silt-sand mixtu
 Clean, fine to medium sands and slightly silty sands
 Coarse sands and sands with little gravel
 Sandy gravel and gravel

L/B = 1

10N = 1000
 17.5N = 1750
 25N = 2500
 30N = 3000

$$Esi \text{ (Ksf)} = 962.5$$

0.2" Settlement

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.0120 ft	0.14 inches	<1	OK
Long Term	0.0169 ft	0.20 inches	<1	OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od} / \Delta p \geq 0.5$ 0.89

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf 0.17 = $\gamma' D$

where, γ_{moist} unit weight (pcf) 120

γ_{sat} unit weight (pcf) = 132.00

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ 1.0 (short-term) 1.4 (long-term)

t = time, years (end of construction) 0.1 10 (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf 0.83

Δz_i = depth increment i, 0.1B, ft 1.5

I_{zi} = influence factor of soil layer i, fig. 3-4

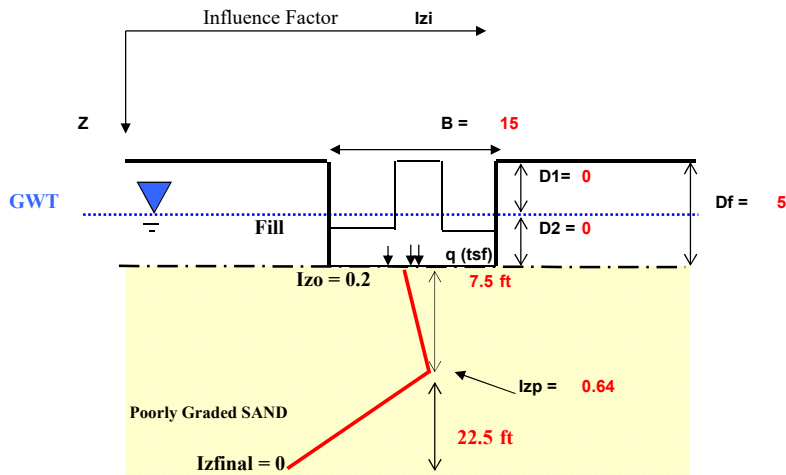
E_{si} = elastic modulus of soil layer i, tsf 481.25

q = applied pressure at foundation, tsf 1.00 2000

B = 15

Df = 5

GWT depth (ft) = 0



$$\begin{aligned} I_{zo} &= 0.10 \\ I_{zp} &= 0.64 \\ I_{zp} - I_{zo} &= 0.54 \end{aligned}$$

$$\begin{aligned} Z_{\text{final}} (\text{ft}) &= 30 \\ Z_{zp} (\text{ft}) &= 7.5 \\ Z_{\text{final}} - Z_{zp} &= 22.5 \end{aligned}$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	Sand and Silt
0.00	0.00	0.10				
1.50	0.10	0.21	610	0.154	0.0003	
3.00	0.20	0.32	610	0.261	0.0004	
4.50	0.30	0.42	610	0.369	0.0006	
6.00	0.40	0.53	610	0.476	0.0008	
7.50	0.50	0.64	610	0.584	0.0010	
9.00	0.60	0.60	610	0.617	0.0010	
10.50	0.70	0.55	610	0.574	0.0009	
12.00	0.80	0.51	610	0.531	0.0009	
13.50	0.90	0.47	610	0.489	0.0008	
15.00	1.00	0.43	610	0.446	0.0007	
16.50	1.10	0.38	610	0.404	0.0007	
18.00	1.20	0.34	610	0.361	0.0006	
19.50	1.30	0.30	610	0.319	0.0005	
21.00	1.40	0.26	610	0.276	0.0005	
22.50	1.50	0.21	610	0.234	0.0004	
24.00	1.60	0.17	610	0.191	0.0003	
25.50	1.70	0.13	610	0.149	0.0002	
27.00	1.80	0.09	610	0.106	0.0002	
28.50	1.90	0.04	610	0.064	0.0001	
30.00	2.00	0.00	610	0.021	0.0000	

$$\sum Izi/E_{si} = 0.01087$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1[\Delta p/\sigma'_{i_{zp}}]^{1/2} = 0.638$

where, γ' (pcf) = effective unit weight **0.035** Using $\gamma' = \gamma_{\text{sat}} - \gamma_{\text{water}}$ (If soil is below GWT)
0.06 Using $\gamma' = \gamma_{\text{moist}}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{i_{zp}} = 0.5 \times B \times \gamma' + D \times \gamma' = 0.435$

2) Elastic Modulus, Esi (ksf)

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N (\text{blows/ft}) = 61$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt
 Clean, fine to medium sands and slightly silty sands
 Coarse sands and sands with little gravel
 Sandy gravel and gravel

L/B = 1

10N = **610**
 17.5N = **1067.5**
 25N = **1525**
 30N = **1830**

$$E_{si} (\text{Ksf}) = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 \cdot C_2 \cdot \Delta p \cdot \Delta z \cdot \sum \frac{I_z I_{z_i}}{E_{si}}$$

	Short Term	0.0087 ft	0.10 inches	<1 inches	OK
	Long Term	0.0121 ft	0.15 inches	<1 inches	OK

where,

C₁ = correction to account for strain relief from embedment, 1-0.5σ'_{vd}/Δp>=0.5

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D, tsf

where,

γ_{sat} unit weight (pcf) = 120

C₂ = correction for time dependent increase in settlement, 1+0.2 log₁₀(t/0.1)

t = time, years (end of construction) 0.1

Δp = net applied footing pressure, q-σ'_{vd}, tsf

Δz_i = depth increment i, 0.1B, ft

I_{zi} = influence factor of soil layer i, fig. 3-4

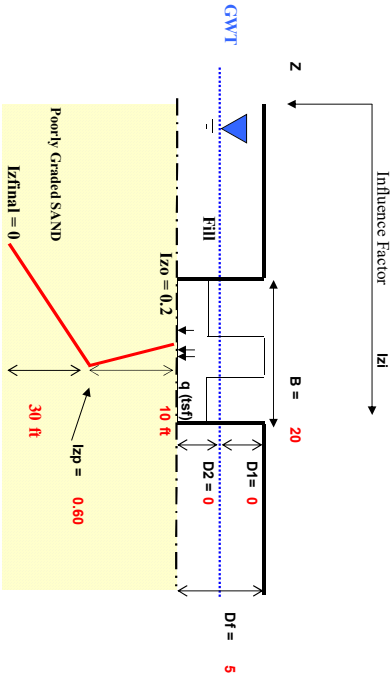
E_{si} = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

B = 20

Df = 5

GWT depth (ft) = 0



$I_{zo} = 0.10$
 $I_{zp} = 0.60$
 $I_{zp}-I_{zo} = 0.50$

$Z_{final (ft)} = 40$
 $Z_{sp (ft)} = 10$
 $Z_{final} - Z_{sp} = 30$

z, ft	z/B	I _{zi}	E _s , tsf	I _{za}	I _{za} /E _s
0.00	0.00	0.10			
2.00	0.10	0.20	610	0.150	0.0002
4.00	0.20	0.30	610	0.250	0.0004
6.00	0.30	0.40	610	0.350	0.0006
8.00	0.40	0.50	610	0.450	0.0007
10.00	0.50	0.60	610	0.550	0.0009
12.00	0.60	0.56	610	0.580	0.0010
14.00	0.70	0.52	610	0.540	0.0009
16.00	0.80	0.48	610	0.500	0.0008
18.00	0.90	0.44	610	0.460	0.0008
20.00	1.00	0.40	610	0.420	0.0007
22.00	1.10	0.36	610	0.380	0.0006
24.00	1.20	0.32	610	0.340	0.0006
26.00	1.30	0.28	610	0.300	0.0005
28.00	1.40	0.24	610	0.260	0.0004
30.00	1.50	0.20	610	0.220	0.0004
32.00	1.60	0.16	610	0.180	0.0003
34.00	1.70	0.12	2250	0.140	0.0001
36.00	1.80	0.08	2250	0.100	0.0000
38.00	1.90	0.04	2250	0.060	0.0000
40.00	2.00	0.00	2250	0.020	0.0000

Sand and Silt
Till
Glacial

$\sum I_{zi}/E_{si} = 0.00987$

1) I_{zp} = peak influence factor of soil layer i, Fig. 3-4 = $0.5+0.1I_{ap}/\sigma'_{v_{ap}})^{1/2} = 0.600$

where, γ' (pcf) = effective unit weight **0.035**
Using $\gamma' = \gamma$ sat - γ water (If soil is below GWT)
0.06 Using $\gamma' = \gamma$ moist (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing -effective overburden pressure at depth I_{zp}, tsf

$\sigma'_{lap} = 0.5 \gamma B \times \gamma' + D \times \gamma' \quad 0.522$

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

Soil Type	N (blows/ft) = 100	L/B = 1
s, sandy silts, slightly cohesive silt-sand mixtu		10N = 1000
Clean, fine to medium sands and slightly silty sands		17.5N = 1750
Coarse sands and sands with little gravel		25N = 2500
Sandy gravel and gravel		30N = 3000
		E _{si} (ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis", 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 \cdot C_2 \cdot \Delta p \cdot A_z \cdot \sum \frac{I_z}{E_{si}}$$

Short Term

Long Term

0.0116 ft

0.0163 ft

0.14 inches

0.20 inches

<1

<1

OK

OK

where,

C₁ = correction to account for strain relief from embedment, 1-0.5 σ'_{vd}/Δp>=0.;

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D, ts

where,

$$\gamma_{\text{ moist unit weight (pcf)}} = 120$$

132.00

C₂ = correction for time dependent increase in settlement, 1+0.2 log₁₀ (t / 0.1)

t = time, years (end of construct

Δp = net applied footing pressure, q-σ'_{vd}, tsf

A_z = depth increment i, 0.1B, ft

I_{zi} = influence factor of soil layer i, fig. 3-4

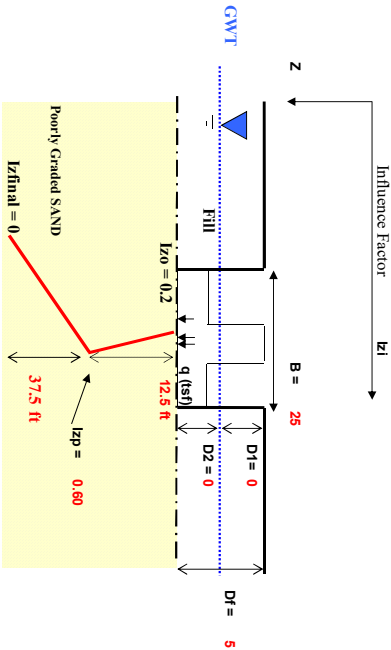
E_{si} = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

Df = 5

B = 25

GWT depth (ft) = 0



$I_{zo} = 0.10$
 $I_{zp} = 0.60$
 $I_{zp} - I_{zo} = 0.50$

$Z_{final(t)} = 50$
 $Z_{avg(t)} = 12.5$
 $Z_{final} - Z_{avg} = 37.5$

z, ft	z/B	I _{zi}	E _{si} , tsf	I _{za}	I _{za} /E _{si}
0.00	0.00	0.10			
2.50	0.10	0.20	610	0.150	0.0002
5.00	0.20	0.30	610	0.250	0.0004
7.50	0.30	0.40	610	0.350	0.0006
10.00	0.40	0.50	610	0.450	0.0007
12.50	0.50	0.60	610	0.549	0.0009
15.00	0.60	0.56	610	0.579	0.0009
17.50	0.70	0.52	610	0.539	0.0009
20.00	0.80	0.48	610	0.499	0.0008
22.50	0.90	0.44	610	0.459	0.0008
25.00	1.00	0.40	610	0.420	0.0007
27.50	1.10	0.36	610	0.380	0.0006
30.00	1.20	0.32	610	0.340	0.0006
32.50	1.30	0.28	610	0.300	0.0005
35.00	1.40	0.24	2250	0.260	0.0001
37.50	1.50	0.20	2250	0.220	0.0001
40.00	1.60	0.16	2250	0.180	0.0001
42.50	1.70	0.12	2250	0.140	0.0001
45.00	1.80	0.08	2250	0.100	0.0000
47.50	1.90	0.04	2250	0.060	0.0000
50.00	2.00	0.00	2250	0.020	0.0000

Sand and Silt
Glacial Till

$\sum I_{zi}/E_{si} = 0.00907$

1) I_{zp} = peak influence factor of soil layer i, **Fig. 3-4** = $0.5 + 0.11 \Delta \ln \sigma'_{avg} \sigma'_{avg}^{-1/2} = 0.599$

where, γ' (pcf) = effective unit wei; **0.035** Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing
 $\sigma'_{avg} = 0.5 \times B \times \gamma' + D \times \gamma' = 0.609$ -effective overburden pressure at depth I_{zp} , tsf

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf), After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978).

Soil Type	N (blows/ft) = 100	L/B = 1
s, sandy silts, slightly cohesive silt-sand mixt		10N = 1000
Clean, fine to medium sands and slightly silty sands		17.5N = 1750
Coarse sands and sands with little gravel		25N = 2500
Sandy gravel and gravel		30N = 3000
		E _{si} (ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.0121 ft	0.14 inches	<1	OK
Long Term	0.0169 ft	0.20 inches	<1	OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od} / \Delta p \geq 0.5$ **0.84**

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf **0.17 = $\gamma' D$**

where, γ_{moist} unit weight (pcf) **120**

γ_{sat} unit weight (pcf) = **131.00**

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ **1.0** (short-term) **1.4** (long-term)

t = time, years (end of construction) **0.1** **10** (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf **0.53**

Δz_i = depth increment i, 0.1B, ft **1.5**

I_{zi} = influence factor of soil layer i, fig. 3-4

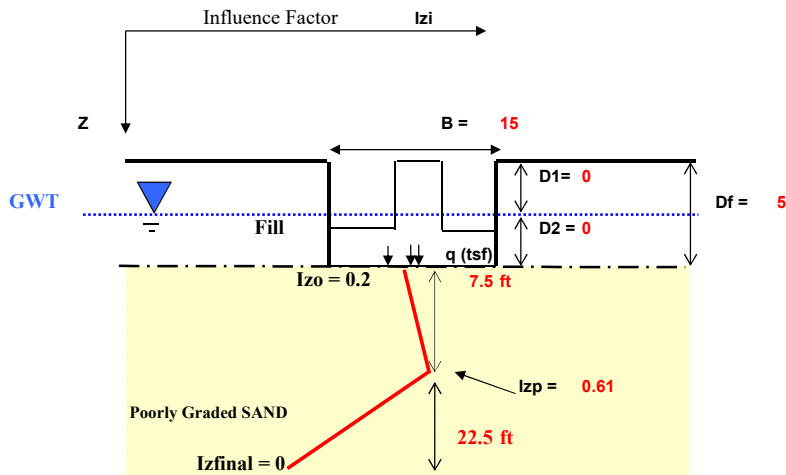
E_{si} = elastic modulus of soil layer i, tsf **481.25**

q = applied pressure at foundation, tsf **0.70** 1400

B = **15**

Df = **5**

GWT depth (ft) = **0**



$$I_{zo} = 0.10$$

$$I_{zp} = 0.61$$

$$I_{zp} - I_{zo} = 0.51$$

$$Z_{final} (ft) = 30$$

$$Z_{zp} (ft) = 7.5$$

$$Z_{final} - Z_{zp} = 22.5$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Sand and Silt
1.50	0.10	0.20	350	0.151	0.0004	
3.00	0.20	0.30	350	0.253	0.0007	
4.50	0.30	0.41	350	0.356	0.0010	
6.00	0.40	0.51	350	0.458	0.0013	
7.50	0.50	0.61	350	0.560	0.0016	
9.00	0.60	0.57	350	0.591	0.0017	
10.50	0.70	0.53	350	0.550	0.0016	
12.00	0.80	0.49	350	0.509	0.0015	
13.50	0.90	0.45	350	0.468	0.0013	
15.00	1.00	0.41	350	0.428	0.0012	
16.50	1.10	0.37	350	0.387	0.0011	
18.00	1.20	0.33	350	0.346	0.0010	
19.50	1.30	0.29	350	0.306	0.0009	
21.00	1.40	0.24	350	0.265	0.0008	
22.50	1.50	0.20	350	0.224	0.0006	Clacial Till
24.00	1.60	0.16	350	0.183	0.0005	
25.50	1.70	0.12	350	0.143	0.0004	
27.00	1.80	0.08	350	0.102	0.0003	
28.50	1.90	0.04	350	0.061	0.0002	
30.00	2.00	0.00	350	0.020	0.0001	

$$\sum Izi/E_{si} = 0.01817$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1[\Delta p/\sigma'_{i_{zp}}]^{1/2} = 0.611$

where, γ' (pcf) = effective unit weight 0.034 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
 0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{i_{zp}} = 0.5 \times B \times \gamma' + D \times \gamma' = 0.429$

2) Elastic Modulus, Esi (ksf)

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N (\text{blows/ft}) = 61$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt
 Clean, fine to medium sands and slightly silty sands
 Coarse sands and sands with little gravel
 Sandy gravel and gravel

L/B = 1

10N = 610
 17.5N = 1067.5
 25N = 1525
 30N = 1830

$$E_{si} (Ksf) = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.0120 ft	0.14 inches	<1	OK
Long Term	0.0168 ft	0.20 inches	<1	OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od}/\Delta p \geq 0.5$ **0.81**

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf **0.17 = $\gamma' D$**

where, γ_{moist} unit weight (pcf) **120**

γ_{sat} unit weight (pcf) = **131.00**

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ **1.0** (short-term) **1.4** (long-term)

t = time, years (end of construction) **0.1** **10** (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf **0.44**

Δz_i = depth increment i, 0.1B, ft **2.0**

I_{zi} = influence factor of soil layer i, fig. 3-4

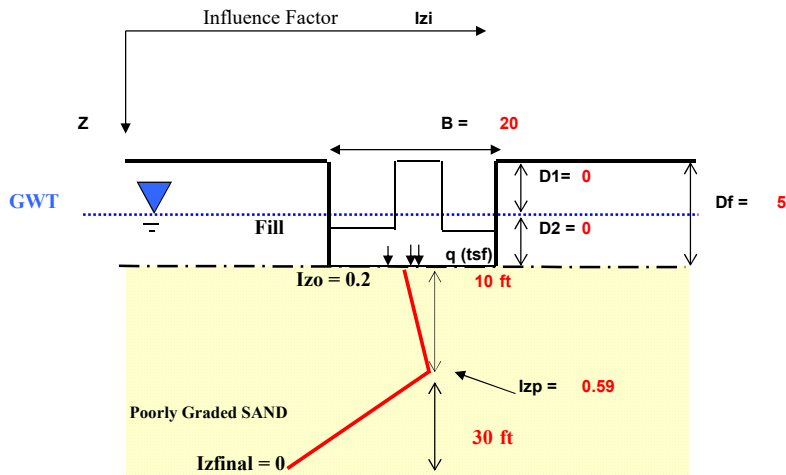
E_{si} = elastic modulus of soil layer i, tsf **481.25**

q = applied pressure at foundation, tsf **0.61** 1225

B = **20**

Df = **5**

GWT depth (ft) = **0**



$$I_{zo} = 0.10$$

$$I_{zp} = 0.59$$

$$I_{zp} - I_{zo} = 0.49$$

$$Z_{final} (ft) = 40$$

$$Z_{zp} (ft) = 10$$

$$Z_{final} - Z_{zp} = 30$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Sand and Silt
2.00	0.10	0.20	350	0.149	0.0004	
4.00	0.20	0.30	350	0.248	0.0007	
6.00	0.30	0.40	350	0.346	0.0010	
8.00	0.40	0.49	350	0.445	0.0013	
10.00	0.50	0.59	350	0.543	0.0016	
12.00	0.60	0.55	350	0.573	0.0016	
14.00	0.70	0.51	350	0.533	0.0015	
16.00	0.80	0.47	350	0.494	0.0014	
18.00	0.90	0.43	350	0.454	0.0013	
20.00	1.00	0.40	350	0.415	0.0012	
22.00	1.10	0.36	350	0.375	0.0011	
24.00	1.20	0.32	350	0.336	0.0010	
26.00	1.30	0.28	350	0.296	0.0008	
28.00	1.40	0.24	350	0.257	0.0007	
30.00	1.50	0.20	350	0.217	0.0006	Clayey Till
32.00	1.60	0.16	350	0.178	0.0005	
34.00	1.70	0.12	2250	0.138	0.0001	
36.00	1.80	0.08	2250	0.099	0.0000	
38.00	1.90	0.04	2250	0.059	0.0000	
40.00	2.00	0.00	2250	0.020	0.0000	

$$\sum Izi/E_{si} = 0.01688$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1[\Delta p/\sigma'_{i, zp}]^{1/2} = 0.593$

where, γ' (pcf) = effective unit weight 0.034
 0.06 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{i, zp} = 0.5 \times B \times \gamma' + D \times \gamma' = 0.515$

2) Elastic Modulus, Esi (ksf)

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N (\text{blows/ft}) = 61$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt
Clean, fine to medium sands and slightly silty sands
Coarse sands and sands with little gravel
Sandy gravel and gravel

L/B = 1

$$10N = 610$$

$$17.5N = 1067.5$$

$$25N = 1525$$

$$30N = 1830$$

$$Esi (Ksf) = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 \cdot C_2 \cdot \Delta p \cdot \Delta z_i \sum \frac{I_z I_{z_i}}{E_{si}}$$

Short Term

Long Term

0.0120 ft

0.0167 ft

0.14 inches

0.20 inches

<1

<1

OK

OK

where,

C₁ = correction to account for strain relief from embedment, 1-0.5σ'_{vd}/Δp>=0.5

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D, tsf

where,

γ_{sat} unit weight (pcf) = 131.00

γ_{moist} unit weight (pcf) = 120

C₂ = correction for time dependent increase in settlement, 1+0.2 log₁₀(t/0.1)

t = time, years (end of construction) = 0.1

Δp = net applied footing pressure, q-σ'_{vd}, tsf

Δz_i = depth increment i, 0.1B, ft

I_{zi} = influence factor of soil layer i, fig. 3-4

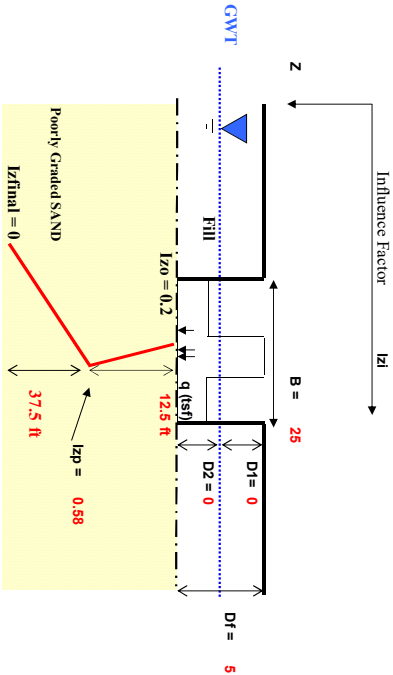
E_{si} = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

B = 25

Df = 5

GWT depth (ft) = 0



$l_{zo} = 0.10$
 $l_{zp} = 0.58$
 $l_{zp}-l_{zo} = 0.48$

$Z_{final(t)} = 50$
 $Z_{sp(t)} = 12.5$
 $Z_{final} - Z_{sp} = 37.5$

z, ft	z/B	l _{zi}	E _{si} , tsf	I _{za}	I _{za} /E _{si}
0.00	0.00	0.10			
2.50	0.10	0.20	350	0.148	0.0004
5.00	0.20	0.29	350	0.245	0.0007
7.50	0.30	0.39	350	0.341	0.0010
10.00	0.40	0.49	350	0.437	0.0012
12.50	0.50	0.58	350	0.534	0.0015
15.00	0.60	0.54	350	0.563	0.0016
17.50	0.70	0.50	350	0.524	0.0015
20.00	0.80	0.47	350	0.485	0.0014
22.50	0.90	0.43	350	0.446	0.0013
25.00	1.00	0.39	350	0.407	0.0012
27.50	1.10	0.35	350	0.369	0.0011
30.00	1.20	0.31	350	0.330	0.0009
32.50	1.30	0.27	350	0.291	0.0008
35.00	1.40	0.23	2250	0.252	0.0001
37.50	1.50	0.19	2250	0.213	0.0001
40.00	1.60	0.16	2250	0.175	0.0001
42.50	1.70	0.12	2250	0.136	0.0001
45.00	1.80	0.08	2250	0.097	0.0000
47.50	1.90	0.04	2250	0.058	0.0000
50.00	2.00	0.00	2250	0.019	0.0000

Sand and Silt
Glacial Till

$\sum I_{zi}/E_{si} = 0.01505$

1) l_{zp} = peak influence factor of soil layer i, Fig. 3-4 = $0.5+0.11ab/\sigma'_{i_{top}})^{1/2} = 0.582$

where, γ' (pcf) = effective unit weig **0.034** Using $\gamma' = \gamma$ sat - γ water (If soil is below GWT)
0.06 Using $\gamma' = \gamma$ moist (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing -effective overburden pressure at depth l_{zp} , tsf
 $\sigma'_{i_{top}} = 0.5 \times B \times \gamma' + D \times \gamma'$ **0.600**

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

Soil Type	N (blows/ft) = 100	L/B = 1
s, sandy silt, slightly cohesive silt-sand mixtu		10N = 1000
Clean, fine to medium sands and slightly silty sands		17.5N = 1750
Coarse sands and sands with little gravel		25N = 2500
Sandy gravel and gravel		30N = 3000
		E _{si} (ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 \cdot C_2 \cdot \Delta p \cdot \Delta z \cdot \sum \frac{I_z I_{z_i}}{E_{si}}$$

Short Term

Long Term

0.0022 ft

0.00720 ft

0.15 inches

0.20 inches

<1

<1

OK

OK

where,

C₁ = correction to account for strain relief from embedment, 1-0.5σ'_{vd}/Δp>=0.5

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D, tsf

where,

γ_{sat} unit weight (pcf) = 132.00

C₂ = correction for time dependent increase in settlement, 1+0.2 log₁₀(t/0.1)

t = time, years (end of constructi

Δp = net applied footing pressure, q-σ'_{vd}, tsf

Δz_i = depth increment i, 0.1B, ft

I_{zi} = influence factor of soil layer i, fig. 3-4

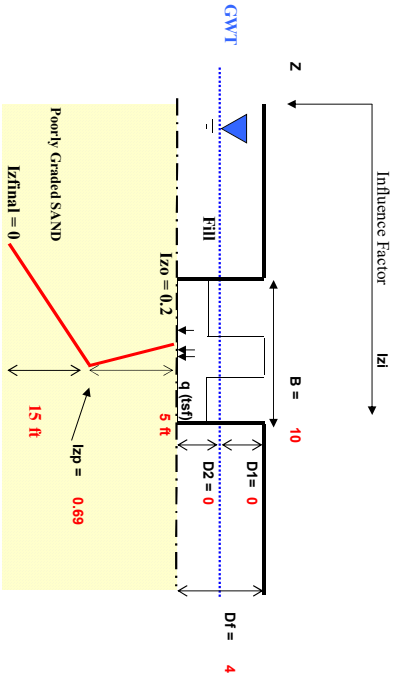
E_{si} = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

B = 10

Df = 4

GWT depth (ft) = 0



$l_{zo} = 0.10$
 $l_{zp} = 0.69$
 $l_{zp}-l_{zo} = 0.59$

$Z_{final (ft)} = 20$
 $Z_{sp (ft)} = 5$
 $Z_{final} - Z_{sp} = 15$

z, ft	z/B	l _{zi}	E _{si} , tsf	I _{za}	I _{za} /E _{si}
0.00	0.00	0.10			
1.00	0.10	0.22	610	0.159	0.0003
2.00	0.20	0.34	610	0.276	0.0005
3.00	0.30	0.45	610	0.394	0.0006
4.00	0.40	0.57	610	0.512	0.0008
5.00	0.50	0.69	610	0.629	0.0010
6.00	0.60	0.64	610	0.665	0.0011
7.00	0.70	0.60	610	0.619	0.0010
8.00	0.80	0.55	610	0.574	0.0009
9.00	0.90	0.50	610	0.528	0.0009
10.00	1.00	0.46	610	0.482	0.0008
11.00	1.10	0.41	610	0.436	0.0007
12.00	1.20	0.37	610	0.390	0.0006
13.00	1.30	0.32	610	0.344	0.0006
14.00	1.40	0.28	610	0.298	0.0005
15.00	1.50	0.23	610	0.252	0.0004
16.00	1.60	0.18	610	0.206	0.0003
17.00	1.70	0.14	610	0.161	0.0003
18.00	1.80	0.09	610	0.115	0.0002
19.00	1.90	0.05	610	0.069	0.0001
20.00	2.00	0.00	610	0.023	0.0000

Sand and Silt

$\sum I_{zi}/E_{si} = 0.01169$

1) l_{zp} = Peak influence factor of soil layer i, Fig. 3-4 = $0.5+0.11ab/\sigma'_{i_{zp}})^{1/2} = 0.688$

where, γ' (pcf) = effective unit weig **0.035** Using $\gamma' = \gamma$ sat - γ water (If soil is below GWT)
0.06 Using $\gamma' = \gamma$ moist (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing
 $\sigma'_{i_{zp}} = 0.5xB \times \gamma' + D \times \gamma' = 0.313$ -effective overburden pressure at depth l_{zp} , tsf

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

Soil Type	N (blows/ft) = 61	$L/B = 1$
s, sandy silt, slightly cohesive silt-sand mixtu		10N = 610
Clean, fine to medium sands and slightly silty sands		17.5N = 1067.5
Coarse sands and sands with little gravel		25N = 1525
Sandy gravel and gravel		30N = 1830

E_{si} (ksf) = **962.5**

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 11-10-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$p_i = C_i * C_i * \Delta p * \Delta z_i \sum z_i /E_{si}$	Short Term	<u>0.0122</u> ft	<u>0.15</u> inches	<1	OK
	Long Term	<u>0.0170</u> ft	<u>0.20</u> inches	<1	OK

where,

C_1 = correction to account for strain relief from embedment, $1-0.5 \sigma'_{od}/\Delta p > 0$;

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D, is

where, γ_{moist} unit weight (pcf) **120**

γ_{sat} unit weight (pcf) = **132.00**

$$C_t = \text{correction for time dependent increase in settlement, } 1 + 0.2 \log_{10} (t / 0.1)$$

	0.1 (short term)	10 (long term)
t = time, years	(end of construction)	(end of construction)

 $\Delta p = \text{net applied footing pressure, } q - \sigma'_{\text{old}}, \text{ tsf}$ Δz_i = depth increment i , 0.1 B, ft

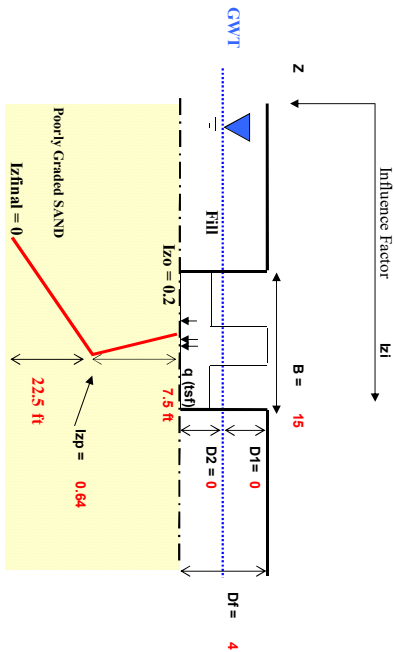
Izi = influence factor of soil layer i, fig. 3-4

 E_{si} = elastic modulus of soil layer i , tsf

q = applied pressure at foundation, tsf

$$B = 15$$
$$Df = 4$$

GW1 depth (ft) = 0



$I_{zo} = 0.10$
 $I_{zp} = 0.64$
 $I_{zp} - I_{zo} = 0.54$

$Z_{final(t)} = 30$
 $Z_{np(t)} = 7.5$
 $Z_{final} - Z_{np} = 22.5$

z, ft	z/B	I _{zi}	E _{si} , tsf	I _{za}	I _{za} /E _{si}
0.00	0.00	0.10			
1.50	0.10	0.21	610	0.154	0.0003
3.00	0.20	0.32	610	0.263	0.0004
4.50	0.30	0.43	610	0.371	0.0006
6.00	0.40	0.53	610	0.480	0.0008
7.50	0.50	0.64	610	0.588	0.0010
9.00	0.60	0.60	610	0.621	0.0010
10.50	0.70	0.56	610	0.578	0.0009
12.00	0.80	0.51	610	0.535	0.0009
13.50	0.90	0.47	610	0.492	0.0008
15.00	1.00	0.43	610	0.450	0.0007
16.50	1.10	0.39	610	0.407	0.0007
18.00	1.20	0.34	610	0.364	0.0006
19.50	1.30	0.30	610	0.321	0.0005
21.00	1.40	0.26	610	0.278	0.0005
22.50	1.50	0.21	610	0.236	0.0004
24.00	1.60	0.17	610	0.193	0.0003
25.50	1.70	0.13	610	0.150	0.0002
27.00	1.80	0.09	610	0.107	0.0002
28.50	1.90	0.04	610	0.064	0.0001
30.00	2.00	0.00	610	0.021	0.0000

Sand and Silt

$\sum I_{zi}/E_{si} = 0.01094$

1) I_{zp} = peak influence factor of soil layer i, fig. 3-4 = $0.5 + 0.11 \Delta np \sigma'_{top}{}^{1/2} = 0.642$

where, γ' (pcf) = effective unit wei; Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf

$\sigma'_{top} = 0.5 \times B \times \gamma' + D \times \gamma' = 0.400$

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf), After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978).

Soil Type	N (blows/ft) = 61	L/B = 1
s ₁ sandy silts, slightly cohesive silt-sand mixt		10N = 610
Clean, fine to medium sands and slightly silty sands		17.5N = 1067.5
Coarse sands and sands with little gravel		25N = 1525
Sandy gravel and gravel		30N = 1830
		E _{si} (ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis", 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 \cdot C_2 \cdot \Delta p \cdot \Delta z_i \sum \frac{I_z}{E_{si}}$$

Short Term

Long Term

$\frac{0.0120}{0.0168}$ ft

$\frac{0.14}{0.20}$ inches

<1

<1

OK

OK

where,

C_1 = correction to account for strain relief from embedment, $1-0.5 \sigma'_{vd}/\Delta p \geq 0$;

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D , ts

where,

$$\gamma_{ moist}$$

unit weight (pcf)

120

132.00

C_2 = correction for time dependent increase in settlement, $1+0.2 \log_{10} (t / 0.1)$

t = time, years (end of construct)

Δp = net applied footing pressure, $q - \sigma'_{vd}$, tsf

Δz_i = depth increment i , 0.1B, ft

I_z = influence factor of soil layer i , fig. 3-4

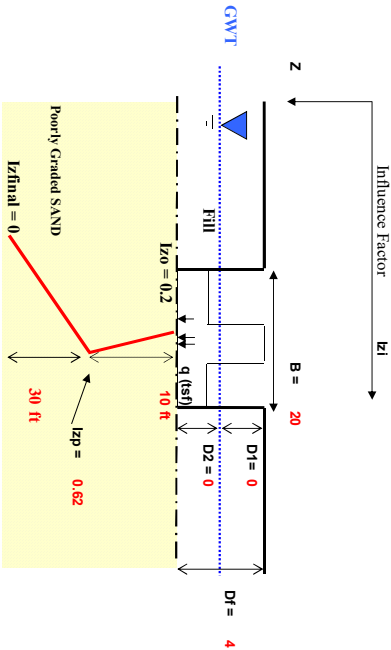
E_{si} = elastic modulus of soil layer i , tsf

q = applied pressure at foundation, tsf

B = 20

D_f = 4

GWT depth (ft) = 0



$I_{zo} = 0.10$
 $I_{zp} = 0.62$
 $I_{zp} - I_{zo} = 0.52$

$Z_{final(t)} = 40$
 $Z_{np(t)} = 10$
 $Z_{final} - Z_{np} = 30$

z, ft	z/B	I _{zi}	E _{si} , tsf	I _{za}	I _{za} /E _{si}
0.00	0.00	0.10			
2.00	0.10	0.20	610	0.152	0.0002
4.00	0.20	0.31	610	0.255	0.0004
6.00	0.30	0.41	610	0.358	0.0006
8.00	0.40	0.51	610	0.462	0.0008
10.00	0.50	0.62	610	0.565	0.0009
12.00	0.60	0.58	610	0.596	0.0010
14.00	0.70	0.53	610	0.555	0.0009
16.00	0.80	0.49	610	0.514	0.0008
18.00	0.90	0.45	610	0.473	0.0008
20.00	1.00	0.41	610	0.432	0.0007
22.00	1.10	0.37	610	0.390	0.0006
24.00	1.20	0.33	610	0.349	0.0006
26.00	1.30	0.29	610	0.308	0.0005
28.00	1.40	0.25	610	0.267	0.0004
30.00	1.50	0.21	610	0.226	0.0004
32.00	1.60	0.16	610	0.185	0.0003
34.00	1.70	0.12	2250	0.144	0.0001
36.00	1.80	0.08	2250	0.103	0.0000
38.00	1.90	0.04	2250	0.062	0.0000
40.00	2.00	0.00	2250	0.021	0.0000

Sand and Silt

$\sum I_{zi}/E_{si} = 0.01012$

1) I_{zp} = peak influence factor of soil layer i, **Fig. 3-4** = $0.5 + 0.11 \Delta np \sigma'_{top}^{-1/2}$ = 0.616

where, γ' (pcf) = effective unit wei; **0.035** Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing
 $\sigma'_{top} = 0.5 \times B \times \gamma' + D \times \gamma'$ **0.487** -effective overburden pressure at depth I_{zp} , tsf

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf). After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978).

Soil Type	N (blows/ft) = 100	L/B = 1
s ₁ sandy silts, slightly cohesive silt-sand mixt		10N = 1000
Clean, fine to medium sands and slightly silty sands		17.5N = 1750
Coarse sands and sands with little gravel		25N = 2500
Sandy gravel and gravel		30N = 3000
		E _{si} (ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.0123 ft	0.15 inches	<1	OK
Long Term	0.0172 ft	0.21 inches	<1	OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od}/\Delta p \geq 0.5$ 0.77

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf 0.17 = $\gamma' D$

where, γ_{moist} unit weight (pcf) 120

γ_{sat} unit weight (pcf) = 131.00

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ 1.0 (short-term) 1.4 (long-term)

t = time, years (end of construction) 0.1 10 (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf 0.38

Δz_i = depth increment i, 0.1B, ft 6.5

I_{zi} = influence factor of soil layer i, fig. 3-4

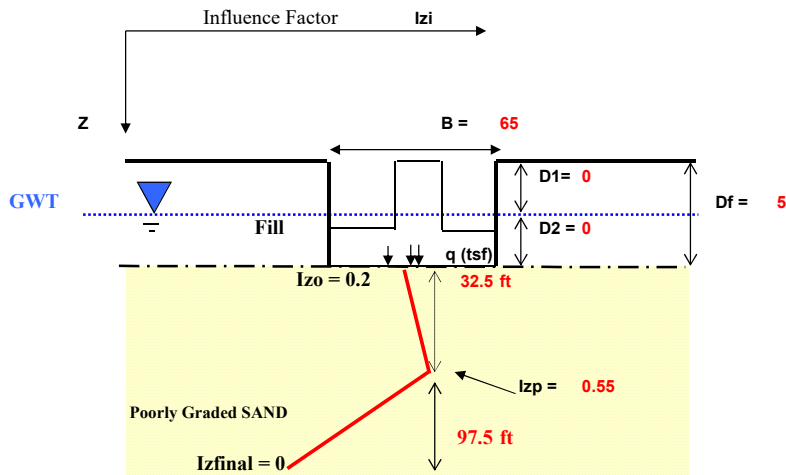
E_{si} = elastic modulus of soil layer i, tsf 481.25

q = applied pressure at foundation, tsf 0.55 1100

B = 65

Df = 5

GWT depth (ft) = 0



$$I_{zo} = 0.10$$

$$I_{zp} = 0.55$$

$$I_{zp} - I_{zo} = 0.45$$

$$Z_{final} (ft) = 130$$

$$Z_{zp} (ft) = 32.5$$

$$Z_{final} - Z_{zp} = 97.5$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Sand and Silt Gravel Till Weather
6.50	0.10	0.19	350	0.145	0.0004	
13.00	0.20	0.28	350	0.236	0.0007	
19.50	0.30	0.37	350	0.327	0.0009	
26.00	0.40	0.46	350	0.418	0.0012	
32.50	0.50	0.55	350	0.509	0.0015	
39.00	0.60	0.52	2250	0.536	0.0002	
45.50	0.70	0.48	2250	0.499	0.0002	
52.00	0.80	0.44	2250	0.462	0.0002	
58.50	0.90	0.41	2250	0.425	0.0002	
65.00	1.00	0.37	2250	0.388	0.0002	Bedrock
71.50	1.10	0.33	2500	0.351	0.0001	
78.00	1.20	0.30	2500	0.314	0.0001	
84.50	1.30	0.26	2500	0.277	0.0001	
91.00	1.40	0.22	2500	0.240	0.0001	
97.50	1.50	0.18	2500	0.203	0.0001	
104.00	1.60	0.15	2500	0.166	0.0001	
110.50	1.70	0.11	2500	0.129	0.0001	
117.00	1.80	0.07	2500	0.092	0.0000	
123.50	1.90	0.04	2500	0.055	0.0000	
130.00	2.00	0.00	2500	0.018	0.0000	

$$\sum Izi/E_{si} = 0.00644$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1[\Delta p/\sigma'_{i_{zp}}]^{1/2} = 0.554$

where, γ' (pcf) = effective unit weight 0.034 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
 0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{i_{zp}} = 0.5 \times B \times \gamma' + D \times \gamma' = 1.286$

2) Elastic Modulus, Esi (ksf)

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N (\text{blows/ft}) = 100$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt
 Clean, fine to medium sands and slightly silty sands
 Coarse sands and sands with little gravel
 Sandy gravel and gravel

L/B = 1

10N = 1000
 17.5N = 1750
 25N = 2500
 30N = 3000

$$E_{si} (Ksf) = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	<u>0.0117</u> ft	<u>0.14</u> inches	<1	OK
Long Term	<u>0.0164</u> ft	<u>0.20</u> inches	<1	OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od} / \Delta p \geq 0.5$ **0.83**

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf **$0.17 = \gamma' D$**

where, γ_{moist} unit weight (pcf) **120**

γ_{sat} unit weight (pcf) = **132.00**

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ **1.0** (short-term) **1.4** (long-term)

t = time, years (end of construction) **0.1** **10** (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf **0.53**

Δz_i = depth increment i, 0.1B, ft **6.0**

I_{zi} = influence factor of soil layer i, fig. 3-4

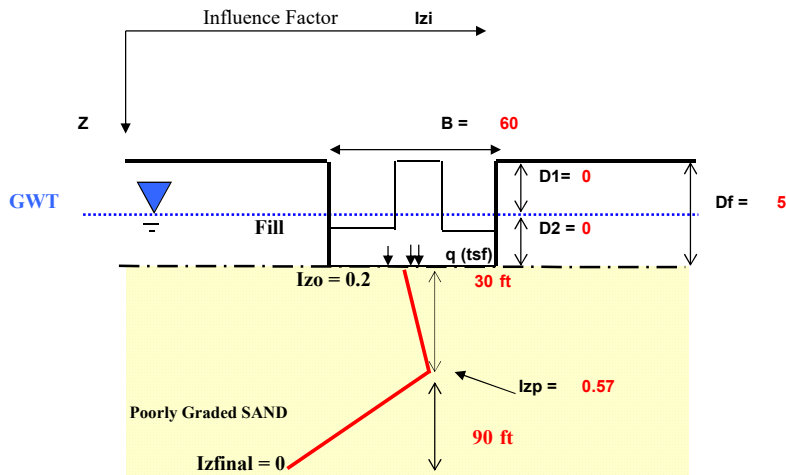
E_{si} = elastic modulus of soil layer i, tsf **481.25**

q = applied pressure at foundation, tsf **0.70** 1400 psf

B = **60**

Df = **5**

GWT depth (ft) = **0**



$$I_{zo} = 0.10$$

$$I_{zp} = 0.57$$

$$I_{zp} - I_{zo} = 0.47$$

$$Z_{final} (ft) = 120$$

$$Z_{zp} (ft) = 30$$

$$Z_{final} - Z_{zp} = 90$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Sand and Silt
6.00	0.10	0.19	610	0.147	0.0002	
12.00	0.20	0.29	610	0.240	0.0004	
18.00	0.30	0.38	610	0.333	0.0005	
24.00	0.40	0.47	610	0.426	0.0007	
30.00	0.50	0.57	610	0.519	0.0009	
36.00	0.60	0.53	2250	0.547	0.0002	
42.00	0.70	0.49	2250	0.509	0.0002	
48.00	0.80	0.45	2250	0.471	0.0002	
54.00	0.90	0.41	2250	0.434	0.0002	
60.00	1.00	0.38	2250	0.396	0.0002	Glacial Till
66.00	1.10	0.34	2250	0.358	0.0002	
72.00	1.20	0.30	3000	0.321	0.0001	
78.00	1.30	0.26	3000	0.283	0.0001	
84.00	1.40	0.23	3000	0.245	0.0001	
90.00	1.50	0.19	3000	0.207	0.0001	
96.00	1.60	0.15	3000	0.170	0.0001	
102.00	1.70	0.11	3000	0.132	0.0000	
108.00	1.80	0.08	3000	0.094	0.0000	
114.00	1.90	0.04	3000	0.057	0.0000	
120.00	2.00	0.00	2500	0.019	0.0000	Bedrock

$$\sum Izi/E_{si} = 0.00445$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1 |\Delta p / \sigma'_{i,zp}|^{1/2} = 0.566$

where, γ' (pcf) = effective unit weight 0.035 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
 0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{i,zp} = 0.5 \times B \times \gamma' + D \times \gamma' = 1.218$

2) **Elastic Modulus, Esi (ksf)**

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N \text{ (blows/ft)} = 100$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt
 Clean, fine to medium sands and slightly silty sands
 Coarse sands and sands with little gravel
 Sandy gravel and gravel

L/B = 1

10N = 1000
 17.5N = 1750
 25N = 2500
 30N = 3000

$$Esi \text{ (Ksf)} = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.0237 ft	0.28 inches	<1	OK
Long Term	0.0332 ft	0.40 inches	<1	OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od}/\Delta p \geq 0.5$ **0.94**

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf **0.17** = $\gamma' D$

where, γ_{moist} unit weight (pcf) **120**

γ_{sat} unit weight (pcf) = **132.00**

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ **1.0** (short-term) **1.4** (long-term)

t = time, years (end of construction) **0.1** **10** (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf **1.45**

Δz_i = depth increment i, 0.1B, ft **1.5**

I_{zi} = influence factor of soil layer i, fig. 3-4

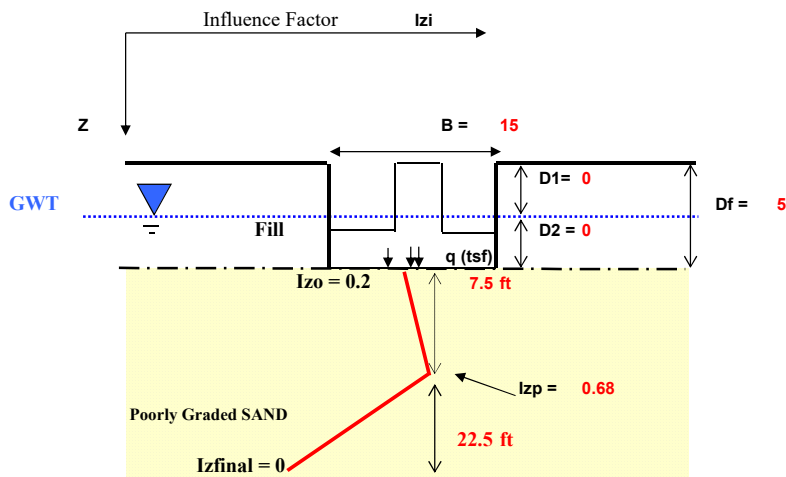
E_{si} = elastic modulus of soil layer i, tsf **481.25**

q = applied pressure at foundation, tsf **1.63** 3250

B = **15**

Df = **5**

GWT depth (ft) = **0**



$$I_{zo} = 0.10$$

$$I_{zp} = 0.68$$

$$I_{zp} - I_{zo} = 0.58$$

$$Z_{final} (ft) = 30$$

$$Z_{zp} (ft) = 7.5$$

$$Z_{final} - Z_{zp} = 22.5$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi
0.00	0.00	0.10			
1.50	0.10	0.22	610	0.158	0.0003
3.00	0.20	0.33	610	0.275	0.0005
4.50	0.30	0.45	610	0.391	0.0006
6.00	0.40	0.57	610	0.508	0.0008
7.50	0.50	0.68	610	0.624	0.0010
9.00	0.60	0.64	610	0.660	0.0011
10.50	0.70	0.59	610	0.614	0.0010
12.00	0.80	0.55	610	0.569	0.0009
13.50	0.90	0.50	610	0.523	0.0009
15.00	1.00	0.46	610	0.478	0.0008
16.50	1.10	0.41	610	0.432	0.0007
18.00	1.20	0.36	610	0.387	0.0006
19.50	1.30	0.32	610	0.341	0.0006
21.00	1.40	0.27	610	0.296	0.0005
22.50	1.50	0.23	610	0.250	0.0004
24.00	1.60	0.18	610	0.205	0.0003
25.50	1.70	0.14	610	0.159	0.0003
27.00	1.80	0.09	610	0.114	0.0002
28.50	1.90	0.05	610	0.068	0.0001
30.00	2.00	0.00	610	0.023	0.0000

Sand and Silt

$$\sum Izi/E_{si} = 0.0116$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1[\Delta p/\sigma'_{i_{zp}}]^{1/2} = 0.683$

where, γ' (pcf) = effective unit weight 0.035
0.06 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing -effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{i_{zp}} = 0.5 \times B \times \gamma' + D \times \gamma' = 0.435$

2) Elastic Modulus, Esi (ksf)

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N (\text{blows/ft}) = 61$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt
Clean, fine to medium sands and slightly silty sands
Coarse sands and sands with little gravel
Sandy gravel and gravel

L/B = 1

10N = 610
17.5N = 1067.5
25N = 1525
30N = 1830

$$Esi (Ksf) = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 \cdot C_2 \cdot \Delta p \cdot \Delta z_i \sum \frac{I_z I_{z_i}}{E_{si}}$$

Short Term

Long Term

0.0238 ft

0.0333 ft

0.29 inches

0.40 inches

<1

<1

OK

OK

where,

C₁ = correction to account for strain relief from embedment, 1-0.5σ'_{vd}/Δp>=0.5

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D, tsf

where,

γ_{moist} unit weight (pcf) = 120

C₂ = correction for time dependent increase in settlement, 1+0.2 log₁₀(t/0.1)

t = time, years (end of construction) 0.1

Δp = net applied footing pressure, q-σ'_{vd}, tsf

Δz_i = depth increment i, 0.1B, ft

I_{zi} = influence factor of soil layer i, fig. 3-4

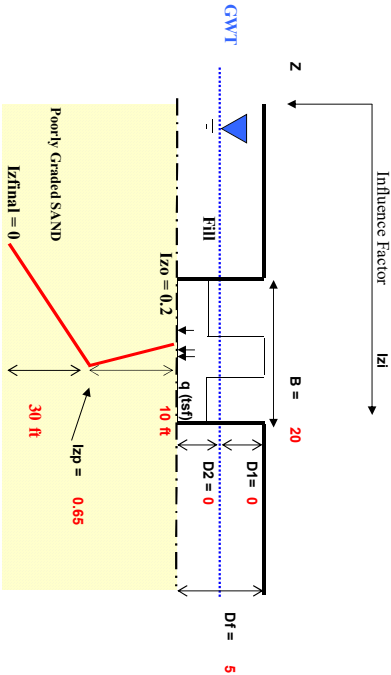
E_{si} = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

B = 20

Df = 5

GWT depth (ft) = 0



$I_{zp} = 0.10$
 $I_{zp} = 0.65$
 $I_{zp} - I_{zo} = 0.55$

$Z_{final(t)} = 40$
 $Z_{sp(t)} = 10$
 $Z_{final} - Z_{sp} = 30$

z, ft	z/B	I _{zi}	E _{si} , tsf	I _{za}	I _{za} /E _{si}
0.00	0.00	0.10			
2.00	0.10	0.21	610	0.155	0.0003
4.00	0.20	0.32	610	0.266	0.0004
6.00	0.30	0.43	610	0.376	0.0006
8.00	0.40	0.54	610	0.486	0.0008
10.00	0.50	0.65	610	0.597	0.0010
12.00	0.60	0.61	610	0.630	0.0010
14.00	0.70	0.56	610	0.587	0.0010
16.00	0.80	0.52	610	0.543	0.0009
18.00	0.90	0.48	610	0.500	0.0008
20.00	1.00	0.43	610	0.456	0.0007
22.00	1.10	0.39	610	0.413	0.0007
24.00	1.20	0.35	610	0.369	0.0006
26.00	1.30	0.30	610	0.326	0.0005
28.00	1.40	0.26	610	0.282	0.0005
30.00	1.50	0.22	610	0.239	0.0004
32.00	1.60	0.17	610	0.196	0.0003
34.00	1.70	0.13	2250	0.152	0.0001
36.00	1.80	0.09	2250	0.109	0.0000
38.00	1.90	0.04	2250	0.065	0.0000
40.00	2.00	0.00	2250	0.022	0.0000

Sand and Silt
Till
Glacial

1) I_{zp} = peak influence factor of soil layer i, Fig. 3-4 = $0.5 + 0.1 I_{ap} / \sigma'_{top} \cdot I_{zi}^{1/2}$ = 0.652

$\sum I_{zi} / E_{si}$ = 0.01068

where, γ' (pcf) = effective unit weight 0.035
Using $\gamma' = \gamma$ sat - γ water (If soil is below GWT)
Using $\gamma' = \gamma$ moist (If soil is above GWT) 0.06

Axisymmetric (L/B=1) - Spread/column footing
 $\sigma'_{top} = 0.5 \gamma B \times \gamma' + D \times \gamma' = 0.522$
-effective overburden pressure at depth I_{zp} , tsf

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

Soil Type	N (blows/ft) = 100	L/B = 1
s, sandy silts, slightly cohesive silt-sand mixtu		10N = 1000
Clean, fine to medium sands and slightly silty sands		17.5N = 1750
Coarse sands and sands with little gravel		25N = 2500
Sandy gravel and gravel		30N = 3000
		E _{si} (ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$p_i = C_i * C_i^* \Delta p^* \Delta z_i \sum z_i /E_{si}$	Short Term	<u>0.0236</u> ft	<u>0.28</u> inches < 1	OK
	Long Term	<u>0.0331</u> ft	<u>0.40</u> inches < 1	OK

where,

C_1 = correction to account for strain relief from embedment, $1-0.5 \sigma'_{od}/\Delta p > 0$;

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D, is

where, γ_{moist} unit weight (pcf) **120**

γ_{sat} unit weight (pcf) = **132.00**

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10} (t / 0.1)$

	0.1 (short term)	10 (long term)
t = time, years	(end of construction)	(end of construction)

 Δp = net applied footing pressure, q- σ'_{old} , tsf Δz_i = depth increment i , 0.1 B, ft

Izi = influence factor of soil layer i, fig. 3-4

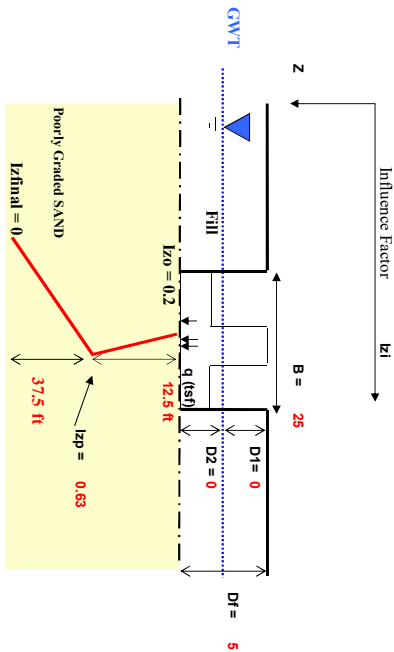
E_{si} = elastic modulus of soil layer i , tsf

q = applied pressure at foundation, tsf

$B = 25$

$$Df = 5$$

GWL depth (ft) = 0



$I_{zo} = 0.10$
 $I_{zp} = 0.63$
 $I_{zp} - I_{zo} = 0.53$

$Z_{final(t)} = 50$
 $Z_{np(t)} = 12.5$
 $Z_{final} - Z_{np} = 37.5$

z, ft	z/B	I _{zi}	E _{si} , tsf	I _{za}	I _{za} /E _{si}	
0.00	0.00	0.10				
2.50	0.10	0.21	610	0.153	0.0003	
5.00	0.20	0.31	610	0.260	0.0004	
7.50	0.30	0.42	610	0.366	0.0006	
10.00	0.40	0.53	610	0.473	0.0008	
12.50	0.50	0.63	610	0.580	0.0010	
15.00	0.60	0.59	610	0.612	0.0010	
17.50	0.70	0.55	610	0.570	0.0009	
20.00	0.80	0.51	610	0.527	0.0009	
22.50	0.90	0.46	610	0.485	0.0008	
25.00	1.00	0.42	610	0.443	0.0007	
27.50	1.10	0.38	610	0.401	0.0007	
30.00	1.20	0.34	610	0.359	0.0006	
32.50	1.30	0.30	610	0.316	0.0005	
35.00	1.40	0.25	2250	0.274	0.0001	
37.50	1.50	0.21	2250	0.232	0.0001	
40.00	1.60	0.17	2250	0.190	0.0001	
42.50	1.70	0.13	2250	0.148	0.0001	
45.00	1.80	0.08	2250	0.105	0.0000	
47.50	1.90	0.04	2250	0.063	0.0000	
50.00	2.00	0.00	2250	0.021	0.0000	

Sand and Silt
Glacial Till

$\sum I_{zi}/E_{si} = 0.00955$

1) I_{zp} = peak influence factor of soil layer i, **Fig. 3-4 = 0.5+0.11Δnp/σ_{avg}^{1/2} = 0.633**

where, γ' (pcf) = effective unit wei; **0.035** Using γ' = γ sat - γ water (If soil is below GWT)
0.06 Using γ' = γ moist (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing

$\sigma'_{top} = 0.5 \times B \times \gamma' + D \times \gamma'$ **0.609**

-effective overburden pressure at depth I_{zp}, tsf

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf). After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978).

Soil Type	N (blows/ft) = 100	L/B = 1
s ₁ sandy silts, slightly cohesive silt-sand mixt		10N = 1000
Clean, fine to medium sands and slightly silty sands		17.5N = 1750
Coarse sands and sands with little gravel		25N = 2500
Sandy gravel and gravel		30N = 3000
		E _{si} (ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.0239 ft	0.29 inches	<1	OK
Long Term	0.0334 ft	0.40 inches	<1	OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od} / \Delta p \geq 0.5$ **0.91**

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf **0.17 = $\gamma' D$**

where, γ_{moist} unit weight (pcf) **120**

γ_{sat} unit weight (pcf) = **131.00**

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ **1.0** (short-term) **1.4** (long-term)

t = time, years (end of construction) **0.1** **10** (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf **0.92**

Δz_i = depth increment i, 0.1B, ft **1.5**

I_{zi} = influence factor of soil layer i, fig. 3-4

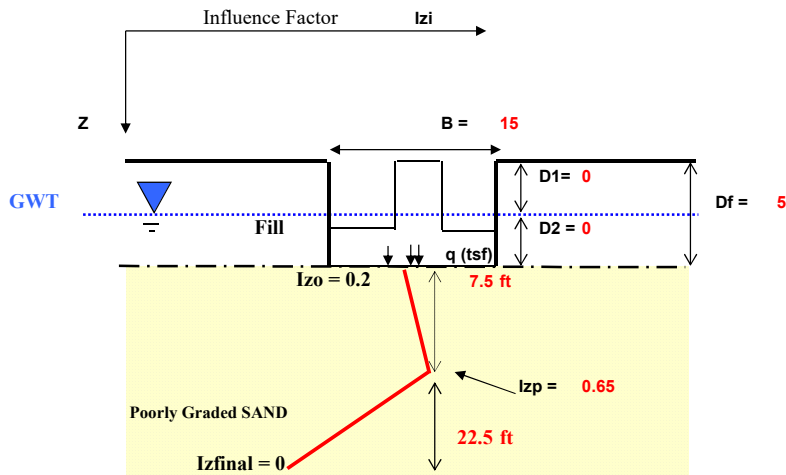
E_{si} = elastic modulus of soil layer i, tsf **481.25**

q = applied pressure at foundation, tsf **1.09** 2175

B = **15**

Df = **5**

GWT depth (ft) = **0**



$$I_{zo} = 0.10$$

$$I_{zp} = 0.65$$

$$I_{zp} - I_{zo} = 0.55$$

$$Z_{final} (ft) = 30$$

$$Z_{zp} (ft) = 7.5$$

$$Z_{final} - Z_{zp} = 22.5$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Sand and Silt
1.50	0.10	0.21	350	0.155	0.0004	
3.00	0.20	0.32	350	0.264	0.0008	
4.50	0.30	0.43	350	0.373	0.0011	
6.00	0.40	0.54	350	0.482	0.0014	
7.50	0.50	0.65	350	0.592	0.0017	
9.00	0.60	0.60	350	0.625	0.0018	
10.50	0.70	0.56	350	0.582	0.0017	
12.00	0.80	0.52	350	0.538	0.0015	
13.50	0.90	0.47	350	0.495	0.0014	
15.00	1.00	0.43	350	0.452	0.0013	
16.50	1.10	0.39	350	0.409	0.0012	
18.00	1.20	0.34	350	0.366	0.0010	
19.50	1.30	0.30	350	0.323	0.0009	
21.00	1.40	0.26	350	0.280	0.0008	
22.50	1.50	0.22	350	0.237	0.0007	Clacial Till
24.00	1.60	0.17	350	0.194	0.0006	
25.50	1.70	0.13	350	0.151	0.0004	
27.00	1.80	0.09	350	0.108	0.0003	
28.50	1.90	0.04	350	0.065	0.0002	
30.00	2.00	0.00	350	0.022	0.0001	

$$\sum Izi/E_{si} = 0.01918$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1[\Delta p/\sigma'_{i, zp}]^{1/2} = 0.646$

where, γ' (pcf) = effective unit weight 0.034
 0.06 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{i, zp} = 0.5 \times B \times \gamma' + D \times \gamma' = 0.429$

2) Elastic Modulus, Esi (ksf)

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N (\text{blows/ft}) = 61$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt
Clean, fine to medium sands and slightly silty sands
Coarse sands and sands with little gravel
Sandy gravel and gravel

L/B = 1

10N = 610
17.5N = 1067.5
25N = 1525
30N = 1830

$$E_{si} (Ksf) = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.0236 ft	0.28 inches	<1	OK
Long Term	0.0330 ft	0.40 inches	<1	OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od}/\Delta p \geq 0.5$ **0.89**

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf **0.17 = $\gamma' D$**

where, γ_{moist} unit weight (pcf) **120**

γ_{sat} unit weight (pcf) = **131.00**

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ **1.0** (short-term) **1.4** (long-term)

t = time, years (end of construction) **0.1** **10** (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf **0.75**

Δz_i = depth increment i, 0.1B, ft **2.0**

I_{zi} = influence factor of soil layer i, fig. 3-4

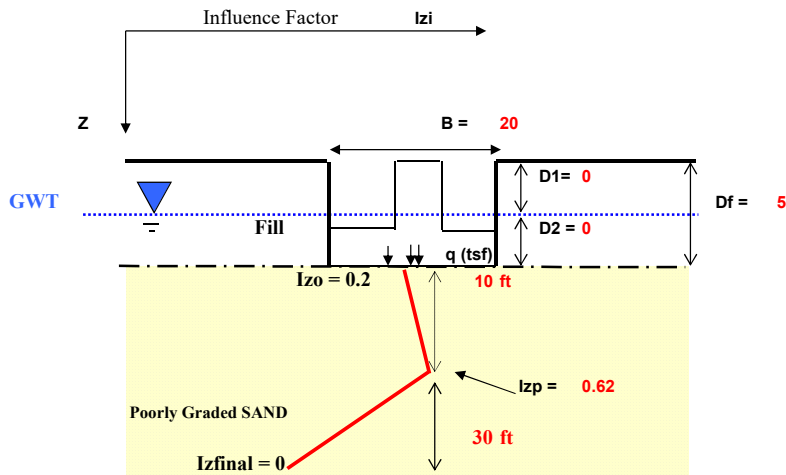
E_{si} = elastic modulus of soil layer i, tsf **481.25**

q = applied pressure at foundation, tsf **0.93** 1850

B = **20**

Df = **5**

GWT depth (ft) = **0**



$$I_{zo} = 0.10$$

$$I_{zp} = 0.62$$

$$I_{zp} - I_{zo} = 0.52$$

$$Z_{final} (ft) = 40$$

$$Z_{zp} (ft) = 10$$

$$Z_{final} - Z_{zp} = 30$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Sand and Silt
2.00	0.10	0.20	350	0.152	0.0004	
4.00	0.20	0.31	350	0.256	0.0007	
6.00	0.30	0.41	350	0.361	0.0010	
8.00	0.40	0.52	350	0.465	0.0013	
10.00	0.50	0.62	350	0.569	0.0016	
12.00	0.60	0.58	350	0.600	0.0017	
14.00	0.70	0.54	350	0.559	0.0016	
16.00	0.80	0.50	350	0.518	0.0015	
18.00	0.90	0.46	350	0.476	0.0014	
20.00	1.00	0.41	350	0.435	0.0012	
22.00	1.10	0.37	350	0.393	0.0011	
24.00	1.20	0.33	350	0.352	0.0010	
26.00	1.30	0.29	350	0.311	0.0009	
28.00	1.40	0.25	350	0.269	0.0008	
30.00	1.50	0.21	350	0.228	0.0007	Clayey Till
32.00	1.60	0.17	350	0.186	0.0005	
34.00	1.70	0.12	2250	0.145	0.0001	
36.00	1.80	0.08	2250	0.104	0.0000	
38.00	1.90	0.04	2250	0.062	0.0000	
40.00	2.00	0.00	2250	0.021	0.0000	

$$\sum Izi/E_{si} = 0.01766$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1[\Delta p/\sigma'_{i, zp}]^{1/2} = 0.621$

where, γ' (pcf) = effective unit weight 0.034
 0.06 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{i, zp} = 0.5 \times B \times \gamma' + D \times \gamma' = 0.515$

2) Elastic Modulus, Esi (ksf)

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N (\text{blows/ft}) = 61$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt
Clean, fine to medium sands and slightly silty sands
Coarse sands and sands with little gravel
Sandy gravel and gravel

L/B = 1

10N = 610
17.5N = 1067.5
25N = 1525
30N = 1830

$$E_{si} (Ksf) = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 \cdot C_2 \cdot \Delta p \cdot \Delta z_i \cdot \sum \frac{I_z I_{z_i}}{E_{si}}$$

Short Term

Long Term

0.0237 ft

0.0332 ft

0.28 inches

0.40 inches

<1

<1

OK

OK

where,

C₁ = correction to account for strain relief from embedment, 1-0.5σ'_{vd}/Δp>=0.5

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D, tsf

where,

γ_{moist} unit weight (pcf) = 120

γ_{sat} unit weight (pcf) = 131.00

C₂ = correction for time dependent increase in settlement, 1+0.2 log₁₀(t/0.1)

t = time, years (end of construction) 0.1

Δp = net applied footing pressure, q-σ'_{vd}, tsf

Δz_i = depth increment i, 0.1B, ft

I_{zi} = influence factor of soil layer i, fig. 3-4

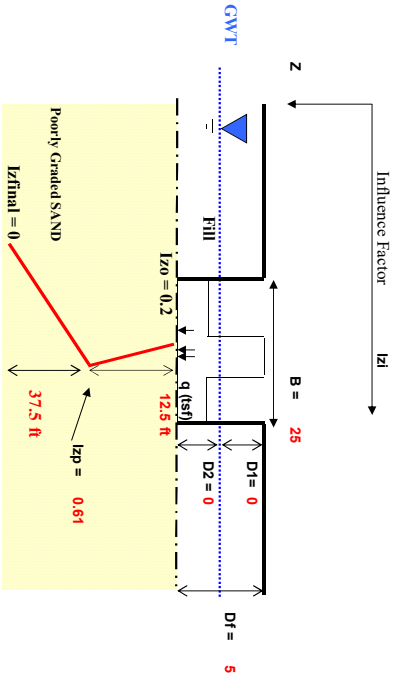
E_{si} = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

B = 25

Df = 5

GWT depth (ft) = 0



$l_{zo} = 0.10$
 $l_{zp} = 0.61$
 $l_{zp}-l_{zo} = 0.51$

$Z_{final(t)} = 50$
 $Z_{sp(t)} = 12.5$
 $Z_{final} - Z_{sp} = 37.5$

z, ft	z/B	l _{zi}	E _{si} , tsf	I _{za}	I _{za} /E _{si}
0.00	0.00	0.10			
2.50	0.10	0.20	350	0.151	0.0004
5.00	0.20	0.30	350	0.252	0.0007
7.50	0.30	0.40	350	0.354	0.0010
10.00	0.40	0.51	350	0.455	0.0013
12.50	0.50	0.61	350	0.557	0.0016
15.00	0.60	0.57	350	0.587	0.0017
17.50	0.70	0.53	350	0.547	0.0016
20.00	0.80	0.49	350	0.506	0.0014
22.50	0.90	0.45	350	0.466	0.0013
25.00	1.00	0.40	350	0.425	0.0012
27.50	1.10	0.36	350	0.385	0.0011
30.00	1.20	0.32	350	0.344	0.0010
32.50	1.30	0.28	350	0.304	0.0009
35.00	1.40	0.24	2250	0.263	0.0001
37.50	1.50	0.20	2250	0.223	0.0001
40.00	1.60	0.16	2250	0.182	0.0001
42.50	1.70	0.12	2250	0.142	0.0001
45.00	1.80	0.08	2250	0.101	0.0000
47.50	1.90	0.04	2250	0.061	0.0000
50.00	2.00	0.00	2250	0.020	0.0000

Sand and Silt
Glacial Till

$\sum I_{zi}/E_{si} = 0.01567$

1) l_{zp} = peak influence factor of soil layer i, Fig. 3-4 = $0.5+0.1(l_{ap}/\sigma'_{top})^{1/2} = 0.607$

where, γ' (pcf) = effective unit weig **0.034** Using $\gamma' = \gamma$ sat - γ water (If soil is below GWT)
0.06 Using $\gamma' = \gamma$ moist (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing -effective overburden pressure at depth l_{zp} , tsf
 $\sigma'_{top} = 0.5 \times B \times \gamma' + D \times \gamma'$ **0.600**

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

Soil Type	N (blows/ft) =	100	$L/B = 1$
s, sandy silts, slightly cohesive silt-sand mixtu			10N = 1000
Clean, fine to medium sands and slightly silty sands			17.5N = 1750
Coarse sands and sands with little gravel			25N = 2500
Sandy gravel and gravel			30N = 3000
			E _{si} (ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 \cdot C_2 \cdot \Delta p \cdot \Delta z_i \sum \frac{I_z I_{z_i}}{E_{si}}$$

Short Term

Long Term

$\frac{0.0237}{0.0331}$ ft

$\frac{0.28}{0.40}$ inches

<1

<1

OK

OK

where,

C₁ = correction to account for strain relief from embedment, 1-0.5σ'_{vd}/Δp>=0.5

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D, tsf

where,

γ_{moist} unit weight (pcf) = 120

C₂ = correction for time dependent increase in settlement, 1+0.2 log₁₀(t/0.1)

t = time, years (end of construction) 0.1

Δp = net applied footing pressure, q-σ'_{vd}, tsf

Δz_i = depth increment i, 0.1B, ft

I_{zi} = influence factor of soil layer i, fig. 3-4

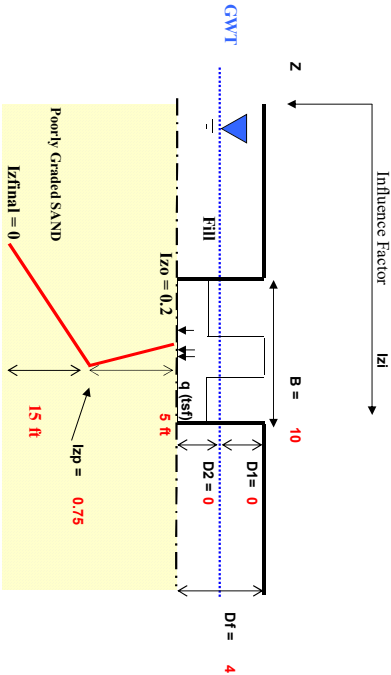
E_{si} = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

B = 10

Df = 4

GWT depth (ft) = 0



$l_{z0} = 0.10$
 $l_{zp} = 0.75$
 $l_{zp}-l_{z0} = 0.65$

$Z_{final(t)} = 20$
 $Z_{sp(t)} = 5$
 $Z_{final} - Z_{sp} = 15$

z, ft	z/B	l _{zi}	E _{si} , tsf	I _{za}	I _{za} /E _{si}
0.00	0.00	0.10			
1.00	0.10	0.23	610	0.165	0.0003
2.00	0.20	0.36	610	0.295	0.0005
3.00	0.30	0.49	610	0.424	0.0007
4.00	0.40	0.62	610	0.554	0.0009
5.00	0.50	0.75	610	0.684	0.0011
6.00	0.60	0.70	610	0.724	0.0012
7.00	0.70	0.65	610	0.674	0.0011
8.00	0.80	0.60	610	0.624	0.0010
9.00	0.90	0.55	610	0.574	0.0009
10.00	1.00	0.50	610	0.524	0.0009
11.00	1.10	0.45	610	0.474	0.0008
12.00	1.20	0.40	610	0.424	0.0007
13.00	1.30	0.35	610	0.374	0.0006
14.00	1.40	0.30	610	0.324	0.0005
15.00	1.50	0.25	610	0.274	0.0004
16.00	1.60	0.20	610	0.225	0.0004
17.00	1.70	0.15	610	0.175	0.0003
18.00	1.80	0.10	610	0.125	0.0002
19.00	1.90	0.05	610	0.075	0.0001
20.00	2.00	0.00	610	0.025	0.0000

Sand and Silt

$\sum I_{zi}/E_{si} = 0.01268$

1) l_{zp} = Peak influence factor of soil layer i, Fig. 3-4 = $0.5+0.11ab/\sigma'_{i_{zp}})^{1/2} = 0.749$

where, γ' (pcf) = effective unit weig **0.035** Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing
 $\sigma'_{i_{zp}} = 0.5 \times B \times \gamma' + D \times \gamma' = 0.313$ -effective overburden pressure at depth l_{zp} , tsf

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

Soil Type	N (blows/ft) = 61	$L/B = 1$
s, sandy silt, slightly cohesive silt-sand mixtu		10N = 610
Clean, fine to medium sands and slightly silty sands		17.5N = 1067.5
Coarse sands and sands with little gravel		25N = 1525
Sandy gravel and gravel		30N = 1830

E_{si} (ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis", 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 * C_2 * \Delta p * A_z \sum \frac{I_z}{E_{si}}$$

	Short Term		
	0.0240 ft	0.29	inches <1
Long Term	0.0336 ft	0.40	inches <1
			OK

where,

C₁ = correction to account for strain relief from embedment, 1-0.5 σ'_{vd}/Δp>=0.;

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D, ts

where,

$$\gamma_{ moist } \text{ unit weight (pcf)} = 120$$

$$\gamma_{ sat } \text{ unit weight (pcf)} = 132.00$$

C₂ = correction for time dependent increase in settlement, 1+0.2 log₁₀ (t / 0.1)

t = time, years (end of construct 0.1 10 (long term)

Δp = net applied footing pressure, q-σ'_{vd}, tsf

A_z_i = depth increment i, 0.1B, ft

I_zi = influence factor of soil layer i, fig. 3-4

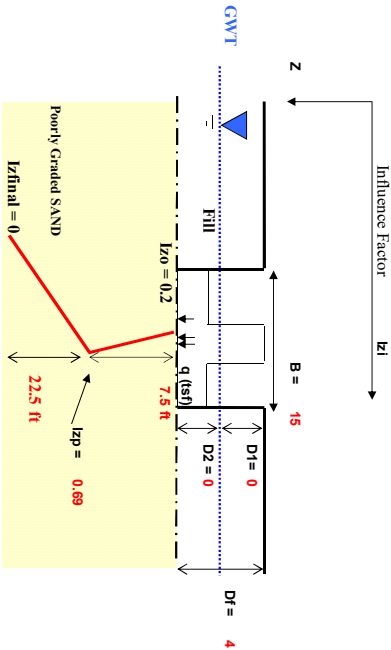
E_{si} = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

$$B = 15$$

$$Df = 4$$

$$\text{GWT depth (ft)} = 0$$



$I_{zo} = 0.10$
 $I_{zp} = 0.69$
 $I_{zp} - I_{zo} = 0.59$

$Z_{final(t)} = 30$
 $Z_{np(t)} = 7.5$
 $Z_{final} - Z_{np} = 22.5$

z, ft	z/B	I _{zi}	E _{si} , tsf	I _{za}	I _{za} /E _{si}
0.00	0.00	0.10			
1.50	0.10	0.22	610	0.159	0.0003
3.00	0.20	0.34	610	0.277	0.0005
4.50	0.30	0.45	610	0.395	0.0006
6.00	0.40	0.57	610	0.513	0.0008
7.50	0.50	0.69	610	0.630	0.0010
9.00	0.60	0.64	610	0.666	0.0011
10.50	0.70	0.60	610	0.620	0.0010
12.00	0.80	0.55	610	0.575	0.0009
13.50	0.90	0.51	610	0.529	0.0009
15.00	1.00	0.46	610	0.483	0.0008
16.50	1.10	0.41	610	0.437	0.0007
18.00	1.20	0.37	610	0.391	0.0006
19.50	1.30	0.32	610	0.345	0.0006
21.00	1.40	0.28	610	0.299	0.0005
22.50	1.50	0.23	610	0.253	0.0004
24.00	1.60	0.18	610	0.207	0.0003
25.50	1.70	0.14	610	0.161	0.0003
27.00	1.80	0.09	610	0.115	0.0002
28.50	1.90	0.05	610	0.069	0.0001
30.00	2.00	0.00	610	0.023	0.0000

Sand and Silt

$\sum I_{zi}/E_{si} = 0.01171$

1) I_{zp} = peak influence factor of soil layer i, **Fig. 3-4** = $0.5 + 0.11 \Delta np \sigma'_{np}{}^{1/2} = 0.689$

where, γ' (pcf) = effective unit wei; **0.035** Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing -effective overburden pressure at depth I_{zp} , tsf

$\sigma'_{np} = 0.5 \times B \times \gamma' + D \times \gamma'$ **0.400**

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf) . After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978).

Soil Type	N (blows/ft) = 61	L/B = 1
s ₁ sandy silts, slightly cohesive silt-sand mixt		10N = 610
Clean, fine to medium sands and slightly silty sands		17.5N = 1067.5
Coarse sands and sands with little gravel		25N = 1525
Sandy gravel and gravel		30N = 1830
		E _{si} (ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis", 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 \cdot C_2 \cdot \Delta p \cdot \Delta z_i \sum \frac{I_z}{E_{si}}$$

Short Term

Long Term

$\frac{0.0240}{0.0336}$ ft

$\frac{0.29}{0.40}$ inches

<1

<1

OK

OK

where,

C_1 = correction to account for strain relief from embedment, $1-0.5 \sigma'_{vd}/\Delta p \geq 0$;

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D , ts

where,

$$\gamma_{ moist}$$

unit weight (pcf)

120

132.00

C_2 = correction for time dependent increase in settlement, $1+0.2 \log_{10} (t / 0.1)$

t = time, years (end of construct

Δp = net applied footing pressure, q- σ'_{vd} , tsf

Δz_i = depth increment i , 0.1B, ft

I_z = influence factor of soil layer i , fig. 3-4

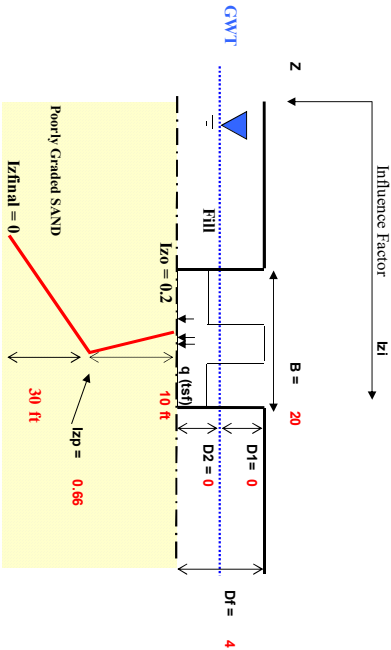
E_{si} = elastic modulus of soil layer i , tsf

q = applied pressure at foundation, tsf

B = 20

D_f = 4

GWT depth (ft) = 0



$I_{zo} = 0.10$
 $I_{zp} = 0.66$
 $I_{zp} - I_{zo} = 0.56$

$Z_{final(t)} = 40$
 $Z_{np(t)} = 10$
 $Z_{final} - Z_{np} = 30$

z, ft	z/B	I _{zi}	E _{si} , tsf	I _{za}	I _{za} /E _{si}
0.00	0.00	0.10			
2.00	0.10	0.21	610	0.156	0.0003
4.00	0.20	0.32	610	0.267	0.0004
6.00	0.30	0.43	610	0.378	0.0006
8.00	0.40	0.54	610	0.489	0.0008
10.00	0.50	0.66	610	0.600	0.0010
12.00	0.60	0.61	610	0.634	0.0010
14.00	0.70	0.57	610	0.590	0.0010
16.00	0.80	0.52	610	0.547	0.0009
18.00	0.90	0.48	610	0.503	0.0008
20.00	1.00	0.44	610	0.459	0.0008
22.00	1.10	0.39	610	0.415	0.0007
24.00	1.20	0.35	610	0.372	0.0006
26.00	1.30	0.31	610	0.328	0.0005
28.00	1.40	0.26	610	0.284	0.0005
30.00	1.50	0.22	610	0.241	0.0004
32.00	1.60	0.17	610	0.197	0.0003
34.00	1.70	0.13	2250	0.153	0.0001
36.00	1.80	0.09	2250	0.109	0.0000
38.00	1.90	0.04	2250	0.066	0.0000
40.00	2.00	0.00	2250	0.022	0.0000

Sand and Silt

$\sum I_{zi}/E_{si} = 0.01075$

1) I_{zp} = peak influence factor of soil layer i, **fig. 3-4 = 0.5+0.11Δnpσ_{avg}^{1/2} = 0.656**

where, γ' (pcf)= effective unit wei; **0.035** Using γ' = γ sat - γ water (If soil is below GWT)
0.06 Using γ' = γ moist (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing
 $\sigma'_{top} = 0.5 \times B \times \gamma' + D \times \gamma'$ **0.487** -effective overburden pressure at depth I_{zp}, tsf

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf). After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978).

Soil Type	N (blows/ft) = 100	L/B = 1
s ₁ sandy silts, slightly cohesive silt-sand mixt		10N = 1000
Clean, fine to medium sands and slightly silty sands		17.5N = 1750
Coarse sands and sands with little gravel		25N = 2500
Sandy gravel and gravel		30N = 3000
		E _{si} (ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.0244 ft	0.29 inches	<1	OK
Long Term	0.0342 ft	0.41 inches	<1	OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od}/\Delta p \geq 0.5$ **0.87**

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf **0.17 = $\gamma' D$**

where, γ_{moist} unit weight (pcf) **120**

γ_{sat} unit weight (pcf) = **131.00**

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ **1.0** (short-term) **1.4** (long-term)

t = time, years (end of construction) **0.1** **10** (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf **0.65**

Δz_i = depth increment i, 0.1B, ft **6.5**

I_{zi} = influence factor of soil layer i, fig. 3-4

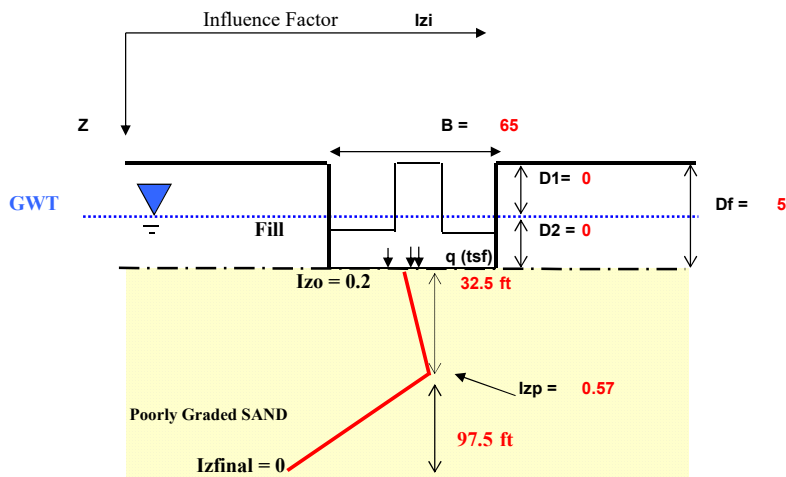
E_{si} = elastic modulus of soil layer i, tsf **481.25**

q = applied pressure at foundation, tsf **0.83** 1650

B = **65**

Df = **5**

GWT depth (ft) = **0**



$$I_{zo} = 0.10$$

$$I_{zp} = 0.57$$

$$I_{zp} - I_{zo} = 0.47$$

$$Z_{final} (ft) = 130$$

$$Z_{zp} (ft) = 32.5$$

$$Z_{final} - Z_{zp} = 97.5$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Sand and Silt Gravel Till Weather
6.50	0.10	0.19	350	0.147	0.0004	
13.00	0.20	0.29	350	0.241	0.0007	
19.50	0.30	0.38	350	0.336	0.0010	
26.00	0.40	0.48	350	0.430	0.0012	
32.50	0.50	0.57	350	0.524	0.0015	
39.00	0.60	0.53	2250	0.552	0.0002	
45.50	0.70	0.50	2250	0.514	0.0002	
52.00	0.80	0.46	2250	0.476	0.0002	
58.50	0.90	0.42	2250	0.438	0.0002	
65.00	1.00	0.38	2250	0.400	0.0002	Bedrock
71.50	1.10	0.34	2500	0.362	0.0001	
78.00	1.20	0.30	2500	0.324	0.0001	
84.50	1.30	0.27	2500	0.286	0.0001	
91.00	1.40	0.23	2500	0.248	0.0001	
97.50	1.50	0.19	2500	0.209	0.0001	
104.00	1.60	0.15	2500	0.171	0.0001	
110.50	1.70	0.11	2500	0.133	0.0001	
117.00	1.80	0.08	2500	0.095	0.0000	
123.50	1.90	0.04	2500	0.057	0.0000	
130.00	2.00	0.00	2500	0.019	0.0000	

$$\sum Izi/E_{si} = 0.00661$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1[\Delta p/\sigma'_{i_{zp}}]^{1/2} = 0.571$

where, γ' (pcf) = effective unit weight 0.034 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
 0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{i_{zp}} = 0.5 \times B \times \gamma' + D \times \gamma' = 1.286$

2) Elastic Modulus, Esi (ksf)

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N (\text{blows/ft}) = 100$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt
 Clean, fine to medium sands and slightly silty sands
 Coarse sands and sands with little gravel
 Sandy gravel and gravel

L/B = 1

10N = 1000
 17.5N = 1750
 25N = 2500
 30N = 3000

$$Esi (Ksf) = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.0239 ft	0.29 inches	<1	OK
Long Term	0.0334 ft	0.40 inches	<1	OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od} / \Delta p \geq 0.5$ **0.91**

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf **0.17 = $\gamma' D$**

where, γ_{moist} unit weight (pcf) **120**

γ_{sat} unit weight (pcf) = **132.00**

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ **1.0** (short-term) **1.4** (long-term)

t = time, years (end of construction) **0.1** **10** (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf **0.95**

Δz_i = depth increment i, 0.1B, ft **6.0**

I_{zi} = influence factor of soil layer i, fig. 3-4

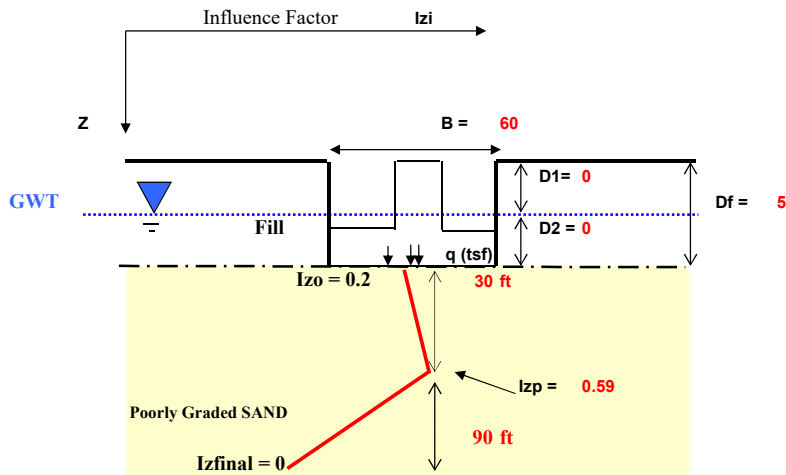
E_{si} = elastic modulus of soil layer i, tsf **481.25**

q = applied pressure at foundation, tsf **1.13** 2250 psf

B = **60**

Df = **5**

GWT depth (ft) = **0**



$$I_{zo} = 0.10$$

$$I_{zp} = 0.59$$

$$I_{zp} - I_{zo} = 0.49$$

$$Z_{final} (ft) = 120$$

$$Z_{zp} (ft) = 30$$

$$Z_{final} - Z_{zp} = 90$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Sand and Silt
6.00	0.10	0.20	610	0.149	0.0002	
12.00	0.20	0.30	610	0.247	0.0004	
18.00	0.30	0.39	610	0.344	0.0006	
24.00	0.40	0.49	610	0.442	0.0007	
30.00	0.50	0.59	610	0.540	0.0009	Glacial Till
36.00	0.60	0.55	2250	0.569	0.0003	
42.00	0.70	0.51	2250	0.530	0.0002	
48.00	0.80	0.47	2250	0.490	0.0002	
54.00	0.90	0.43	2250	0.451	0.0002	
60.00	1.00	0.39	2250	0.412	0.0002	Weathered
66.00	1.10	0.35	2250	0.373	0.0002	
72.00	1.20	0.31	3000	0.333	0.0001	
78.00	1.30	0.27	3000	0.294	0.0001	
84.00	1.40	0.24	3000	0.255	0.0001	
90.00	1.50	0.20	3000	0.216	0.0001	Bedrock
96.00	1.60	0.16	3000	0.177	0.0001	
102.00	1.70	0.12	3000	0.137	0.0000	
108.00	1.80	0.08	3000	0.098	0.0000	
114.00	1.90	0.04	3000	0.059	0.0000	
120.00	2.00	0.00	2500	0.020	0.0000	

$$\sum Izi/E_{si} = 0.00461$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1 |\Delta p / \sigma'_{i,zp}|^{1/2} = 0.588$

where, γ' (pcf) = effective unit weight 0.035 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
 0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{i,zp} = 0.5 \times B \times \gamma' + D \times \gamma' = 1.218$

2) **Elastic Modulus, Esi (ksf)**

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N \text{ (blows/ft)} = 100$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt
 Clean, fine to medium sands and slightly silty sands
 Coarse sands and sands with little gravel
 Sandy gravel and gravel

L/B = 1

10N = 1000
 17.5N = 1750
 25N = 2500
 30N = 3000

$$Esi \text{ (Ksf)} = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.0358 ft	0.43 inches	<1	OK
Long Term	0.0502 ft	0.60 inches	<1	OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od}/\Delta p \geq 0.5$

0.96

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf

0.17 = $\gamma' D$

where, γ_{moist} unit weight (pcf) = 120

γ_{sat} unit weight (pcf) = 132.00

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$

1.0 (short-term)

1.4 (long-term)

t = time, years (end of construction) = 0.1

10 (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf

2.05

Δz_i = depth increment i, 0.1B, ft

1.5

I_{zi} = influence factor of soil layer i, fig. 3-4

E_{si} = elastic modulus of soil layer i, tsf

481.25

q = applied pressure at foundation, tsf

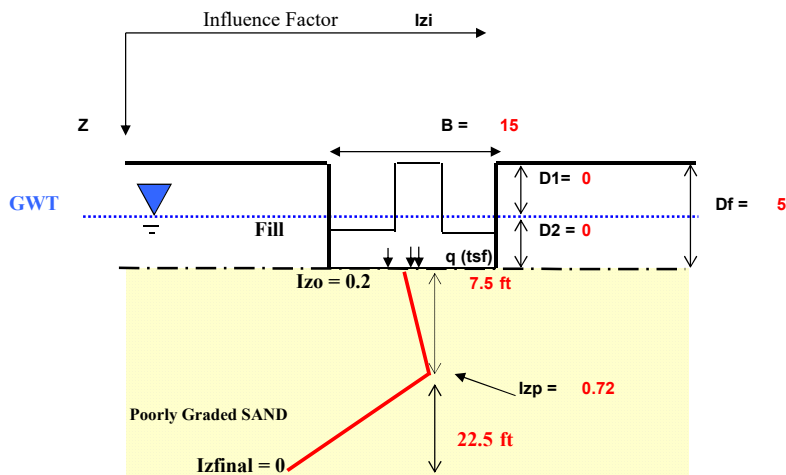
2.23

4450

B = 15

Df = 5

GWT depth (ft) = 0



$$I_{zo} = 0.10$$

$$I_{zp} = 0.72$$

$$I_{zp} - I_{zo} = 0.62$$

$$Z_{final} (ft) = 30$$

$$Z_{zp} (ft) = 7.5$$

$$Z_{final} - Z_{zp} = 22.5$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	Sand and Silt
0.00	0.00	0.10				
1.50	0.10	0.22	610	0.162	0.0003	
3.00	0.20	0.35	610	0.285	0.0005	
4.50	0.30	0.47	610	0.409	0.0007	
6.00	0.40	0.59	610	0.532	0.0009	
7.50	0.50	0.72	610	0.655	0.0011	
9.00	0.60	0.67	610	0.693	0.0011	
10.50	0.70	0.62	610	0.645	0.0011	
12.00	0.80	0.57	610	0.598	0.0010	
13.50	0.90	0.53	610	0.550	0.0009	
15.00	1.00	0.48	610	0.502	0.0008	
16.50	1.10	0.43	610	0.454	0.0007	
18.00	1.20	0.38	610	0.406	0.0007	
19.50	1.30	0.33	610	0.359	0.0006	
21.00	1.40	0.29	610	0.311	0.0005	
22.50	1.50	0.24	610	0.263	0.0004	
24.00	1.60	0.19	610	0.215	0.0004	
25.50	1.70	0.14	610	0.167	0.0003	
27.00	1.80	0.10	610	0.120	0.0002	
28.50	1.90	0.05	610	0.072	0.0001	
30.00	2.00	0.00	610	0.024	0.0000	

$$\sum Izi/E_{si} = 0.01217$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1[\Delta p/\sigma'_{i_{zp}}]^{1/2} = 0.717$

where, γ' (pcf) = effective unit weight 0.035
 0.06 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{i_{zp}} = 0.5 \times B \times \gamma' + D \times \gamma' = 0.435$

2) Elastic Modulus, Esi (ksf)

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N (\text{blows/ft}) = 61$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt
Clean, fine to medium sands and slightly silty sands
Coarse sands and sands with little gravel
Sandy gravel and gravel

L/B = 1

10N = 610
17.5N = 1067.5
25N = 1525
30N = 1830

$$Esi (Ksf) = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 \cdot C_2 \cdot \Delta p \cdot \Delta z_i \sum \frac{I_{zi}}{E_{si}}$$

Short Term

Long Term

0.0359 ft

0.0503 ft

0.43 inches

0.60 inches

<1

<1

OK

OK

where,

C₁ = correction to account for strain relief from embedment, 1-0.5σ'_{vd}/Δp>=0.5

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D, tsf

where,

γ_{moist} unit weight (pcf) = 120

C₂ = correction for time dependent increase in settlement, 1+0.2 log₁₀(t/0.1)

t = time, years (end of construction)

Δp = net applied footing pressure, q-σ'_{vd}, tsf

Δz_i = depth increment i, 0.1B, ft

I_{zi} = influence factor of soil layer i, fig. 3-4

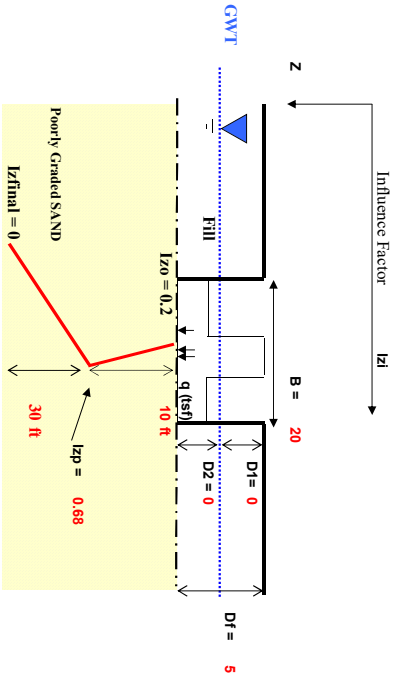
E_{si} = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

B = 20

Df = 5

GWT depth (ft) = 0



$$I_{zp} = 0.10$$
$$I_{zp} = 0.68$$
$$I_{zp} - I_{zo} = 0.58$$
$$Z_{final (ft)} = 40$$
$$Z_{sp (ft)} = 10$$
$$Z_{final} - Z_{sp} = 30$$

z, ft	z/B	I _{zi}	E _{si} , tsf	I _{za}	I _{za} /E _{si}
0.00	0.00	0.10			
2.00	0.10	0.22	610	0.158	0.0003
4.00	0.20	0.33	610	0.274	0.0004
6.00	0.30	0.45	610	0.390	0.0006
8.00	0.40	0.56	610	0.506	0.0008
10.00	0.50	0.68	610	0.622	0.0010
12.00	0.60	0.64	610	0.658	0.0011
14.00	0.70	0.59	610	0.612	0.0010
16.00	0.80	0.54	610	0.567	0.0009
18.00	0.90	0.50	610	0.522	0.0009
20.00	1.00	0.45	610	0.476	0.0008
22.00	1.10	0.41	610	0.431	0.0007
24.00	1.20	0.36	610	0.386	0.0006
26.00	1.30	0.32	610	0.340	0.0006
28.00	1.40	0.27	610	0.295	0.0005
30.00	1.50	0.23	610	0.250	0.0004
32.00	1.60	0.18	610	0.204	0.0003
34.00	1.70	0.14	2250	0.159	0.0001
36.00	1.80	0.09	2250	0.113	0.0001
38.00	1.90	0.05	2250	0.068	0.0000
40.00	2.00	0.00	2250	0.023	0.0000

Sand and Silt
Till
Glacial

$$I_{zp} = \text{peak influence factor of soil layer i, Fig. 3-4} = 0.5 + 0.1 I_{ap} / \sigma'_{top}{}^{1/2} = 0.681$$
$$\sum I_{zi} / E_{si} = 0.01113$$

where, γ' (pcf) = effective unit weight **0.035**
0.06
Using $\gamma' = \gamma \text{ sat} - \gamma \text{ water}$ (If soil is below GWT)
Using $\gamma' = \gamma \text{ moist}$ (If soil is above GWT)
Axisymmetric (L/B=1) - Spread/column footing
 $\sigma'_{top} = 0.5 \gamma B \times \gamma' + D \times \gamma' = 0.522$
-effective overburden pressure at depth I_{zp}, tsf

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)
$$N (\text{blows/ft}) = 100$$
$$L/B = 1$$

Soil Type	10N =	1000
s, sandy silts, slightly cohesive silt-sand mixtu	17.5N =	1750
Clean, fine to medium sands and slightly silty sands	25N =	2500
Coarse sands and sands with little gravel	30N =	3000
Sandy gravel and gravel		

$$E_{si} \text{ (ksf)} = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis", 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 \cdot C_2 \cdot \Delta p \cdot \Delta z_i \sum I_z / E_{si}$$

Short Term

Long Term

$\frac{0.0357}{0.0499}$ ft

$\frac{0.43}{0.60}$ inches

<1

<1

OK

OK

where,

C_1 = correction to account for strain relief from embedment, $1-0.5 \sigma'_{vd} / \Delta p > 0$;

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D_1 , ts

where,

$$\gamma_{ moist}$$

unit weight (pcf)

120

132.00

C_2 = correction for time dependent increase in settlement, $1+0.2 \log_{10} (t / 0.1)$

t = time, years (end of construct

Δp = net applied footing pressure, $q - \sigma'_{vd}$, tsf

Δz_i = depth increment i , 0.1B, ft

I_z = influence factor of soil layer i , fig. 3-4

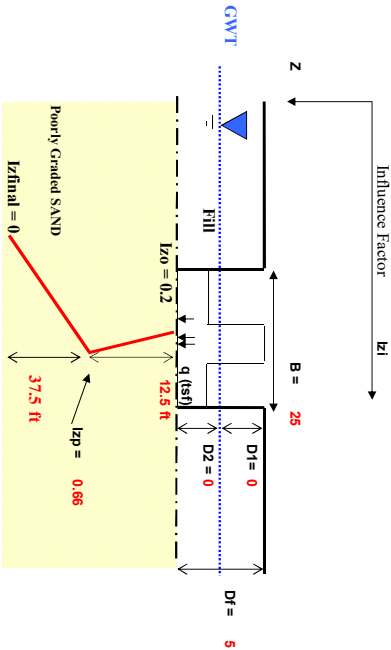
E_{si} = elastic modulus of soil layer i , tsf

q = applied pressure at foundation, tsf

$B = 25$

$D_f = 5$

GWT depth (ft) = 0



$I_{zo} = 0.10$
 $I_{zp} = 0.66$
 $I_{zp} - I_{zo} = 0.56$

$Z_{final(t)} = 50$
 $Z_{np(t)} = 12.5$
 $Z_{final} - Z_{np} = 37.5$

z, ft	z/B	I _{zi}	E _{si} , tsf	I _{za}	I _{za} /E _{si}
0.00	0.00	0.10			
2.50	0.10	0.21	610	0.156	0.0003
5.00	0.20	0.32	610	0.267	0.0004
7.50	0.30	0.43	610	0.379	0.0006
10.00	0.40	0.55	610	0.491	0.0008
12.50	0.50	0.66	610	0.602	0.0010
15.00	0.60	0.61	610	0.636	0.0010
17.50	0.70	0.57	610	0.592	0.0010
20.00	0.80	0.53	610	0.549	0.0009
22.50	0.90	0.48	610	0.505	0.0008
25.00	1.00	0.44	610	0.461	0.0008
27.50	1.10	0.39	610	0.417	0.0007
30.00	1.20	0.35	610	0.373	0.0006
32.50	1.30	0.31	610	0.329	0.0005
35.00	1.40	0.26	2250	0.285	0.0001
37.50	1.50	0.22	2250	0.241	0.0001
40.00	1.60	0.18	2250	0.197	0.0001
42.50	1.70	0.13	2250	0.154	0.0001
45.00	1.80	0.09	2250	0.110	0.0000
47.50	1.90	0.04	2250	0.066	0.0000
50.00	2.00	0.00	2250	0.022	0.0000

Sand and Silt
Glacial Till

$\sum I_{zi}/E_{si} = 0.00992$

1) I_{zp} = peak influence factor of soil layer i, **Fig. 3-4 = 0.5+0.11Δnp/σ_{avg}^{1/2} = 0.658**

where, γ' (pcf) = effective unit wei; **0.035** Using γ' = γ sat - γ water (If soil is below GWT)
0.06 Using γ' = γ moist (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing -effective overburden pressure at depth I_{zp}, tsf

$\sigma'_{avg} = 0.5 \times B \times \gamma' + D \times \gamma'$ **0.609**

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf). After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978).

Soil Type	N (blows/ft) = 100	L/B = 1
s, sandy silts, slightly cohesive silt-sand mixt		10N = 1000
Clean, fine to medium sands and slightly silty sands		17.5N = 1750
Coarse sands and sands with little gravel		25N = 2500
Sandy gravel and gravel		30N = 3000
		E _{si} (ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.0357 ft	0.43 inches	<1	OK
Long Term	0.0499 ft	0.60 inches	<1	OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od}/\Delta p \geq 0.5$ **0.93**

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf **0.17 = $\gamma' D$**

where, γ_{moist} unit weight (pcf) **120**

γ_{sat} unit weight (pcf) = **131.00**

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ **1.0** (short-term) **1.4** (long-term)

t = time, years (end of construction) **0.1** **10** (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf **1.28**

Δz_i = depth increment i, 0.1B, ft **1.5**

I_{zi} = influence factor of soil layer i, fig. 3-4

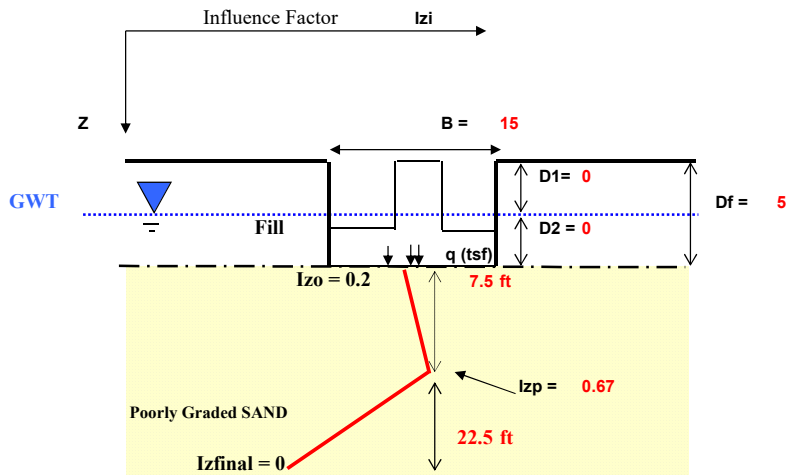
E_{si} = elastic modulus of soil layer i, tsf **481.25**

q = applied pressure at foundation, tsf **1.45** 2900

B = **15**

Df = **5**

GWT depth (ft) = **0**



$$\begin{aligned} I_{zo} &= 0.10 \\ I_{zp} &= 0.67 \\ I_{zp} - I_{zo} &= 0.57 \end{aligned}$$

$$\begin{aligned} Z_{\text{final}} (\text{ft}) &= 30 \\ Z_{zp} (\text{ft}) &= 7.5 \\ Z_{\text{final}} - Z_{zp} &= 22.5 \end{aligned}$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Sand and Silt
1.50	0.10	0.21	350	0.157	0.0004	
3.00	0.20	0.33	350	0.272	0.0008	
4.50	0.30	0.44	350	0.386	0.0011	
6.00	0.40	0.56	350	0.501	0.0014	
7.50	0.50	0.67	350	0.615	0.0018	
9.00	0.60	0.63	350	0.650	0.0019	
10.50	0.70	0.58	350	0.605	0.0017	
12.00	0.80	0.54	350	0.561	0.0016	
13.50	0.90	0.49	350	0.516	0.0015	
15.00	1.00	0.45	350	0.471	0.0013	
16.50	1.10	0.40	350	0.426	0.0012	
18.00	1.20	0.36	350	0.381	0.0011	
19.50	1.30	0.31	350	0.336	0.0010	
21.00	1.40	0.27	350	0.291	0.0008	
22.50	1.50	0.22	350	0.247	0.0007	Clacial Till
24.00	1.60	0.18	350	0.202	0.0006	
25.50	1.70	0.13	350	0.157	0.0004	
27.00	1.80	0.09	350	0.112	0.0003	
28.50	1.90	0.04	350	0.067	0.0002	
30.00	2.00	0.00	350	0.022	0.0001	

$$\sum Izi/E_{si} = 0.01993$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1[\Delta p/\sigma'_{i_{zp}}]^{1/2} = 0.673$

where, γ' (pcf) = effective unit weight 0.034 Using $\gamma' = \gamma_{\text{sat}} - \gamma_{\text{water}}$ (If soil is below GWT)
0.06 Using $\gamma' = \gamma_{\text{moist}}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing -effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{i_{zp}} = 0.5 \times B \times \gamma' + D \times \gamma' \quad 0.429$

2) Elastic Modulus, Esi (ksf)

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N (\text{blows/ft}) = 61$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt
 Clean, fine to medium sands and slightly silty sands
 Coarse sands and sands with little gravel
 Sandy gravel and gravel

L/B = 1

$$\begin{aligned} 10N &= 610 \\ 17.5N &= 1067.5 \\ 25N &= 1525 \\ 30N &= 1830 \end{aligned}$$

$$E_{si} (\text{Ksf}) = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.0358 ft	0.43 inches	<1	OK
Long Term	0.0502 ft	0.60 inches	<1	OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od} / \Delta p \geq 0.5$ **0.92**

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf **0.17 = $\gamma' D$**

where, γ_{moist} unit weight (pcf) **120**

γ_{sat} unit weight (pcf) = **131.00**

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ **1.0** (short-term) **1.4** (long-term)

t = time, years (end of construction) **0.1** **10** (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf **1.07**

Δz_i = depth increment i, 0.1B, ft **2.0**

I_{zi} = influence factor of soil layer i, fig. 3-4

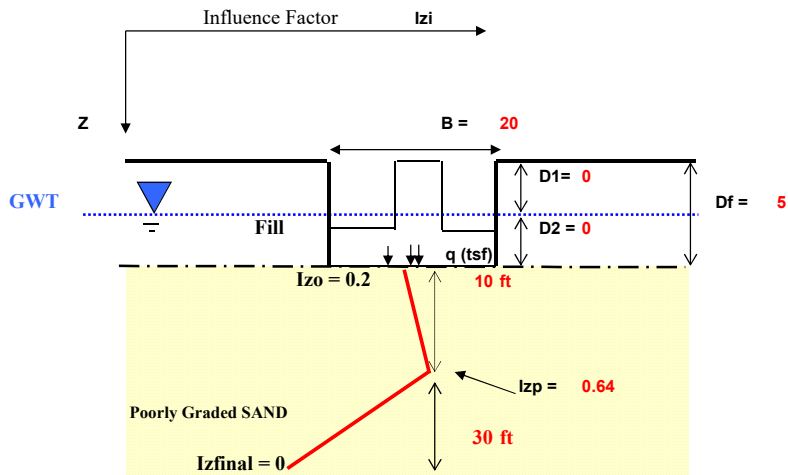
E_{si} = elastic modulus of soil layer i, tsf **481.25**

q = applied pressure at foundation, tsf **1.24** 2475

B = **20**

Df = **5**

GWT depth (ft) = **0**



$$I_{zo} = 0.10$$

$$I_{zp} = 0.64$$

$$I_{zp} - I_{zo} = 0.54$$

$$Z_{final} (ft) = 40$$

$$Z_{zp} (ft) = 10$$

$$Z_{final} - Z_{zp} = 30$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Sand and Silt
2.00	0.10	0.21	350	0.154	0.0004	
4.00	0.20	0.32	350	0.263	0.0008	
6.00	0.30	0.43	350	0.372	0.0011	
8.00	0.40	0.54	350	0.481	0.0014	
10.00	0.50	0.64	350	0.590	0.0017	
12.00	0.60	0.60	350	0.622	0.0018	
14.00	0.70	0.56	350	0.580	0.0017	
16.00	0.80	0.52	350	0.537	0.0015	
18.00	0.90	0.47	350	0.494	0.0014	
20.00	1.00	0.43	350	0.451	0.0013	
22.00	1.10	0.39	350	0.408	0.0012	
24.00	1.20	0.34	350	0.365	0.0010	
26.00	1.30	0.30	350	0.322	0.0009	
28.00	1.40	0.26	350	0.279	0.0008	
30.00	1.50	0.21	350	0.236	0.0007	Clacial Till
32.00	1.60	0.17	350	0.193	0.0006	
34.00	1.70	0.13	2250	0.150	0.0001	
36.00	1.80	0.09	2250	0.107	0.0000	
38.00	1.90	0.04	2250	0.064	0.0000	
40.00	2.00	0.00	2250	0.021	0.0000	

$$\sum Izi/E_{si} = 0.01828$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1[\Delta p/\sigma'_{i_{zp}}]^{1/2} = 0.644$

where, γ' (pcf) = effective unit weight 0.034
 0.06 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{i_{zp}} = 0.5 \times B \times \gamma' + D \times \gamma' = 0.515$

2) Elastic Modulus, Esi (ksf)

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N (\text{blows/ft}) = 61$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt
Clean, fine to medium sands and slightly silty sands
Coarse sands and sands with little gravel
Sandy gravel and gravel

L/B = 1

10N = 610
17.5N = 1067.5
25N = 1525
30N = 1830

$$Esi (Ksf) = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 \cdot C_2 \cdot \Delta p \cdot \Delta z_i \sum \frac{I_{zi}}{E_{si}}$$

Short Term

Long Term

0.0355 ft

0.04498 ft

inches <1

inches <1

OK

OK

where,

C₁ = correction to account for strain relief from embedment, 1-0.5σ'_{vd}/Δp>=0.5

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D, tsf

where,

γ_{sat} unit weight (pcf) = 131.00

C₂ = correction for time dependent increase in settlement, 1+0.2 log₁₀(t/0.1)

t = time, years (end of construction) 0.1

Δp = net applied footing pressure, q-σ'_{vd}, tsf

Δz_i = depth increment i, 0.1B, ft

I_{zi} = influence factor of soil layer i, fig. 3-4

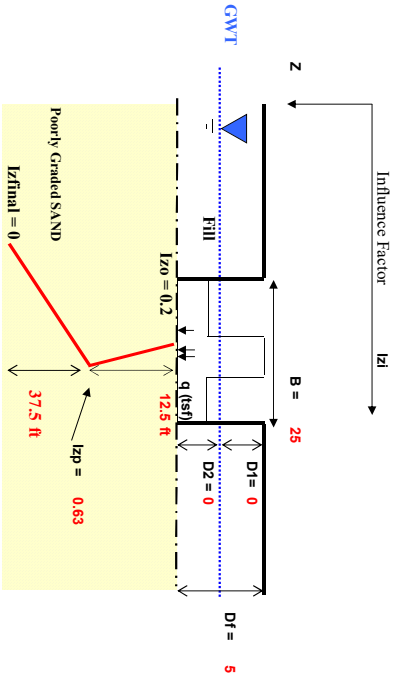
E_{si} = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

B = 25

Df = 5

GWT depth (ft) = 0



$l_{z0} = 0.10$
 $l_{zp} = 0.63$
 $l_{zp}-l_{z0} = 0.53$

$Z_{final}(ft) = 50$
 $Z_{sp}(ft) = 12.5$
 $Z_{final} - Z_{sp} = 37.5$

z, ft	z/B	l _{zi}	E _{si} , tsf	I _{za}	I _{zav} E _{si}
0.00	0.00	0.10			
2.50	0.10	0.21	350	0.153	0.0004
5.00	0.20	0.31	350	0.258	0.0007
7.50	0.30	0.42	350	0.363	0.0010
10.00	0.40	0.52	350	0.469	0.0013
12.50	0.50	0.63	350	0.574	0.0016
15.00	0.60	0.69	350	0.606	0.0017
17.50	0.70	0.54	350	0.564	0.0016
20.00	0.80	0.50	350	0.522	0.0015
22.50	0.90	0.46	350	0.481	0.0014
25.00	1.00	0.42	350	0.439	0.0013
27.50	1.10	0.38	350	0.397	0.0011
30.00	1.20	0.33	350	0.355	0.0010
32.50	1.30	0.29	350	0.313	0.0009
35.00	1.40	0.25	2250	0.272	0.0001
37.50	1.50	0.21	2250	0.230	0.0001
40.00	1.60	0.17	2250	0.188	0.0001
42.50	1.70	0.13	2250	0.146	0.0001
45.00	1.80	0.08	2250	0.104	0.0000
47.50	1.90	0.04	2250	0.063	0.0000
50.00	2.00	0.00	2250	0.021	0.0000

Sand and Silt
Glacial Till

$\sum I_{zi}/E_{si} = 0.01615$

1) l_{zp} = Peak influence factor of soil layer i, Fig. 3-4 = $0.5+0.1(l_{ap}/\sigma'_{i_{top}})^{1/2} = 0.627$

where, γ' (pcf) = effective unit weig **0.034** Using $\gamma' = \gamma$ sat - γ water (If soil is below GWT)
0.06 Using $\gamma' = \gamma$ moist (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing -effective overburden pressure at depth l_{zp} , tsf
 $\sigma'_{i_{top}} = 0.5 \times B \times \gamma' + D \times \gamma'$ **0.600**

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

Soil Type	N (blows/ft) = 100	$L/B = 1$
s, sandy silt, slightly cohesive silt-sand mixtu		10N = 1000
Clean, fine to medium sands and slightly silty sands		17.5N = 1750
Coarse sands and sands with little gravel		25N = 2500
Sandy gravel and gravel		30N = 3000
		E _{si} (ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footings

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$\rho_1 = C_1 * C_t * \Delta p^* \Delta z_i \sum z_i/E_{si} $	Short Term	<u>0.0359</u> ft	<u>0.43</u> inches < 1	OK
	Long Term	<u>0.0502</u> ft	<u>0.60</u> inches < 1	OK

where,

$$C_1 = \text{correction to account for strain relief from embedment, } 1-0.5\sigma_{\text{od}}/\Delta p \geq 0.5$$

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D, tsf

where, γ_{moist} unit weight (pcf

γ_{sat} unit weight (pcf) = **132.00**

$$C_t = \text{correction for time dependent increase in settlement, } 1 + 0.2 \log_{10}(t / 0.1)$$

t = time, years (end of constructic

 $\Delta p = \text{net applied footing pressure, } q - \sigma'_{od}, \text{ tsi}$ ΔZ_i = depth increment i , 0.1 B, ft

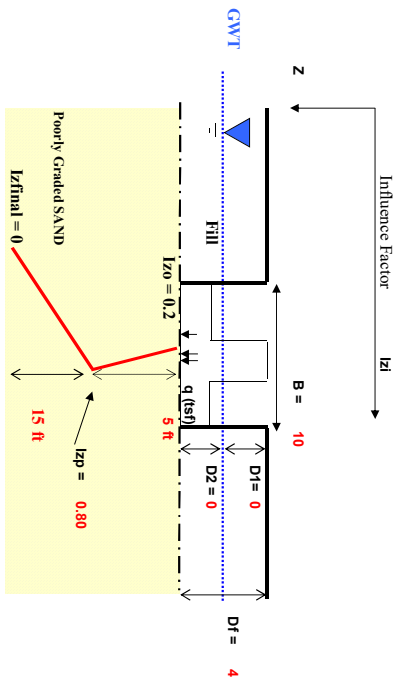
E_i = elastic modulus of soil layer i , tsf

q = applied pressure at foundation, tsf

$B = 10$

Df=4

GWI depth (ft) = 0



$$I_{zo} = 0.10$$
$$I_{zp} = 0.80$$
$$I_{zp}-I_{zo} = 0.70$$
$$Z_{final}(ft) = 20$$
$$Z_{sp}(ft) = 5$$
$$Z_{final} - Z_{sp} = 15$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi
0.00	0.00	0.10			
1.00	0.10	0.24	610	0.170	0.0003
2.00	0.20	0.38	610	0.309	0.0005
3.00	0.30	0.52	610	0.448	0.0007
4.00	0.40	0.66	610	0.587	0.0010
5.00	0.50	0.80	610	0.726	0.0012
6.00	0.60	0.74	610	0.769	0.0013
7.00	0.70	0.69	610	0.716	0.0012
8.00	0.80	0.64	610	0.663	0.0011
9.00	0.90	0.58	610	0.610	0.0010
10.00	1.00	0.53	610	0.557	0.0009
11.00	1.10	0.48	610	0.504	0.0008
12.00	1.20	0.42	610	0.451	0.0007
13.00	1.30	0.37	610	0.398	0.0007
14.00	1.40	0.32	610	0.345	0.0006
15.00	1.50	0.27	610	0.292	0.0005
16.00	1.60	0.21	610	0.239	0.0004
17.00	1.70	0.16	610	0.186	0.0003
18.00	1.80	0.11	610	0.133	0.0002
19.00	1.90	0.05	610	0.080	0.0001
20.00	2.00	0.00	610	0.027	0.0000

Sand and Silt

$$\sum Izi/E_{si} = 0.01345$$

1)
$$I_{zp} = \text{Peak influence factor of soil layer i, Fig. 3-4} = 0.5 + 0.11ab/\sigma'_{top}{}^{1/2} = 0.796$$

where, γ' (pcf) = effective unit weig **0.035** Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing -effective overburden pressure at depth I_{zp} , tsf
$$\sigma'_{top} = 0.5 \times B \times \gamma' + D \times \gamma' \quad \text{0.313}$$

2) Elastic Modulus, Esi (ksf)

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

<u>Soil Type</u>	<u>N (blows/ft) =</u>	<u>61</u>	<u>L/B = 1</u>
s, sandy silt, slightly cohesive silt-sand mixtu			10N = 610
Clean, fine to medium sands and slightly silty sands			17.5N = 1067.5
Coarse sands and sands with little gravel			25N = 1525
Sandy gravel and gravel			30N = 1830

$$Esi \text{ (ksf)} = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis", 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 \cdot C_2 \cdot \Delta p \cdot \Delta z_i \sum \frac{I_z}{E_{si}}$$

Short Term

Long Term

$\frac{0.0358}{0.0501}$ ft

$\frac{0.43}{0.60}$ inches

<1

<1

OK

OK

where,

C_1 = correction to account for strain relief from embedment, $1-0.5 \sigma'_{vd}/\Delta p \geq 0$;

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D , ts

where,

$$\gamma_{ moist}$$

unit weight (pcf)

120

132.00

C_2 = correction for time dependent increase in settlement, $1+0.2 \log_{10} (t / 0.1)$

t = time, years (end of construct)

Δp = net applied footing pressure, $q - \sigma'_{vd}$, tsf

Δz_i = depth increment i , 0.1B, ft

I_z = influence factor of soil layer i , fig. 3-4

E_{si} = elastic modulus of soil layer i , tsf

q = applied pressure at foundation, tsf

$$B$$

15

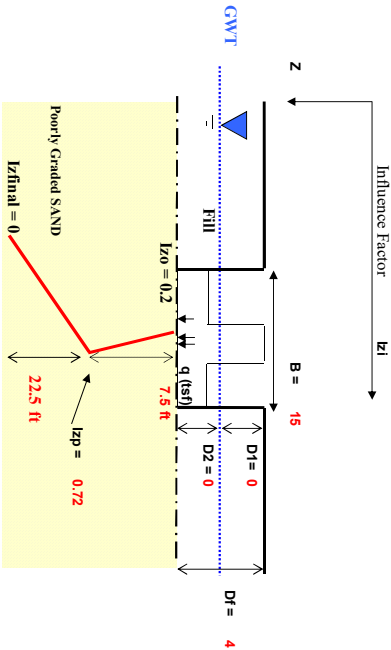
15

$$D_f$$

4

4

GWT depth (ft) = 0



$I_{zo} = 0.10$
 $I_{zp} = 0.72$
 $I_{zp}-I_{zo} = 0.62$

$Z_{final(t)} = 30$
 $Z_{np(t)} = 7.5$
 $Z_{final} - Z_{np} = 22.5$

z, ft	z/B	I _{zi}	E _{si} , tsf	I _{za}	I _{za} /E _{si}
0.00	0.00	0.10			
1.50	0.10	0.22	610	0.162	0.0003
3.00	0.20	0.35	610	0.287	0.0005
4.50	0.30	0.47	610	0.412	0.0007
6.00	0.40	0.60	610	0.537	0.0009
7.50	0.50	0.72	610	0.662	0.0011
9.00	0.60	0.68	610	0.700	0.0011
10.50	0.70	0.63	610	0.652	0.0011
12.00	0.80	0.58	610	0.603	0.0010
13.50	0.90	0.53	610	0.555	0.0009
15.00	1.00	0.48	610	0.507	0.0008
16.50	1.10	0.43	610	0.459	0.0008
18.00	1.20	0.39	610	0.410	0.0007
19.50	1.30	0.34	610	0.362	0.0006
21.00	1.40	0.29	610	0.314	0.0005
22.50	1.50	0.24	610	0.266	0.0004
24.00	1.60	0.19	610	0.217	0.0004
25.50	1.70	0.14	610	0.169	0.0003
27.00	1.80	0.10	610	0.121	0.0002
28.50	1.90	0.05	610	0.072	0.0001
30.00	2.00	0.00	610	0.024	0.0000

Sand and Silt

$\sum I_{zi}/E_{si} = 0.01228$

1) I_{zp} = peak influence factor of soil layer i, **fig. 3-4 = 0.5+0.11Δnp/σ_{avg}^{1/2} = 0.724**

where, γ' (pcf)= effective unit wei; **0.035** Using γ' = γ sat - γ water (If soil is below GWT)
0.06 Using γ' = γ moist (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing -effective overburden pressure at depth I_{zp}, tsf

$\sigma'_{avg} = 0.5 \times B \times \gamma' + D \times \gamma'$ **0.400**

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf) . After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978).

Soil Type	N (blows/ft) = 61	L/B = 1
s ₁ sandy silts, slightly cohesive silt-sand mixt		10N = 610
Clean, fine to medium sands and slightly silty sands		17.5N = 1067.5
Coarse sands and sands with little gravel		25N = 1525
Sandy gravel and gravel		30N = 1830
		E _{si} (ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis", 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 \cdot C_2 \cdot \Delta p \cdot \Delta z_i \sum \frac{I_z}{E_{si}}$$

Short Term

Long Term

$\frac{0.0359}{0.0503}$ ft

$\frac{0.43}{0.60}$ inches

<1

<1

OK

OK

where,

C_1 = correction to account for strain relief from embedment, $1-0.5 \sigma'_{vd}/\Delta p \geq 0$;

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D , ts

where,

$$\gamma_{ moist}$$

unit weight (pcf)

120

132.00

C_2 = correction for time dependent increase in settlement, $1+0.2 \log_{10} (t / 0.1)$

t = time, years (end of construct)

Δp = net applied footing pressure, $q - \sigma'_{vd}$, tsf

Δz_i = depth increment i , 0.1B, ft

I_z = influence factor of soil layer i , fig. 3-4

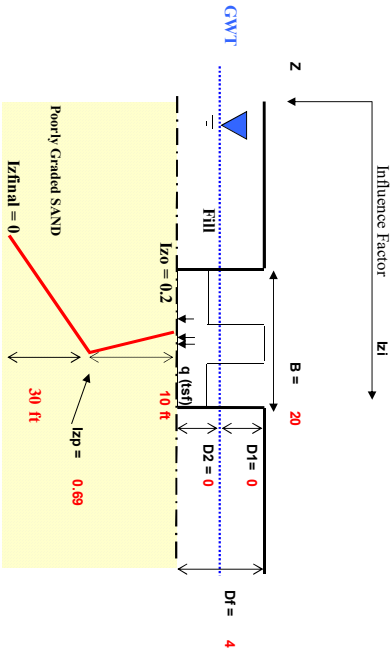
E_{si} = elastic modulus of soil layer i , tsf

q = applied pressure at foundation, tsf

B = 20

D_f = 4

GWT depth (ft) = 0



$I_{zo} = 0.10$
 $I_{zp} = 0.69$
 $I_{zp}-I_{zo} = 0.59$

$Z_{final(t)} = 40$
 $Z_{np(t)} = 10$
 $Z_{final} - Z_{np} = 30$

z, ft	z/B	I _{zi}	E _{si} , tsf	I _{za}	I _{za} /E _{si}
0.00	0.00	0.10			
2.00	0.10	0.22	610	0.159	0.0003
4.00	0.20	0.33	610	0.276	0.0005
6.00	0.30	0.45	610	0.393	0.0006
8.00	0.40	0.57	610	0.510	0.0008
10.00	0.50	0.69	610	0.627	0.0010
12.00	0.60	0.64	610	0.662	0.0011
14.00	0.70	0.59	610	0.617	0.0010
16.00	0.80	0.55	610	0.571	0.0009
18.00	0.90	0.50	610	0.525	0.0009
20.00	1.00	0.46	610	0.480	0.0008
22.00	1.10	0.41	610	0.434	0.0007
24.00	1.20	0.37	610	0.388	0.0006
26.00	1.30	0.32	610	0.343	0.0006
28.00	1.40	0.27	610	0.297	0.0005
30.00	1.50	0.23	610	0.251	0.0004
32.00	1.60	0.18	610	0.206	0.0003
34.00	1.70	0.14	2250	0.160	0.0001
36.00	1.80	0.09	2250	0.114	0.0001
38.00	1.90	0.05	2250	0.069	0.0000
40.00	2.00	0.00	2250	0.023	0.0000

Sand and Silt

$\sum I_{zi}/E_{si} = 0.01121$

1) I_{zp} = peak influence factor of soil layer i, **Fig. 3-4** = $0.5+0.11\Delta np/\sigma'_{top}^{1/2} = 0.685$

where, γ' (pcf) = effective unit wei; **0.035** Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing -effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{top} = 0.5 \times B \times \gamma' + D \times \gamma'$ **0.487**

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf). After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978).

Soil Type	N (blows/ft) = 100	L/B = 1
s ₁ sandy silts, slightly cohesive silt-sand mixt		10N = 1000
Clean, fine to medium sands and slightly silty sands		17.5N = 1750
Coarse sands and sands with little gravel		25N = 2500
Sandy gravel and gravel		30N = 3000
		E _{si} (ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.0358 ft	0.43 inches	<1	OK
Long Term	0.0502 ft	0.60 inches	<1	OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od} / \Delta p \geq 0.5$ **0.91**

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf **0.17 = $\gamma' D$**

where, γ_{moist} unit weight (pcf) **120**

γ_{sat} unit weight (pcf) = **131.00**

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ **1.0** (short-term) **1.4** (long-term)

t = time, years (end of construction) **0.1** **10** (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf **0.90**

Δz_i = depth increment i, 0.1B, ft **6.5**

I_{zi} = influence factor of soil layer i, fig. 3-4

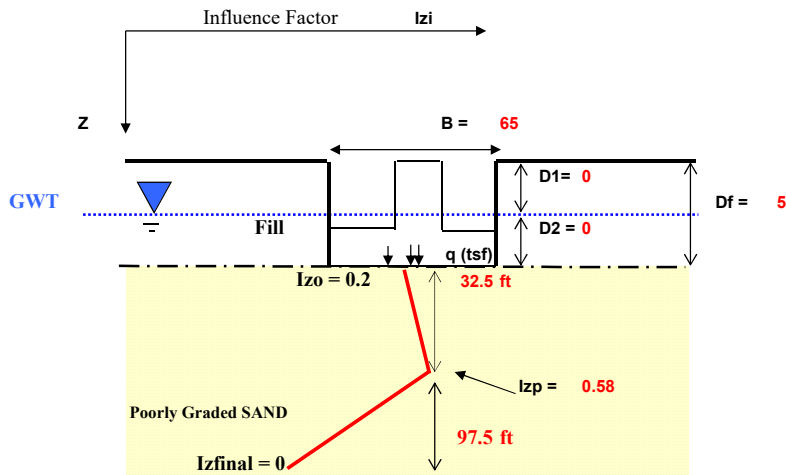
E_{si} = elastic modulus of soil layer i, tsf **481.25**

q = applied pressure at foundation, tsf **1.08** 2150

B = **65**

Df = **5**

GWT depth (ft) = **0**



$$\begin{aligned} I_{zo} &= 0.10 \\ I_{zp} &= 0.58 \\ I_{zp} - I_{zo} &= 0.48 \end{aligned}$$

$$\begin{aligned} Z_{\text{final}} (\text{ft}) &= 130 \\ Z_{zp} (\text{ft}) &= 32.5 \\ Z_{\text{final}} - Z_{zp} &= 97.5 \end{aligned}$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Sand and Silt
6.50	0.10	0.20	350	0.148	0.0004	
13.00	0.20	0.29	350	0.245	0.0007	
19.50	0.30	0.39	350	0.342	0.0010	
26.00	0.40	0.49	350	0.439	0.0013	
32.50	0.50	0.58	350	0.535	0.0015	
39.00	0.60	0.54	2250	0.564	0.0003	
45.50	0.70	0.51	2250	0.525	0.0002	
52.00	0.80	0.47	2250	0.487	0.0002	
58.50	0.90	0.43	2250	0.448	0.0002	
65.00	1.00	0.39	2250	0.409	0.0002	Bedrock
71.50	1.10	0.35	2500	0.370	0.0001	
78.00	1.20	0.31	2500	0.331	0.0001	
84.50	1.30	0.27	2500	0.292	0.0001	
91.00	1.40	0.23	2500	0.253	0.0001	
97.50	1.50	0.19	2500	0.214	0.0001	
104.00	1.60	0.16	2500	0.175	0.0001	
110.50	1.70	0.12	2500	0.136	0.0001	
117.00	1.80	0.08	2500	0.097	0.0000	
123.50	1.90	0.04	2500	0.058	0.0000	
130.00	2.00	0.00	2500	0.019	0.0000	

$$\sum Izi/E_{si} = 0.00674$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1[\Delta p/\sigma'_{i_{zp}}]^{1/2} = 0.584$

where, γ' (pcf) = effective unit weight 0.034
 0.06 Using $\gamma' = \gamma_{\text{sat}} - \gamma_{\text{water}}$ (If soil is below GWT)
Using $\gamma' = \gamma_{\text{moist}}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{i_{zp}} = 0.5 \times B \times \gamma' + D \times \gamma' = 1.286$

2) Elastic Modulus, Esi (ksf)

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N (\text{blows/ft}) = 100$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt
Clean, fine to medium sands and slightly silty sands
Coarse sands and sands with little gravel
Sandy gravel and gravel

L/B = 1

$$\begin{aligned} 10N &= 1000 \\ 17.5N &= 1750 \\ 25N &= 2500 \\ 30N &= 3000 \end{aligned}$$

$$E_{si} (\text{Ksf}) = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.0359 ft	0.43 inches	<1	OK
Long Term	0.0502 ft	0.60 inches	<1	OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od}/\Delta p \geq 0.5$ **0.94**

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf **0.17 = $\gamma' D$**

where, γ_{moist} unit weight (pcf) **120**

γ_{sat} unit weight (pcf) = **132.00**

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ **1.0** (short-term) **1.4** (long-term)

t = time, years (end of construction) **0.1** **10** (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf **1.35**

Δz_i = depth increment i, 0.1B, ft **6.0**

I_{zi} = influence factor of soil layer i, fig. 3-4

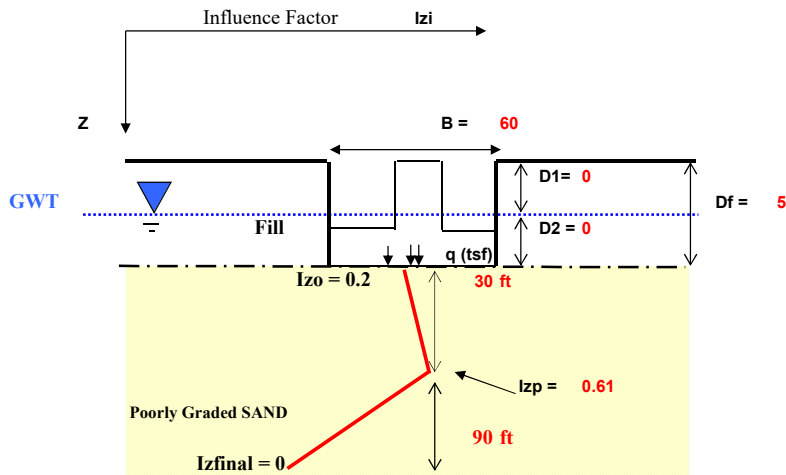
E_{si} = elastic modulus of soil layer i, tsf **481.25**

q = applied pressure at foundation, tsf **1.53** 3050 psf

B = **60**

Df = **5**

GWT depth (ft) = **0**



$$I_{zo} = 0.10$$

$$I_{zp} = 0.61$$

$$I_{zp} - I_{zo} = 0.51$$

$$Z_{final} (ft) = 120$$

$$Z_{zp} (ft) = 30$$

$$Z_{final} - Z_{zp} = 90$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Sand and Silt
6.00	0.10	0.20	610	0.151	0.0002	
12.00	0.20	0.30	610	0.252	0.0004	
18.00	0.30	0.40	610	0.353	0.0006	
24.00	0.40	0.50	610	0.454	0.0007	
30.00	0.50	0.61	610	0.555	0.0009	
36.00	0.60	0.56	2250	0.585	0.0003	
42.00	0.70	0.52	2250	0.545	0.0002	
48.00	0.80	0.48	2250	0.504	0.0002	
54.00	0.90	0.44	2250	0.464	0.0002	
60.00	1.00	0.40	2250	0.424	0.0002	Glacial Till
66.00	1.10	0.36	2250	0.383	0.0002	
72.00	1.20	0.32	3000	0.343	0.0001	
78.00	1.30	0.28	3000	0.303	0.0001	
84.00	1.40	0.24	3000	0.262	0.0001	
90.00	1.50	0.20	3000	0.222	0.0001	
96.00	1.60	0.16	3000	0.182	0.0001	
102.00	1.70	0.12	3000	0.141	0.0000	
108.00	1.80	0.08	3000	0.101	0.0000	
114.00	1.90	0.04	3000	0.061	0.0000	
120.00	2.00	0.00	2500	0.020	0.0000	Bedrock

$$\sum Izi/E_{si} = 0.00473$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1 |\Delta p / \sigma'_{izp}|^{1/2} = 0.605$

where, γ' (pcf) = effective unit weight 0.035 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
 0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{izp} = 0.5 \times B \times \gamma' + D \times \gamma' = 1.218$

2) **Elastic Modulus, Esi (ksf)**

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N \text{ (blows/ft)} = 100$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt
 Clean, fine to medium sands and slightly silty sands
 Coarse sands and sands with little gravel
 Sandy gravel and gravel

L/B = 1

10N = 1000
 17.5N = 1750
 25N = 2500
 30N = 3000

$$Esi \text{ (Ksf)} = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.0476 ft	0.57 inches	<1	OK
Long Term	0.0666 ft	0.80 inches	<1	OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od}/\Delta p \geq 0.5$ **0.97**

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf **0.17 = $\gamma' D$**

where, γ_{moist} unit weight (pcf) **120**

γ_{sat} unit weight (pcf) = **132.00**

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ **1.0** (short-term) **1.4** (long-term)

t = time, years (end of construction) **0.1** **10** (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf **2.60**

Δz_i = depth increment i, 0.1B, ft **1.5**

I_{zi} = influence factor of soil layer i, fig. 3-4

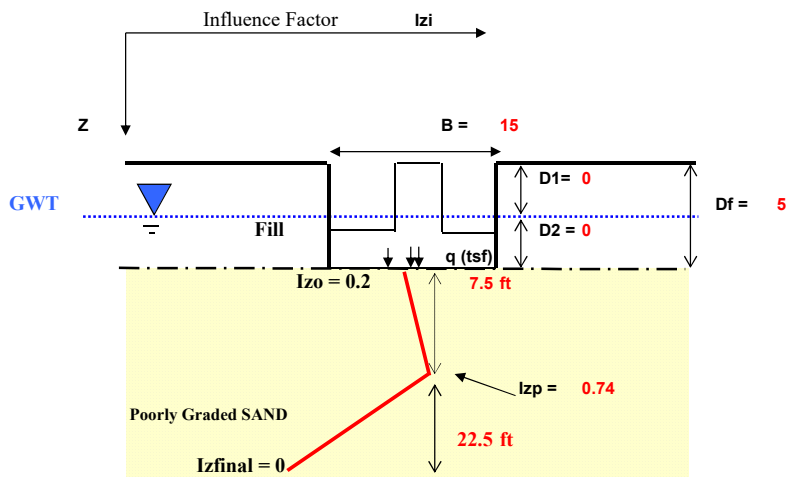
E_{si} = elastic modulus of soil layer i, tsf **481.25**

q = applied pressure at foundation, tsf **2.78** 5550

B = **15**

Df = **5**

GWT depth (ft) = **0**



$$I_{zo} = 0.10$$

$$I_{zp} = 0.74$$

$$I_{zp} - I_{zo} = 0.64$$

$$Z_{final} (ft) = 30$$

$$Z_{zp} (ft) = 7.5$$

$$Z_{final} - Z_{zp} = 22.5$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi
0.00	0.00	0.10			
1.50	0.10	0.23	610	0.164	0.0003
3.00	0.20	0.36	610	0.293	0.0005
4.50	0.30	0.49	610	0.422	0.0007
6.00	0.40	0.62	610	0.551	0.0009
7.50	0.50	0.74	610	0.680	0.0011
9.00	0.60	0.69	610	0.720	0.0012
10.50	0.70	0.65	610	0.670	0.0011
12.00	0.80	0.60	610	0.620	0.0010
13.50	0.90	0.55	610	0.571	0.0009
15.00	1.00	0.50	610	0.521	0.0009
16.50	1.10	0.45	610	0.472	0.0008
18.00	1.20	0.40	610	0.422	0.0007
19.50	1.30	0.35	610	0.372	0.0006
21.00	1.40	0.30	610	0.323	0.0005
22.50	1.50	0.25	610	0.273	0.0004
24.00	1.60	0.20	610	0.223	0.0004
25.50	1.70	0.15	610	0.174	0.0003
27.00	1.80	0.10	610	0.124	0.0002
28.50	1.90	0.05	610	0.074	0.0001
30.00	2.00	0.00	610	0.025	0.0000

Sand and Silt

$$\sum Izi/E_{si} = 0.01262$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1[\Delta p/\sigma'_{i_{zp}}]^{1/2} = 0.745$

where, γ' (pcf) = effective unit weight 0.035
0.06 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing -effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{i_{zp}} = 0.5 \times B \times \gamma' + D \times \gamma' = 0.435$

2) Elastic Modulus, Esi (ksf)

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N (\text{blows/ft}) = 61$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt
Clean, fine to medium sands and slightly silty sands
Coarse sands and sands with little gravel
Sandy gravel and gravel

L/B = 1

10N = 610
17.5N = 1067.5
25N = 1525
30N = 1830

$$Esi (Ksf) = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 \cdot C_2 \cdot \Delta p \cdot \Delta z_i \sum \frac{I_{zi}}{E_{si}}$$

Short Term

Long Term

0.0474 ft

0.0664 ft

0.57 inches

0.80 inches

<1

<1

OK

OK

where,

C₁ = correction to account for strain relief from embedment, 1-0.5σ'_{vd}/Δp>=0.5

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D, tsf

where,

γ_{sat} unit weight (pcf) = 120

C₂ = correction for time dependent increase in settlement, 1+0.2 log₁₀(t/0.1)

t = time, years (end of constructi

Δp = net applied footing pressure, q-σ'_{vd}, tsf

Δz_i = depth increment i, 0.1B, ft

I_{zi} = influence factor of soil layer i, fig. 3-4

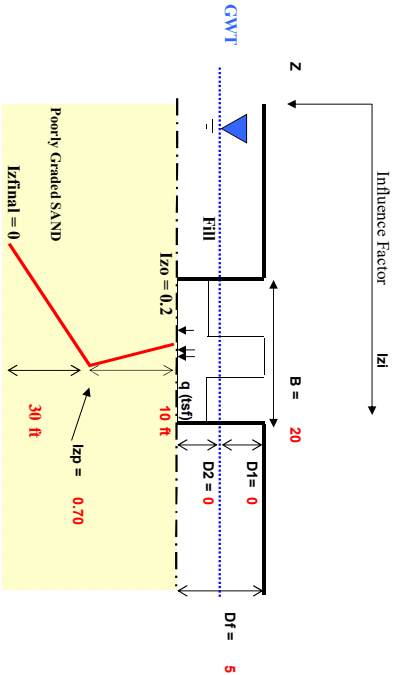
E_{si} = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

B = 20

Df = 5

GWT depth (ft) = 0



$I_{zo} = 0.10$
 $I_{zp} = 0.70$
 $I_{zp}-I_{zo} = 0.60$

$Z_{final (ft)} = 40$
 $Z_{sp (ft)} = 10$
 $Z_{final} - Z_{sp} = 30$

z, ft	z/B	Izi	Es, tsf	Iza	Iza/Es
0.00	0.00	0.10			
2.00	0.10	0.22	610	0.160	0.0003
4.00	0.20	0.34	610	0.281	0.0005
6.00	0.30	0.46	610	0.401	0.0007
8.00	0.40	0.58	610	0.522	0.0009
10.00	0.50	0.70	610	0.643	0.0011
12.00	0.60	0.66	610	0.680	0.0011
14.00	0.70	0.61	610	0.633	0.0010
16.00	0.80	0.56	610	0.586	0.0010
18.00	0.90	0.52	610	0.539	0.0009
20.00	1.00	0.47	610	0.492	0.0008
22.00	1.10	0.42	610	0.445	0.0007
24.00	1.20	0.37	610	0.398	0.0007
26.00	1.30	0.33	610	0.351	0.0006
28.00	1.40	0.28	610	0.305	0.0005
30.00	1.50	0.23	610	0.258	0.0004
32.00	1.60	0.19	610	0.211	0.0003
34.00	1.70	0.14	2250	0.164	0.0001
36.00	1.80	0.09	2250	0.117	0.0001
38.00	1.90	0.05	2250	0.070	0.0000
40.00	2.00	0.00	2250	0.023	0.0000

Sand and Silt
Till
Glacial

I_{zp} = peak influence factor of soil layer i, Fig. 3-4 = $0.5+0.1I_{ap}/\sigma'_{vp}$ ^{1/2} = 0.703

$\sum Izi/E_{si}$ = 0.01149

where, γ' (pcf) = effective unit weight 0.035
Using $\gamma' = \gamma$ sat - γ water (If soil is below GWT)
Using $\gamma' = \gamma$ moist (If soil is above GWT) 0.06

Axisymmetric (L/B=1) - Spread/column footing
 $\sigma'_{vp} = 0.5 \gamma B \times \gamma' + D \times \gamma' = 0.522$
-effective overburden pressure at depth I_{zp} , tsf

2) Elastic Modulus, Es (ksf)

Convert SPT (blows/ft) to Es (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

Soil Type	N (blows/ft) = 100	L/B = 1
s, sandy silts, slightly cohesive silt-sand mixtu		10N = 1000
Clean, fine to medium sands and slightly silty sands		17.5N = 1750
Coarse sands and sands with little gravel		25N = 2500
Sandy gravel and gravel		30N = 3000
		Es (ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$p_i = C_i * C_i * \Delta p * \Delta z_i \sum z_i /E_{si}$	Short Term	<u>0.0476</u> ft	<u>0.57</u> inches	<1	OK
	Long Term	<u>0.0666</u> ft	<u>0.80</u> inches	<1	OK

where,

C_1 = correction to account for strain relief from embedment, $1-0.5 \sigma'_{od}/\Delta p > 0$;

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D, is

where, γ_{moist} unit weight (pcf) **120**

γ_{sat} unit weight (pcf) = **132.00**

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10} (t / 0.1)$

	0.1 (short term)	10 (long term)
t = time, years	(end of construction)	(end of construction)

 $\Delta p = \text{net applied footing pressure, } q - \sigma'_{\text{old}}, \text{ tsf}$ Δz_i = depth increment i , 0.1 B, ft

Izi = influence factor of soil layer i, fig. 3-4

E_{si} = elastic modulus of soil layer i , tsf

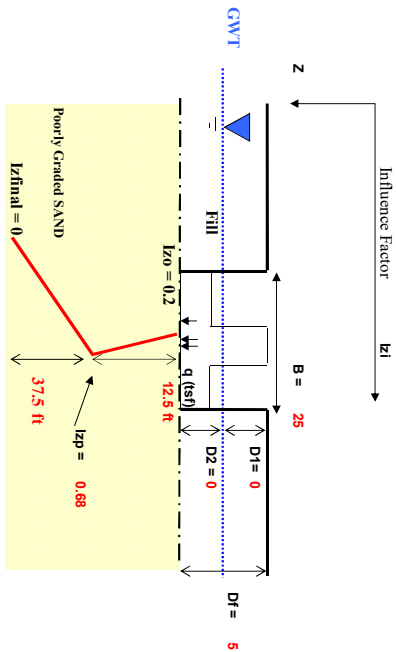
q = applied pressure at foundation, tsf

B = 25

$$Df = 5$$

$$GW/T \text{ doth } (f) = 0$$

GW depth (ft) = 0



$I_{zo} = 0.10$
 $I_{zp} = 0.68$
 $I_{zp} - I_{zo} = 0.58$

$Z_{final(t)} = 50$
 $Z_{np(t)} = 12.5$
 $Z_{final} - Z_{np} = 37.5$

z, ft	z/B	I _{zi}	E _{si} , tsf	I _{za}	I _{za} /E _{si}
0.00	0.00	0.10			
2.50	0.10	0.22	610	0.158	0.0003
5.00	0.20	0.33	610	0.274	0.0004
7.50	0.30	0.45	610	0.389	0.0006
10.00	0.40	0.56	610	0.505	0.0008
12.50	0.50	0.68	610	0.621	0.0010
15.00	0.60	0.63	610	0.656	0.0011
17.50	0.70	0.59	610	0.611	0.0010
20.00	0.80	0.54	610	0.566	0.0009
22.50	0.90	0.50	610	0.521	0.0009
25.00	1.00	0.45	610	0.475	0.0008
27.50	1.10	0.41	610	0.430	0.0007
30.00	1.20	0.36	610	0.385	0.0006
32.50	1.30	0.32	610	0.339	0.0006
35.00	1.40	0.27	2250	0.294	0.0001
37.50	1.50	0.23	2250	0.249	0.0001
40.00	1.60	0.18	2250	0.204	0.0001
42.50	1.70	0.14	2250	0.158	0.0001
45.00	1.80	0.09	2250	0.113	0.0001
47.50	1.90	0.05	2250	0.068	0.0000
50.00	2.00	0.00	2250	0.023	0.0000

Sand and Silt
Glacial Till

$\sum I_{zi}/E_{si} = 0.01022$

1) I_{zp} = peak influence factor of soil layer i, **fig. 3-4** = $0.5 + 0.11 \Delta np \sigma'_{top}{}^{1/2} = 0.679$

where, γ' (pcf) = effective unit wei; **0.035** Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing
 $\sigma'_{top} = 0.5 \times B \times \gamma' + D \times \gamma' = 0.609$ -effective overburden pressure at depth I_{zp} , tsf

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf). After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978).

Soil Type	N (blows/ft) = 100	L/B = 1
s ₁ sandy silts, slightly cohesive silt-sand mixt		10N = 1000
Clean, fine to medium sands and slightly silty sands		17.5N = 1750
Coarse sands and sands with little gravel		25N = 2500
Sandy gravel and gravel		30N = 3000
		E _{si} (ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.0476 ft	0.57 inches <1	OK
Long Term	0.0666 ft	0.80 inches <1	OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od}/\Delta p \geq 0.5$ **0.95**

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf **0.17 = $\gamma' D$**

where, γ_{moist} unit weight (pcf) **120**

γ_{sat} unit weight (pcf) = **131.00**

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ **1.0** (short-term) **1.4** (long-term)

t = time, years (end of construction) **0.1** **10** (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf **1.63**

Δz_i = depth increment i, 0.1B, ft **1.5**

I_{zi} = influence factor of soil layer i, fig. 3-4

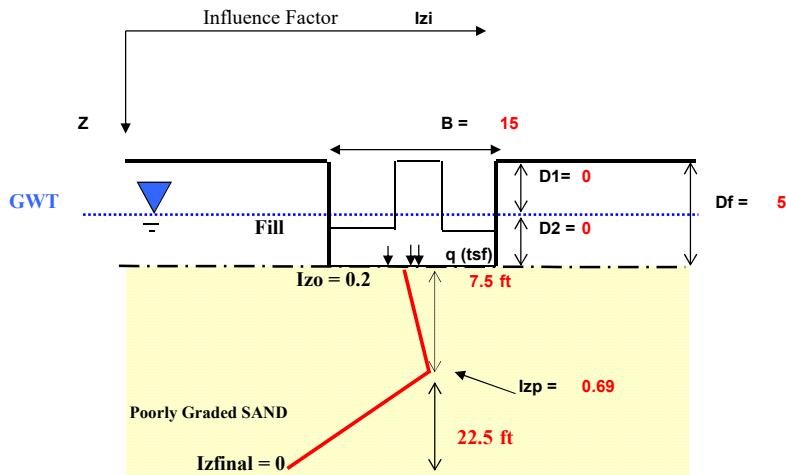
E_{si} = elastic modulus of soil layer i, tsf **481.25**

q = applied pressure at foundation, tsf **1.80** 3600

B = **15**

Df = **5**

GWT depth (ft) = **0**



$$I_{zo} = 0.10$$

$$I_{zp} = 0.69$$

$$I_{zp} - I_{zo} = 0.59$$

$$Z_{final} (ft) = 30$$

$$Z_{zp} (ft) = 7.5$$

$$Z_{final} - Z_{zp} = 22.5$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Sand and Silt
1.50	0.10	0.22	350	0.159	0.0005	
3.00	0.20	0.34	350	0.278	0.0008	
4.50	0.30	0.46	350	0.397	0.0011	
6.00	0.40	0.58	350	0.516	0.0015	
7.50	0.50	0.69	350	0.635	0.0018	
9.00	0.60	0.65	350	0.672	0.0019	
10.50	0.70	0.60	350	0.625	0.0018	
12.00	0.80	0.56	350	0.579	0.0017	
13.50	0.90	0.51	350	0.533	0.0015	
15.00	1.00	0.46	350	0.486	0.0014	
16.50	1.10	0.42	350	0.440	0.0013	
18.00	1.20	0.37	350	0.394	0.0011	
19.50	1.30	0.32	350	0.347	0.0010	
21.00	1.40	0.28	350	0.301	0.0009	
22.50	1.50	0.23	350	0.255	0.0007	Clacial Till
24.00	1.60	0.19	350	0.208	0.0006	
25.50	1.70	0.14	350	0.162	0.0005	
27.00	1.80	0.09	350	0.116	0.0003	
28.50	1.90	0.05	350	0.069	0.0002	
30.00	2.00	0.00	350	0.023	0.0001	

$$\sum Izi/E_{si} = 0.02057$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1[\Delta p/\sigma'_{i_{zp}}]^{1/2} = 0.695$

where, γ' (pcf) = effective unit weight **0.034** Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{i_{zp}} = 0.5 \times B \times \gamma' + D \times \gamma' = 0.429$

2) Elastic Modulus, Esi (ksf)

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N (\text{blows/ft}) = 61$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt
 Clean, fine to medium sands and slightly silty sands
 Coarse sands and sands with little gravel
 Sandy gravel and gravel

L/B = 1

10N = **610**
 17.5N = **1067.5**
 25N = **1525**
 30N = **1830**

$$Esi (Ksf) = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.0476 ft	0.57 inches	<1	OK
Long Term	0.0667 ft	0.80 inches	<1	OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od} / \Delta p \geq 0.5$ 0.94

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf 0.17 = $\gamma' D$

where, γ_{moist} unit weight (pcf) 120

γ_{sat} unit weight (pcf) = 131.00

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ 1.0 (short-term) 1.4 (long-term)

t = time, years (end of construction) 0.1 10 (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf 1.35

Δz_i = depth increment i, 0.1B, ft 2.0

I_{zi} = influence factor of soil layer i, fig. 3-4

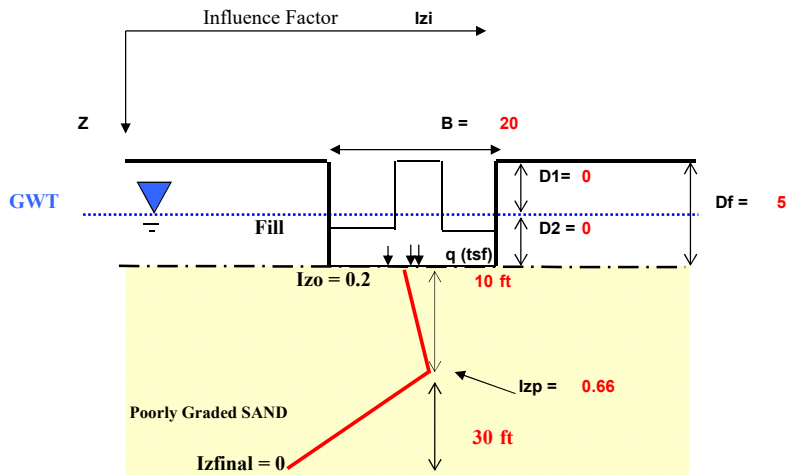
E_{si} = elastic modulus of soil layer i, tsf 481.25

q = applied pressure at foundation, tsf 1.53 3050

B = 20

Df = 5

GWT depth (ft) = 0



$$I_{zo} = 0.10$$

$$I_{zp} = 0.66$$

$$I_{zp} - I_{zo} = 0.56$$

$$Z_{final} (ft) = 40$$

$$Z_{zp} (ft) = 10$$

$$Z_{final} - Z_{zp} = 30$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Sand and Silt
2.00	0.10	0.21	350	0.156	0.0004	
4.00	0.20	0.32	350	0.269	0.0008	
6.00	0.30	0.44	350	0.381	0.0011	
8.00	0.40	0.55	350	0.494	0.0014	
10.00	0.50	0.66	350	0.606	0.0017	
12.00	0.60	0.62	350	0.640	0.0018	
14.00	0.70	0.57	350	0.596	0.0017	
16.00	0.80	0.53	350	0.552	0.0016	
18.00	0.90	0.49	350	0.508	0.0015	
20.00	1.00	0.44	350	0.464	0.0013	
22.00	1.10	0.40	350	0.419	0.0012	
24.00	1.20	0.35	350	0.375	0.0011	
26.00	1.30	0.31	350	0.331	0.0009	
28.00	1.40	0.26	350	0.287	0.0008	
30.00	1.50	0.22	350	0.243	0.0007	Clacial Till
32.00	1.60	0.18	350	0.199	0.0006	
34.00	1.70	0.13	2250	0.155	0.0001	
36.00	1.80	0.09	2250	0.110	0.0000	
38.00	1.90	0.04	2250	0.066	0.0000	
40.00	2.00	0.00	2250	0.022	0.0000	

$$\sum Izi/E_{si} = 0.01878$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1[\Delta p/\sigma'_{i_{zp}}]^{1/2} = 0.662$

where, γ' (pcf) = effective unit weight 0.034
 0.06 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{i_{zp}} = 0.5 \times B \times \gamma' + D \times \gamma' = 0.515$

2) Elastic Modulus, Esi (ksf)

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N (\text{blows/ft}) = 61$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt
Clean, fine to medium sands and slightly silty sands
Coarse sands and sands with little gravel
Sandy gravel and gravel

L/B = 1

10N = 610
17.5N = 1067.5
25N = 1525
30N = 1830

$$Esi (Ksf) = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 \cdot C_2 \cdot \Delta p \cdot \Delta z_i \sum \frac{I_z I_{zi}}{E_{si}}$$

Short Term

Long Term

0.0429 ft

0.0670 ft

0.57 inches

0.80 inches

<1

<1

OK

OK

where,

C₁ = correction to account for strain relief from embedment, 1-0.5σ'_{vd}/Δp>=0.5

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D, tsf

where,

γ_{sat} unit weight (pcf) = 131.00

C₂ = correction for time dependent increase in settlement, 1+0.2 log₁₀(t/0.1)

t = time, years (end of construction) 0.1

Δp = net applied footing pressure, q-σ'_{vd}, tsf

Δz_i = depth increment i, 0.1B, ft

I_{zi} = influence factor of soil layer i, fig. 3-4

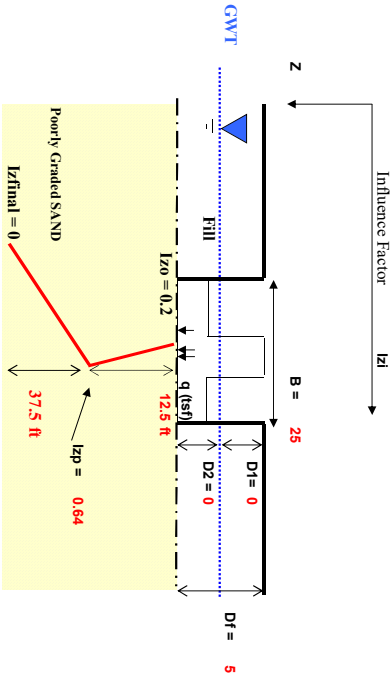
E_{si} = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

B = 25

Df = 5

GWT depth (ft) = 0



$l_{zo} = 0.10$
 $l_{zp} = 0.64$
 $l_{zp}-l_{zo} = 0.54$

$Z_{final (ft)} = 50$
 $Z_{sp (ft)} = 12.5$
 $Z_{final} - Z_{sp} = 37.5$

z, ft	z/B	l _{zi}	E _{si} , tsf	I _{za}	I _{za} /E _{si}
0.00	0.00	0.10			
2.50	0.10	0.21	350	0.154	0.0004
5.00	0.20	0.32	350	0.263	0.0008
7.50	0.30	0.43	350	0.372	0.0011
10.00	0.40	0.54	350	0.481	0.0014
12.50	0.50	0.64	350	0.589	0.0017
15.00	0.60	0.60	350	0.622	0.0018
17.50	0.70	0.56	350	0.579	0.0017
20.00	0.80	0.52	350	0.536	0.0015
22.50	0.90	0.47	350	0.494	0.0014
25.00	1.00	0.43	350	0.451	0.0013
27.50	1.10	0.39	350	0.408	0.0012
30.00	1.20	0.34	350	0.365	0.0010
32.50	1.30	0.30	350	0.322	0.0009
35.00	1.40	0.26	2250	0.279	0.0001
37.50	1.50	0.21	2250	0.236	0.0001
40.00	1.60	0.17	2250	0.193	0.0001
42.50	1.70	0.13	2250	0.150	0.0001
45.00	1.80	0.09	2250	0.107	0.0000
47.50	1.90	0.04	2250	0.064	0.0000
50.00	2.00	0.00	2250	0.021	0.0000

Sand and Silt
Glacial Till

$\sum I_{zi}/E_{si} = 0.01657$

1) l_{zp} = Peak influence factor of soil layer i, Fig. 3-4 = $0.5+0.1(l_{ap}/\sigma'_{top})^{1/2} = 0.644$

where, γ' (pcf) = effective unit weig **0.034** Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing
 $\sigma'_{top} = 0.5 \times B \times \gamma' + D \times \gamma'$ **0.600** -effective overburden pressure at depth l_{zp} , tsf

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

Soil Type	N (blows/ft) = 100	$L/B = 1$
s, sandy silt, slightly cohesive silt-sand mixtu		10N = 1000
Clean, fine to medium sands and slightly silty sands		17.5N = 1750
Coarse sands and sands with little gravel		25N = 2500
Sandy gravel and gravel		30N = 3000

E_{si} (ksf) = **962.5**

$I_{zo} = 0.10$
 $I_{zp} = 0.75$
 $I_{zp}-I_{zo} = 0.65$

$Z_{final(t)} = 30$
 $Z_{np(t)} = 7.5$
 $Z_{final} - Z_{np} = 22.5$

z, ft	z/B	I _{zi}	E _{si} , tsf	I _{za}	I _{za} /E _{si}
0.00	0.00	0.10			
1.50	0.10	0.23	610	0.165	0.0003
3.00	0.20	0.36	610	0.296	0.0005
4.50	0.30	0.49	610	0.426	0.0007
6.00	0.40	0.62	610	0.557	0.0009
7.50	0.50	0.75	610	0.688	0.0011
9.00	0.60	0.70	610	0.728	0.0012
10.50	0.70	0.65	610	0.678	0.0011
12.00	0.80	0.60	610	0.627	0.0010
13.50	0.90	0.55	610	0.577	0.0009
15.00	1.00	0.50	610	0.527	0.0009
16.50	1.10	0.45	610	0.477	0.0008
18.00	1.20	0.40	610	0.427	0.0007
19.50	1.30	0.35	610	0.376	0.0006
21.00	1.40	0.30	610	0.326	0.0005
22.50	1.50	0.25	610	0.276	0.0005
24.00	1.60	0.20	610	0.226	0.0004
25.50	1.70	0.15	610	0.176	0.0003
27.00	1.80	0.10	610	0.125	0.0002
28.50	1.90	0.05	610	0.075	0.0001
30.00	2.00	0.00	610	0.025	0.0000

Sand and Silt

$\sum I_{zi}/E_{si} = 0.01275$

1) I_{zp} = peak influence factor of soil layer i, **Fig. 3-4** = $0.5+0.11\Delta np\sigma'_{top}{}^{1/2} = 0.753$

where, γ' (pcf)= effective unit wei; **0.035** Using $\gamma' = \gamma$ sat - γ water (If soil is below GWT)
0.06 Using $\gamma' = \gamma$ moist (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing

$\sigma'_{top} = 0.5 \times B \times \gamma' + D \times \gamma'$ **0.400**

-effective overburden pressure at depth I_{zp} , tsf

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf) . After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978).

N (blows/ft) = **61**

Soil Type	$L/B = 1$
s ₁ sandy silts, slightly cohesive silt-sand mixt	610
Clean, fine to medium sands and slightly silty sands	1067.5
Coarse sands and sands with little gravel	1525
Sandy gravel and gravel	1830

E_{si} (ksf) = **962.5**

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis", 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 \cdot C_2 \cdot \Delta p \cdot \Delta z_i \sum \frac{I_z}{E_{si}}$$

Short Term

Long Term

$\frac{0.0479}{0.0670}$ ft

$\frac{0.57}{0.80}$ inches

<1

<1

OK

OK

where,

C_1 = correction to account for strain relief from embedment, $1-0.5 \sigma'_{vd}/\Delta p \geq 0$;

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D , ts

where,

$$\gamma_{ moist}$$

unit weight (pcf)

120

132.00

C_2 = correction for time dependent increase in settlement, $1+0.2 \log_{10} (t / 0.1)$

t = time, years (end of construct

Δp = net applied footing pressure, $q - \sigma'_{vd}$, tsf

Δz_i = depth increment i , 0.1B, ft

I_z = influence factor of soil layer i , fig. 3-4

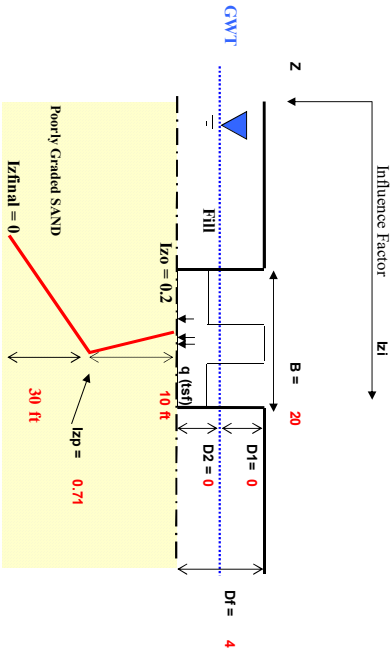
E_{si} = elastic modulus of soil layer i , tsf

q = applied pressure at foundation, tsf

B = 20

D_f = 4

GWT depth (ft) = 0



$I_{zo} = 0.10$
 $I_{zp} = 0.71$
 $I_{zp}-I_{zo} = 0.61$

$Z_{final(t)} = 40$
 $Z_{np(t)} = 10$
 $Z_{final} - Z_{np} = 30$

z, ft	z/B	I _{zi}	E _{si} , tsf	I _{za}	I _{za} /E _{si}
0.00	0.00	0.10			
2.00	0.10	0.22	610	0.161	0.0003
4.00	0.20	0.34	610	0.283	0.0005
6.00	0.30	0.47	610	0.405	0.0007
8.00	0.40	0.59	610	0.527	0.0009
10.00	0.50	0.71	610	0.648	0.0011
12.00	0.60	0.66	610	0.686	0.0011
14.00	0.70	0.61	610	0.638	0.0010
16.00	0.80	0.57	610	0.591	0.0010
18.00	0.90	0.52	610	0.544	0.0009
20.00	1.00	0.47	610	0.497	0.0008
22.00	1.10	0.43	610	0.449	0.0007
24.00	1.20	0.38	610	0.402	0.0007
26.00	1.30	0.33	610	0.355	0.0006
28.00	1.40	0.28	610	0.307	0.0005
30.00	1.50	0.24	610	0.260	0.0004
32.00	1.60	0.19	610	0.213	0.0003
34.00	1.70	0.14	2250	0.166	0.0001
36.00	1.80	0.09	2250	0.118	0.0001
38.00	1.90	0.05	2250	0.071	0.0000
40.00	2.00	0.00	2250	0.024	0.0000

Sand and Silt

$\sum I_{zi}/E_{si} = 0.01159$

1) I_{zp} = peak influence factor of soil layer i, **Fig. 3-4** = $0.5+0.11\Delta np/\sigma'_{top}^{1/2}$ = **0.709**

where, γ' (pcf) = effective unit wei; **0.035** Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing -effective overburden pressure at depth I_{zp} , tsf

$\sigma'_{top} = 0.5 \times B \times \gamma' + D \times \gamma'$ **0.487**

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf). After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978).

Soil Type	N (blows/ft) = 100	L/B = 1
s ₁ sandy silts, slightly cohesive silt-sand mixt		10N = 1000
Clean, fine to medium sands and slightly silty sands		17.5N = 1750
Coarse sands and sands with little gravel		25N = 2500
Sandy gravel and gravel		30N = 3000
		E _{si} (ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.0476 ft	0.57 inches	<1	OK
Long Term	0.0666 ft	0.80 inches	<1	OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od} / \Delta p \geq 0.5$ **0.93**

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf **0.17 = $\gamma' D$**

where, γ_{moist} unit weight (pcf) **120**

γ_{sat} unit weight (pcf) = **131.00**

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ **1.0** (short-term) **1.4** (long-term)

t = time, years (end of construction) **0.1** **10** (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf **1.15**

Δz_i = depth increment i, 0.1B, ft **6.5**

I_{zi} = influence factor of soil layer i, fig. 3-4

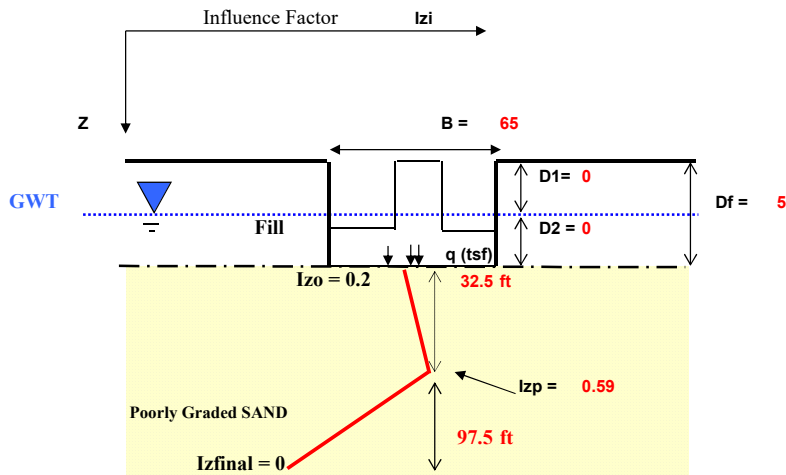
E_{si} = elastic modulus of soil layer i, tsf **481.25**

q = applied pressure at foundation, tsf **1.33** 2650

B = **65**

Df = **5**

GWT depth (ft) = **0**



$$\begin{aligned} I_{zo} &= 0.10 \\ I_{zp} &= 0.59 \\ I_{zp} - I_{zo} &= 0.49 \end{aligned}$$

$$\begin{aligned} Z_{\text{final (ft)}} &= 130 \\ Z_{zp \text{ (ft)}} &= 32.5 \\ Z_{\text{final}} - Z_{zp} &= 97.5 \end{aligned}$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Sand and Silt
6.50	0.10	0.20	350	0.149	0.0004	
13.00	0.20	0.30	350	0.248	0.0007	
19.50	0.30	0.40	350	0.347	0.0010	
26.00	0.40	0.50	350	0.446	0.0013	
32.50	0.50	0.59	350	0.545	0.0016	
39.00	0.60	0.56	2250	0.575	0.0003	
45.50	0.70	0.52	2250	0.535	0.0002	
52.00	0.80	0.48	2250	0.496	0.0002	
58.50	0.90	0.44	2250	0.456	0.0002	
65.00	1.00	0.40	2250	0.416	0.0002	Bedrock
71.50	1.10	0.36	2500	0.377	0.0002	
78.00	1.20	0.32	2500	0.337	0.0001	
84.50	1.30	0.28	2500	0.297	0.0001	
91.00	1.40	0.24	2500	0.258	0.0001	
97.50	1.50	0.20	2500	0.218	0.0001	
104.00	1.60	0.16	2500	0.178	0.0001	
110.50	1.70	0.12	2500	0.139	0.0001	
117.00	1.80	0.08	2500	0.099	0.0000	
123.50	1.90	0.04	2500	0.059	0.0000	
130.00	2.00	0.00	2500	0.020	0.0000	

$$\sum Izi/E_{si} = 0.00686$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1[\Delta p/\sigma'_{i_{zp}}]^{1/2} = 0.595$

where, γ' (pcf) = effective unit weight 0.034 Using $\gamma' = \gamma_{\text{sat}} - \gamma_{\text{water}}$ (If soil is below GWT)
 0.06 Using $\gamma' = \gamma_{\text{moist}}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{i_{zp}} = 0.5 \times B \times \gamma' + D \times \gamma' = 1.286$

2) Elastic Modulus, Esi (ksf)

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N \text{ (blows/ft)} = 100$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt
 Clean, fine to medium sands and slightly silty sands
 Coarse sands and sands with little gravel
 Sandy gravel and gravel

L/B = 1

$$\begin{aligned} 10N &= 1000 \\ 17.5N &= 1750 \\ 25N &= 2500 \\ 30N &= 3000 \end{aligned}$$

$$E_{si} \text{ (Ksf)} = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

**Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.*

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$	Short Term	<u>0.0475</u> ft	<u>0.57</u> inches	<1	OK
	Long Term	<u>0.0664</u> ft	<u>0.80</u> inches	<1	OK

where,

C_1 = correction to account for strain relief from embedment, $1-0.5\sigma'_{od}/\Delta p \geq 0.5$ 0.95

$$\sigma'_{od} = \text{effective vertical overburden pressure at bottom of footing or depth } D, \text{ ts} \quad \mathbf{0.17} = \gamma' D$$

where, γ_{moist} unit weight (pcf) 120

$$\gamma_{\text{sat}} \text{ unit weight (pcf)} = 132.00$$

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ **1.0** (short-term) **1.4** (long-term)

t = time, years (end of construction) **0.1** **10** (long term)

$$\Delta p = \text{net applied footing pressure, } q - \sigma'_{od}, \text{ tsf}$$
 Δz_i = depth increment i , 0.1B, ft

I_{zi} = influence factor of soil layer i , fig. 3-4

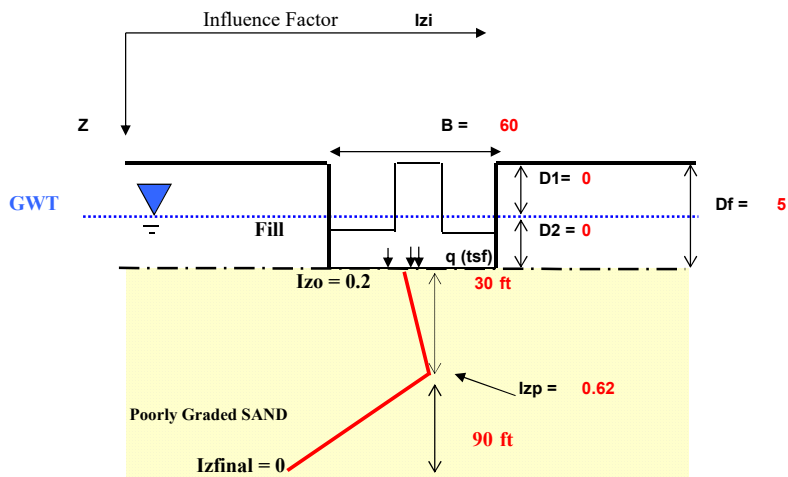
E_{si} = elastic modulus of soil layer i , tsf

q = applied pressure at foundation, tsf

B = 60

$$Df = 5$$

GWT depth (ft) = 0



$$I_{zo} = 0.10$$

$$I_{zp} = 0.62$$

$$I_{zp} - I_{zo} = 0.52$$

$$Z_{final} (ft) = 120$$

$$Z_{zp} (ft) = 30$$

$$Z_{final} - Z_{zp} = 90$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Sand and Silt
6.00	0.10	0.20	610	0.152	0.0002	
12.00	0.20	0.31	610	0.256	0.0004	
18.00	0.30	0.41	610	0.360	0.0006	
24.00	0.40	0.52	610	0.463	0.0008	
30.00	0.50	0.62	610	0.567	0.0009	
36.00	0.60	0.58	2250	0.598	0.0003	
42.00	0.70	0.54	2250	0.557	0.0002	
48.00	0.80	0.50	2250	0.516	0.0002	
54.00	0.90	0.45	2250	0.475	0.0002	
60.00	1.00	0.41	2250	0.433	0.0002	Glacial Till
66.00	1.10	0.37	2250	0.392	0.0002	
72.00	1.20	0.33	3000	0.351	0.0001	
78.00	1.30	0.29	3000	0.310	0.0001	
84.00	1.40	0.25	3000	0.268	0.0001	
90.00	1.50	0.21	3000	0.227	0.0001	
96.00	1.60	0.17	3000	0.186	0.0001	
102.00	1.70	0.12	3000	0.144	0.0000	
108.00	1.80	0.08	3000	0.103	0.0000	
114.00	1.90	0.04	3000	0.062	0.0000	
120.00	2.00	0.00	2500	0.021	0.0000	Weathered Bedrock

$$\sum Izi/E_{si} = 0.00483$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1 |\Delta p / \sigma'_{i, zp}|^{1/2} = 0.619$

where, γ' (pcf) = effective unit weight 0.035 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
 0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{i, zp} = 0.5 \times B \times \gamma' + D \times \gamma' = 1.218$

2) Elastic Modulus, Esi (ksf)

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N \text{ (blows/ft)} = 100$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt
 Clean, fine to medium sands and slightly silty sands
 Coarse sands and sands with little gravel
 Sandy gravel and gravel

L/B = 1

10N = 1000
 17.5N = 1750
 25N = 2500
 30N = 3000

$$E_{si} \text{ (Ksf)} = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 \cdot C_t \cdot \Delta p \cdot \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.0559 ft	0.67 inches <1	OK
Long Term	0.0782 ft	0.94 inches <1	OK

where,

C_1

= correction to account for strain relief from embedment, $1 - 0.5 \sigma'_{od} / \Delta p \geq 0.5$

0.97

σ'_{od}

= effective vertical overburden pressure at bottom of footing or depth D, tsf

0.17

= $\gamma' D$

where,

γ_{moist}

unit weight (pcf)

120

γ_{sat}

unit weight (pcf)

132.00

C_t

= correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$

1.0

(short-term)

1.4

(long-term)

t

= time, years (end of constructic

0.1

10

(long term)

Δp

= net applied footing pressure, $q - \sigma'_{od}$, tsf

2.98

Δz_i

= depth increment i, 0.1B, ft

1.5

I_{zi}

= influence factor of soil layer i, fig. 3-4

E_{si}

= elastic modulus of soil layer i, tsf

481.25

q

= applied pressure at foundation, tsf

3.15

6300

B

=

15

D_f

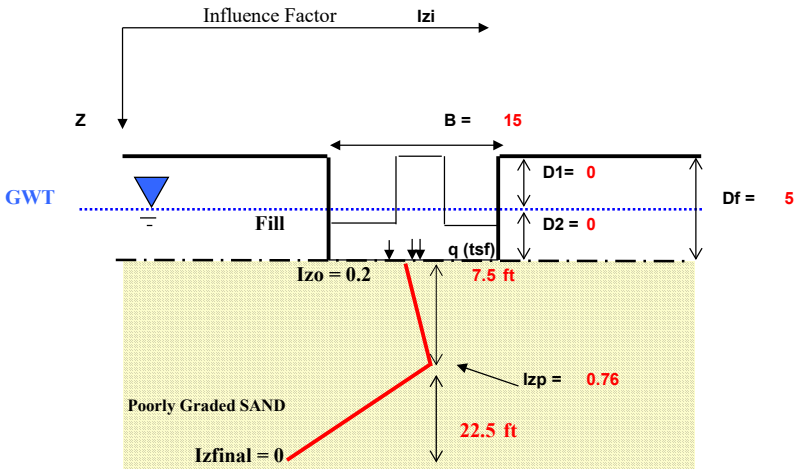
=

5

GWT depth (ft)

=

0



$$I_{zo} = 0.10$$

$$I_{zp} = 0.76$$

$$I_{zp} - I_{zo} = 0.66$$

$$Z_{final (ft)} = 30$$

$$Z_{zp (ft)} = 7.5$$

$$Z_{final} - Z_{zp} = 22.5$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi
0.00	0.00	0.10			
1.50	0.10	0.23	610	0.166	0.0003
3.00	0.20	0.36	610	0.298	0.0005
4.50	0.30	0.50	610	0.431	0.0007
6.00	0.40	0.63	610	0.563	0.0009
7.50	0.50	0.76	610	0.695	0.0011
9.00	0.60	0.71	610	0.736	0.0012
10.50	0.70	0.66	610	0.685	0.0011
12.00	0.80	0.61	610	0.635	0.0010
13.50	0.90	0.56	610	0.584	0.0010
15.00	1.00	0.51	610	0.533	0.0009
16.50	1.10	0.46	610	0.482	0.0008
18.00	1.20	0.41	610	0.432	0.0007
19.50	1.30	0.36	610	0.381	0.0006
21.00	1.40	0.30	610	0.330	0.0005
22.50	1.50	0.25	610	0.279	0.0005
24.00	1.60	0.20	610	0.228	0.0004
25.50	1.70	0.15	610	0.178	0.0003
27.00	1.80	0.10	610	0.127	0.0002
28.50	1.90	0.05	610	0.076	0.0001
30.00	2.00	0.00	610	0.025	0.0000

Sand and Silt

$$\sum Izi/E_{si} = 0.01289$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1 |\Delta p / \sigma'_{izp}|^{1/2} = 0.762$

where, γ' (pcf) = effective unit weight 0.035
 0.06

Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{izp} = 0.5 \times B \times \gamma' + D \times \gamma' = 0.435$

2) **Elastic Modulus, Esi (ksf)**

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N \text{ (blows/ft)} = 61$$

Soil Type

ts, sandy silts, slightly cohesive silt-sand mixtu
 Clean, fine to medium sands and slightly silty sands
 Coarse sands and sands with little gravel
 Sandy gravel and gravel

L/B = 1

$$10N = 610$$

$$17.5N = 1067.5$$

$$25N = 1525$$

$$30N = 1830$$

$$Esi \text{ (Ksf)} = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 * C_2 * \Delta p * A_z \sum \frac{I_z I_{z_i}}{E_{si}}$$

Short Term

Long Term

0.0573 ft

0.0803 ft

0.69 inches

0.96 inches

<1

<1

OK

OK

where,

C₁ = correction to account for strain relief from embedment, 1-0.5σ'_{vd}/Δp>0.5

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D, tsf

where,

γ_{moist} unit weight (pcf) = 120

γ_{sat} unit weight (pcf) = 132.00

C₂ = correction for time dependent increase in settlement, 1+0.2 log₁₀ (t / 0.1)

t = time, years (end of constructi

0.1 10 (long term)

Δp = net applied footing pressure, q-σ'_{vd}, tsf

Δz_i = depth increment i, 0.1B, ft

I_{zi} = influence factor of soil layer i, fig. 3-4

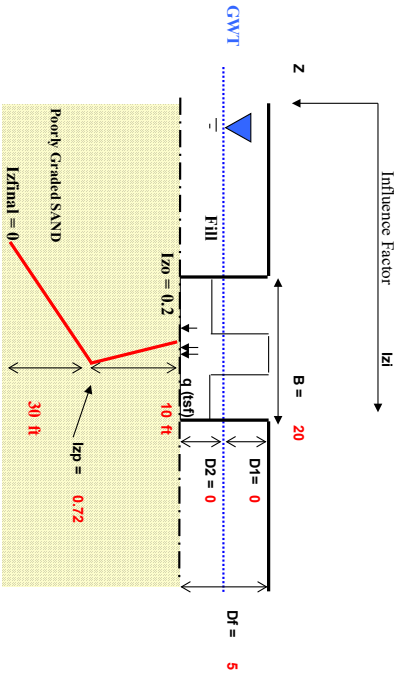
E_{si} = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

B = 20

Df = 5

GWT depth (ft) = 0



5400 psf

$$I_{zo} = 0.10$$
$$I_{zp} = 0.72$$
$$I_{zp}I_{zo} = 0.62$$
$$Z_{max(0)} = 40$$
$$Z_{ap(0)} = 10$$
$$Z_{max} - Z_{ap} = 30$$

z, ft	z/B	I _{zi}	E _{si} , tsf	I _{za}	I _{za} E _{si}
0.00	0.00	0.10			
2.00	0.10	0.22	610	0.162	0.0003
4.00	0.20	0.35	610	0.286	0.0005
6.00	0.30	0.47	610	0.410	0.0007
8.00	0.40	0.60	610	0.534	0.0009
10.00	0.50	0.72	610	0.658	0.0011
12.00	0.60	0.67	610	0.696	0.0011
14.00	0.70	0.62	610	0.648	0.0011
16.00	0.80	0.58	610	0.600	0.0010
18.00	0.90	0.53	610	0.552	0.0009
20.00	1.00	0.48	610	0.504	0.0008
22.00	1.10	0.43	610	0.456	0.0007
24.00	1.20	0.38	610	0.408	0.0007
26.00	1.30	0.34	610	0.360	0.0006
28.00	1.40	0.29	610	0.312	0.0005
30.00	1.50	0.24	610	0.264	0.0004
32.00	1.60	0.19	610	0.216	0.0004
34.00	1.70	0.14	2250	0.168	0.0001
36.00	1.80	0.10	2250	0.120	0.0001
38.00	1.90	0.05	2250	0.072	0.0000
40.00	2.00	0.00	2250	0.024	0.0000

Sand and Silt
Till
Glacial

$$\Sigma I_{zi}/E_{si} = 0.01175$$

1) $I_{zp} = \text{peak influence factor of soil layer 1, Fig. 3-4} = 0.5 + 0.11 \Delta p / \sigma'_{avg} \sqrt{1^2} = 0.720$

where, γ' (pcf) = effective unit weight
 0.035
 0.06

Using $\gamma' = \gamma \text{ sat} - \gamma \text{ water}$ (If soil is below GWT)
Using $\gamma' = \gamma \text{ moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing
 $\sigma'_{avg} = 0.5 \Delta B \times \gamma' + D \times \gamma'$ 0.522

-effective overburden pressure at depth I_{zp}, tsf

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf) . After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

Soil Type	N (blows/ft) =	$L/B = 1$
s, sandy silts, slightly cohesive silt-sand mixt	100	10N = 1000
Clean, fine to medium sands and slightly silty sands		17.5N = 1750
Coarse sands and sands with little gravel		25N = 2500
Sandy gravel and gravel		30N = 3000
		E _{si} (ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$P_i = C_i \cdot C_1 \cdot A_p \cdot \Delta z_i \cdot \sum \frac{I_z I_i}{E_{s_i}}$$

Short Term

Long Term

0.0570 ft

0.0799 ft

0.68 inches <1

0.96 inches <1

OK

OK

where,

C_1 = correction to account for strain relief from embedment, $1-0.5\sigma'_{at}/\Delta p \geq 0.5$

σ'_{at} = effective vertical overburden pressure at bottom of footing or depth D , tsf

γ_{sat} = unit weight (pcf)

120

132.00

0.96

γ_{sat} = unit weight (pcf)

σ'_{at} = effective vertical overburden pressure at bottom of footing or depth D , tsf

γ_{sat} = unit weight (pcf)

120

132.00

0.17 = γ D

C_i = correction for time dependent increase in settlement, $1+0.2 \log_{10}(t / 0.1)$

t = time, years (end of construction)

Δp = net applied footing pressure, $q-\sigma'_{at}$, tsf

0.1

10 (long term)

1.0 (short-term), 1.4 (long-term)

Δz_i = depth increment i , 0.1B, ft

I_z = influence factor of soil layer i , fig. 3-4

E_{s_i} = elastic modulus of soil layer i , tsf

2.28

2.5

481.25

q = applied pressure at foundation, tsf

B = 25

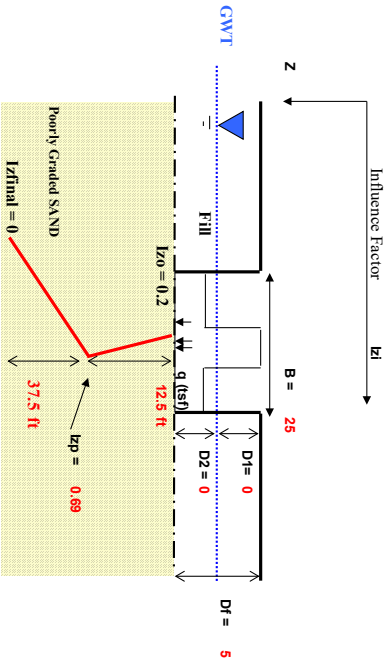
D_f = 5

4900 psf

2.45

2.45

GWT depth (ft) = 0



$l_{zo} = 0.10$
 $l_{zp} = 0.69$
 $l_{zp}-l_{zo} = 0.59$

$Z_{\text{final}}(t) = 50$
 $Z_{\text{sp}}(t) = 12.5$
 $Z_{\text{final}} - Z_{\text{sp}} = 37.5$

z, ft	z/B	l_{zi}	E_{si}, tsf	l_{za}	l_{za}/E_{si}	
0.00	0.00	0.10				Sand and Silt
2.50	0.10	0.22	610	0.159	0.0003	
5.00	0.20	0.34	610	0.278	0.0005	
7.50	0.30	0.46	610	0.397	0.0007	
10.00	0.40	0.57	610	0.515	0.0008	
12.50	0.50	0.69	610	0.634	0.0010	
15.00	0.60	0.65	610	0.670	0.0011	
17.50	0.70	0.60	610	0.624	0.0010	
20.00	0.80	0.55	610	0.578	0.0009	
22.50	0.90	0.51	610	0.532	0.0009	
25.00	1.00	0.46	610	0.485	0.0008	Glacial Till
27.50	1.10	0.42	610	0.439	0.0007	
30.00	1.20	0.37	610	0.393	0.0006	
32.50	1.30	0.32	610	0.347	0.0006	
35.00	1.40	0.28	2250	0.300	0.0001	
37.50	1.50	0.23	2250	0.254	0.0001	
40.00	1.60	0.18	2250	0.208	0.0001	
42.50	1.70	0.14	2250	0.162	0.0001	
45.00	1.80	0.09	2250	0.116	0.0001	
47.50	1.90	0.05	2250	0.069	0.0000	
50.00	2.00	0.00	2250	0.023	0.0000	

$\Sigma l_{zi}/E_{si} = 0.01042$

1) l_{zp} = peak influence factor of soil layer i, Fig. 3-4 = $0.5+0.1(\Delta p/\sigma'_{vm})^{1/2} = 0.693$

where, $\gamma' (cf)$ = effective unit weig U sing $\gamma' = \gamma \text{ sat} - \gamma \text{ water}$ (If soil is below GWT)
Using $\gamma' = \gamma \text{ moist}$ (If soil is above GWT)

Axisymmetric ($L/B=1$) - Spread/column footing -effective overburden pressure at depth l_{zp} , tsf
 $\sigma'_{vm} = 0.5\alpha B \times \gamma' + D \times \gamma' \quad 0.609$

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf) .After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978).

N (blows/ft) = 100 $L/B = 1$

Soil Type $10N = 1000$
s, sandy silts, slightly cohesive silt-sand mixt
 $17.5N = 1750$
Clean, fine to medium sands and slightly silty sands $25N = 2500$
Coarse sands and sands with little gravel $30N = 3000$
Sandy gravel and gravel $E_{si} \text{ (ksf)} = 962.5$

Settlement - Schmertmann Approximation for Spread Footing

**Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.*

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.0560 ft	0.67 inches <1	OK
Long Term	0.0783 ft	0.94 inches <1	OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5 \sigma'_{od} / \Delta p \geq 0.5$ 0.94

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf 0.17 = $\gamma' D$

where, γ_{moist} unit weight (pcf) 120

γ_{sat} unit weight (pcf) = 131.00

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ 1.0 (short-term) 1.4 (long-term)

t = time, years (end of constructic 0.1 10 (long term)

Δp = net applied footing pressure, $q - \sigma'_{od}$, tsf 1.33

Δz_i = depth increment i, 0.1B, ft 6.5

I_{zi} = influence factor of soil layer i, fig. 3-4

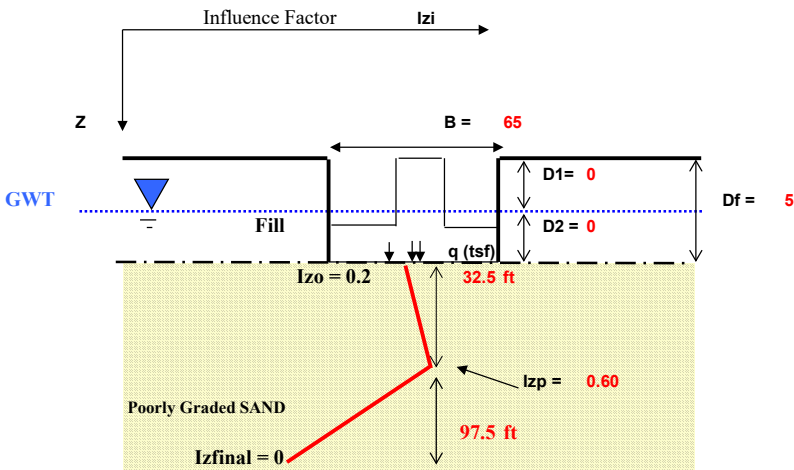
E_{si} = elastic modulus of soil layer i, tsf 481.25

q = applied pressure at foundation, tsf 1.50 3000

B = 65

Df = 5

GWT depth (ft) = 0



$$I_{zo} = 0.10$$

$$I_{zp} = 0.60$$

$$I_{zp} - I_{zo} = 0.50$$

$$Z_{final (ft)} = 130$$

$$Z_{zp (ft)} = 32.5$$

$$Z_{final} - Z_{zp} = 97.5$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Sand and Silt
6.50	0.10	0.20	350	0.150	0.0004	
13.00	0.20	0.30	350	0.250	0.0007	
19.50	0.30	0.40	350	0.351	0.0010	
26.00	0.40	0.50	350	0.451	0.0013	
32.50	0.50	0.60	350	0.551	0.0016	Gravel Till Weather
39.00	0.60	0.56	2250	0.582	0.0003	
45.50	0.70	0.52	2250	0.541	0.0002	
52.00	0.80	0.48	2250	0.501	0.0002	
58.50	0.90	0.44	2250	0.461	0.0002	
65.00	1.00	0.40	2250	0.421	0.0002	Bedrock
71.50	1.10	0.36	2500	0.381	0.0002	
78.00	1.20	0.32	2500	0.341	0.0001	
84.50	1.30	0.28	2500	0.301	0.0001	
91.00	1.40	0.24	2500	0.261	0.0001	
97.50	1.50	0.20	2500	0.221	0.0001	
104.00	1.60	0.16	2500	0.180	0.0001	
110.50	1.70	0.12	2500	0.140	0.0001	
117.00	1.80	0.08	2500	0.100	0.0000	
123.50	1.90	0.04	2500	0.060	0.0000	
130.00	2.00	0.00	2500	0.020	0.0000	

$$\sum Izi/E_{si} = 0.00693$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1 |\Delta p / \sigma'_{izp}|^{1/2} = 0.602$

where, γ' (pcf) = effective unit weight 0.034 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{izp} = 0.5 \times B \times \gamma' + D \times \gamma' = 1.286$

2) **Elastic Modulus, Esi (ksf)**

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N \text{ (blows/ft)} = 100$$

Soil Type

ts, sandy silts, slightly cohesive silt-sand mixtu
 Clean, fine to medium sands and slightly silty sands
 Coarse sands and sands with little gravel
 Sandy gravel and gravel

L/B = 1

10N = 1000
 17.5N = 1750
 25N = 2500
 30N = 3000

$$Esi \text{ (Ksf)} = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

**Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.*

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.0570 ft	0.68 inches <1	OK
Long Term	0.0798 ft	0.96 inches <1	OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5 \sigma'_{od} / \Delta p \geq 0.5$ 0.96

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf 0.17 = $\gamma' D$

where, γ_{moist} unit weight (pcf) 120

γ_{sat} unit weight (pcf) = 132.00

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ 1.0 (short-term) 1.4 (long-term)

t = time, years (end of constructic 0.1 10 (long term)

Δp = net applied footing pressure, q - σ'_{od} , tsf 2.03

Δz_i = depth increment i, 0.1B, ft 6.0

I_{zi} = influence factor of soil layer i, fig. 3-4

E_{si} = elastic modulus of soil layer i, tsf 481.25

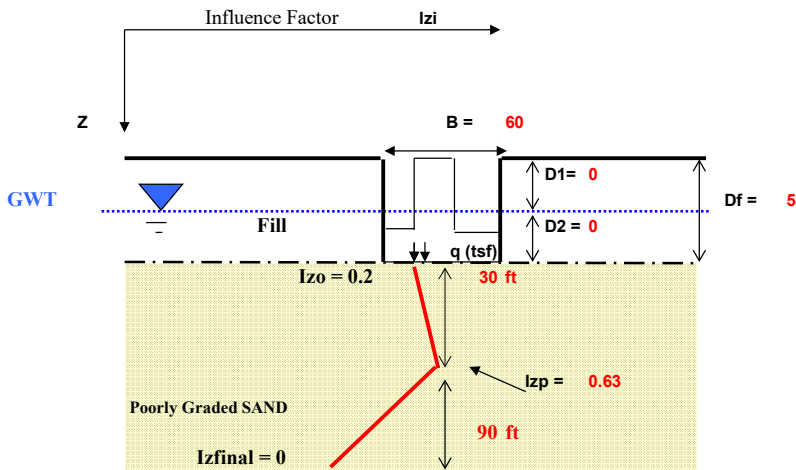
q = applied pressure at foundation, tsf 2.20

B = 60

Df = 5

GWT depth (ft) = 0

4400 psf



$$I_{zo} = 0.10$$

$$I_{zp} = 0.63$$

$$I_{zp} - I_{zo} = 0.53$$

$$Z_{final} (ft) = 120$$

$$Z_{zp} (ft) = 30$$

$$Z_{final} - Z_{zp} = 90$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi
0.00	0.00	0.10			
6.00	0.10	0.21	610	0.153	0.0003
12.00	0.20	0.31	610	0.259	0.0004
18.00	0.30	0.42	610	0.364	0.0006
24.00	0.40	0.52	610	0.470	0.0008
30.00	0.50	0.63	610	0.576	0.0009
36.00	0.60	0.59	2250	0.608	0.0003
42.00	0.70	0.55	2250	0.566	0.0003
48.00	0.80	0.50	2250	0.524	0.0002
54.00	0.90	0.46	2250	0.482	0.0002
60.00	1.00	0.42	2250	0.440	0.0002
66.00	1.10	0.38	2250	0.398	0.0002
72.00	1.20	0.34	3000	0.356	0.0001
78.00	1.30	0.29	3000	0.314	0.0001
84.00	1.40	0.25	3000	0.273	0.0001
90.00	1.50	0.21	3000	0.231	0.0001
96.00	1.60	0.17	3000	0.189	0.0001
102.00	1.70	0.13	3000	0.147	0.0000
108.00	1.80	0.08	3000	0.105	0.0000
114.00	1.90	0.04	3000	0.063	0.0000
120.00	2.00	0.00	2500	0.021	0.0000

$$\sum Izi/E_{si} = 0.0049$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1[\Delta p/\sigma'_{izp}]^{1/2} = 0.629$

where, γ' (pcf) = effective unit weight 0.035 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing -effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{izp} = 0.5 \times B \times \gamma' + D \times \gamma' \quad 1.218$

2) **Elastic Modulus, Esi (ksf)**

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N \text{ (blows/ft)} = 100$$

Soil Type

ts, sandy silts, slightly cohesive silt-sand mixtu
 Clean, fine to medium sands and slightly silty sands
 Coarse sands and sands with little gravel
 Sandy gravel and gravel

L/B = 1

10N = 1000
 17.5N = 1750
 25N = 2500
 30N = 3000

$$Esi \text{ (Ksf)} = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

**Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.*

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term

0.0571 ft

0.69 inches <1

OK

Long Term

0.0800 ft

0.96 inches <1

OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5 \sigma'_{od} / \Delta p \geq 0.5$

0.95

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf

$0.17 = \gamma' D$

where, γ_{moist} unit weight (pcf)

120

γ_{sat} unit weight (pcf) =

131.00

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$

1.0 (short-term)

1.4 (long-term)

t = time, years (end of constructio

0.1

10 (long term)

Δp = net applied footing pressure, q - σ'_{od} , tsf

1.58

Δz_i = depth increment i, 0.1B, ft

2.0

I_{zi} = influence factor of soil layer i, fig. 3-4

E_{si} = elastic modulus of soil layer i, tsf

481.25

q = applied pressure at foundation, tsf

1.75

3500

B =

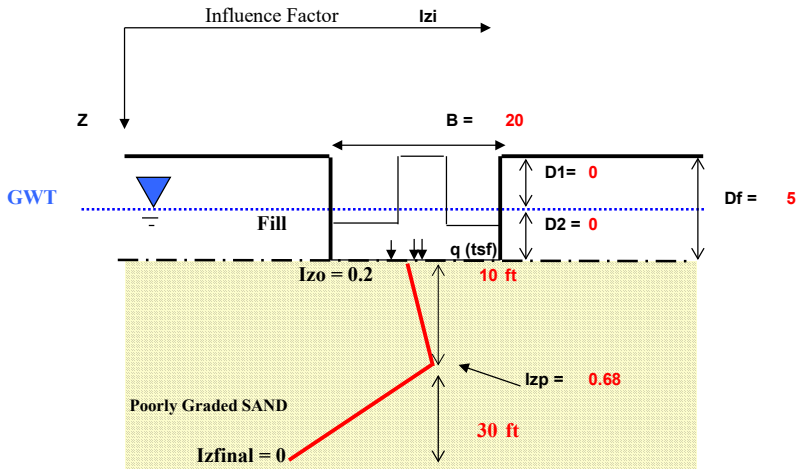
20

Df =

5

GWT depth (ft) =

0



$$I_{zo} = 0.10$$

$$I_{zp} = 0.68$$

$$I_{zp} - I_{zo} = 0.58$$

$$Z_{final (ft)} = 40$$

$$Z_{zp (ft)} = 10$$

$$Z_{final} - Z_{zp} = 30$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Sand and Silt
2.00	0.10	0.22	350	0.158	0.0005	
4.00	0.20	0.33	350	0.273	0.0008	
6.00	0.30	0.45	350	0.388	0.0011	
8.00	0.40	0.56	350	0.503	0.0014	
10.00	0.50	0.68	350	0.618	0.0018	
12.00	0.60	0.63	350	0.653	0.0019	
14.00	0.70	0.59	350	0.608	0.0017	
16.00	0.80	0.54	350	0.563	0.0016	
18.00	0.90	0.50	350	0.518	0.0015	
20.00	1.00	0.45	350	0.473	0.0014	
22.00	1.10	0.41	350	0.428	0.0012	
24.00	1.20	0.36	350	0.383	0.0011	
26.00	1.30	0.32	350	0.338	0.0010	
28.00	1.40	0.27	350	0.293	0.0008	
30.00	1.50	0.23	350	0.248	0.0007	Glacial Till
32.00	1.60	0.18	350	0.203	0.0006	
34.00	1.70	0.14	2250	0.158	0.0001	
36.00	1.80	0.09	2250	0.113	0.0001	
38.00	1.90	0.05	2250	0.068	0.0000	
40.00	2.00	0.00	2250	0.023	0.0000	

$$\sum Izi/E_{si} = 0.01914$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1 |\Delta p / \sigma'_{izp}|^{1/2} = 0.675$

where, γ' (pcf) = effective unit weight 0.034 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{izp} = 0.5 \times B \times \gamma' + D \times \gamma' = 0.515$

2) **Elastic Modulus, Esi (ksf)**

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N \text{ (blows/ft)} = 61$$

Soil Type

ts, sandy silts, slightly cohesive silt-sand mixtu
 Clean, fine to medium sands and slightly silty sands
 Coarse sands and sands with little gravel
 Sandy gravel and gravel

L/B = 1

10N = 610
 17.5N = 1067.5
 25N = 1525
 30N = 1830

$$Esi \text{ (Ksf)} = 962.5$$

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$p_i = C_1 * C_2 * \Delta p * A_z \sum \frac{I_z I_{z_i}}{E_{si}}$$

	Short Term	0.0565 ft	0.68 inches	<1	OK
	Long Term	0.0791 ft	0.95 inches	<1	OK

where,

C₁ = correction to account for strain relief from embedment, 1-0.5σ'_{vd}/Δp>0.5

σ'_{vd} = effective vertical overburden pressure at bottom of footing or depth D, tsf

where,

γ_{moist} unit weight (pcf) = 120

γ_{sat} unit weight (pcf) = 131.00

C₂ = correction for time dependent increase in settlement, 1+0.2 log₁₀ (t / 0.1)

t = time, years (end of constructi

Δp = net applied footing pressure, q-σ'_{vd}, tsf

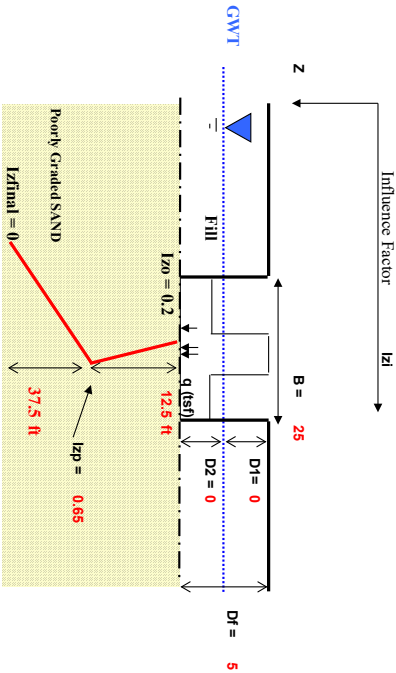
A_z = depth increment i, 0.1B, ft

I_z = influence factor of soil layer i, fig. 3-4

E_{si} = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

B = 25
Df = 5
GWT depth (ft) = 0
3200 psf



$l_{z0} = 0.10$
 $l_{zp} = 0.65$
 $l_{zp} - l_{z0} = 0.55$

$Z_{max(0)} = 50$
 $Z_{ap(0)} = 12.5$
 $Z_{max} - Z_{ap} = 37.5$

z, ft	z/B	l _z	E _s , tsf	l _{za}	l _{za} /E _s
0.00	0.00	0.10			
2.50	0.10	0.21	350	0.155	0.0004
5.00	0.20	0.32	350	0.266	0.0008
7.50	0.30	0.43	350	0.377	0.0011
10.00	0.40	0.54	350	0.488	0.0014
12.50	0.50	0.65	350	0.599	0.0017
15.00	0.60	0.61	350	0.632	0.0018
17.50	0.70	0.57	350	0.589	0.0017
20.00	0.80	0.52	350	0.545	0.0016
22.50	0.90	0.48	350	0.502	0.0014
25.00	1.00	0.44	350	0.458	0.0013
27.50	1.10	0.39	350	0.414	0.0012
30.00	1.20	0.35	350	0.371	0.0011
32.50	1.30	0.31	350	0.327	0.0009
35.00	1.40	0.26	2250	0.284	0.0001
37.50	1.50	0.22	2250	0.240	0.0001
40.00	1.60	0.17	2250	0.196	0.0001
42.50	1.70	0.13	2250	0.153	0.0001
45.00	1.80	0.09	2250	0.109	0.0000
47.50	1.90	0.04	2250	0.065	0.0000
50.00	2.00	0.00	2250	0.022	0.0000

Sand and Silt

Glacial Till

$\sum l_{zi}/E_{si} = 0.01683$

1) l_{zp} = peak influence factor of soil layer i, fig. 3-4 = $0.5 + 0.1 \Delta p / \sigma'_{vm}$ $1/2 = 0.654$

where, γ' (pcf) = effective unit weight
 0.034
 0.06

Using $\gamma' = \gamma$ sat - γ water (If soil is below GWT)
Using $\gamma' = \gamma$ moist (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing
 $\sigma'_{vm} = 0.5 \gamma B \times \gamma' + D \times \gamma'$ 0.600

-effective overburden pressure at depth l_{zp} , tsf

2) Elastic Modulus, Esi (ksf)

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

Soil Type	N (blows/ft) = 100	$L/B = 1$
s, sandy silts, slightly cohesive silt-sand mixtu		10N = 1000
Clean, fine to medium sands and slightly silty sands		17.5N = 1750
Coarse sands and sands with little gravel		25N = 2500
Sandy gravel and gravel		30N = 3000
		Esi (Ksf) = 962.5

Settlement - Schmertmann Approximation for Spread Footing (Case 2)

*Based on USACE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$P_i = C_1 \cdot C_2 \cdot A_p \cdot \Delta z_i \cdot \sum \frac{I_z I_i}{E_{si}}$$

Short Term

Long Term

0.0552 ft

0.0773 ft

0.66 inches <1

0.93 inches <1

OK

OK

where,

C₁ = correction to account for strain relief from embedment, 1-0.5σ'_{ad}/Δp>=0.5

σ'_{ad} = effective vertical overburden pressure at bottom of footing or depth D, tsf

where, γ_{soil} unit weight (pcf) = 120

γ_{sat} unit weight (pcf) = 131.00

C₂ = correction for time dependent increase in settlement, 1+0.2 log₁₀(t / 0.1)

t = time, years (end of constructi

Δp = net applied footing pressure, q-σ'_{ad}, tsf

Δz_i = depth increment i, 0.1B, ft

I_{zi} = influence factor of soil layer i, fig. 3-4

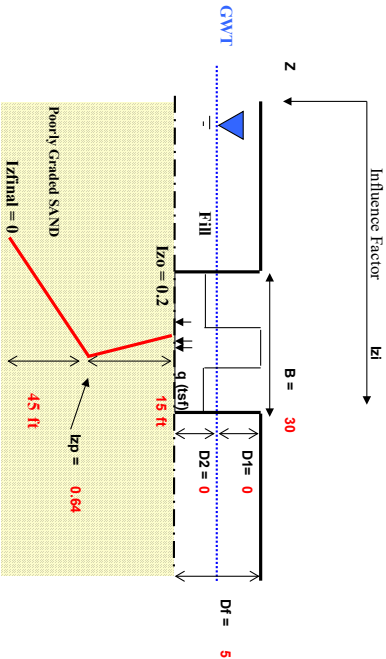
E_{si} = elastic modulus of soil layer i, tsf

q = applied pressure at foundation, tsf

B = 30

Df = 5

GWT depth (ft) = 0



$$I_{zo} = 0.10$$
$$I_{zp} = 0.64$$
$$I_{zp} - I_{zo} = 0.54$$
$$Z_{max}(ft) = 60$$
$$Z_{ap}(ft) = 15$$
$$Z_{max} - Z_{ap} = 45$$

z, ft	z/B	I_{zi}	E_{si}, tsf	I_{za}	I_{za}/E_{si}	
0.00	0.00	0.10				Sand and Silt
3.00	0.10	0.21	350	0.154	0.0004	
6.00	0.20	0.32	350	0.262	0.0007	
9.00	0.30	0.42	350	0.370	0.0011	
12.00	0.40	0.53	350	0.477	0.0014	
15.00	0.50	0.64	350	0.585	0.0017	
18.00	0.60	0.60	350	0.618	0.0018	
21.00	0.70	0.55	350	0.575	0.0016	
24.00	0.80	0.51	350	0.533	0.0015	
27.00	0.90	0.47	350	0.490	0.0014	
30.00	1.00	0.43	350	0.447	0.0013	Glacial Till
33.00	1.10	0.38	350	0.405	0.0012	
36.00	1.20	0.34	2250	0.362	0.0002	
39.00	1.30	0.30	2250	0.320	0.0001	
42.00	1.40	0.26	2250	0.277	0.0001	
45.00	1.50	0.21	2250	0.234	0.0001	
48.00	1.60	0.17	2250	0.192	0.0001	
51.00	1.70	0.13	2250	0.149	0.0001	
54.00	1.80	0.09	2250	0.107	0.0000	
57.00	1.90	0.04	2250	0.064	0.0000	
60.00	2.00	0.00	2250	0.021	0.0000	

$$\Sigma I_{zi}/E_{si} = 0.01481$$

1) I_{zp} = peak influence factor of soil layer i, Fig. 3-4 = $0.5 + 0.1 I_{ap} / \sigma'_{vm}$ $I_{zp} = 0.639$

where, $\gamma' (cf)$ = effective unit weig 0.034 Using $\gamma' = \gamma_{sat} - \gamma_{water}$ (If soil is below GWT)
 0.06 Using $\gamma' = \gamma_{moist}$ (If soil is above GWT)

Axisymmetric ($L/B=1$) - Spread/column footing $\sigma'_{vm} = 0.5 \times B \times \gamma' + D \times \gamma'$ 0.686 -effective overburden pressure at depth I_{zp} , tsf

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf) .After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978).

$$N \text{ (blows/ft)} = 100 \quad \frac{L/B = 1}{10N = 1000}$$

$$17.5N = 1750$$

$$25N = 2500$$

$$30N = 3000$$

$$E_{si} \text{ (ksf)} = 962.5$$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt

Clean, fine to medium sands and slightly silty sands

Coarse sands and sands with little gravel

Sandy gravel and gravel

Settlement - Schmertmann Approximation for Spread Footing

*Based on USACE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$P_i = C_i * C_i * \Delta p_i * \Delta z_i * \sum \frac{I_{zi}}{E_{si}}$$

Short Term

Long Term

0.0602 ft

0.0843 ft

0.72 inches <1

1.01 inches <1

OK

NOT OK

where,

C_i = correction to account for strain relief from embedment, $1-0.5\sigma'_{at}/\Delta p_i \geq 0.5$

σ'_{at} = effective vertical overburden pressure at bottom of footing or depth D_i , tsf

γ_{sat} unit weight (pcf)

120

where,

γ_{sat} unit weight (pcf) =

132.00

C_i = correction for time dependent increase in settlement, $1+0.2 \log_{10}(t / 0.1)$

t = time, years (end of construction)

0.1

10 (long term)

1.0 (short-term)

1.4 (long-term)

Δp_i = net applied footing pressure, q- σ'_{at} , tsf

0.1

Δz_i = depth increment i, 0.1B, ft

0.1B, ft

I_{zi} = influence factor of soil layer i, fig. 3-4

0.73

E_{si} = elastic modulus of soil layer i, tsf

5300

q = applied pressure at foundation, tsf

481.25

B =

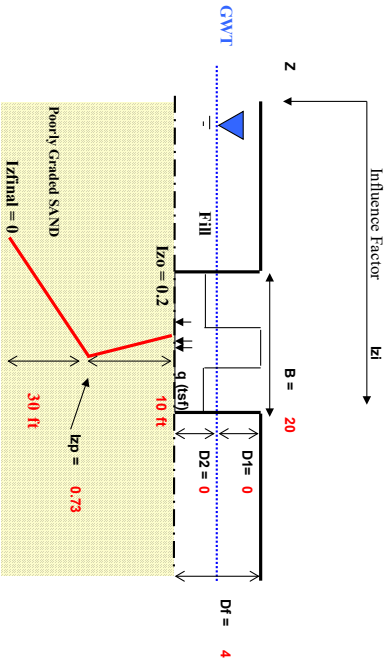
20

Df =

4

GWT depth (ft) =

0



$l_{zo} = 0.10$
 $l_{zp} = 0.73$
 $l_{zp}-l_{zo} = 0.63$

$Z_{final (t)} = 40$
 $Z_{sp (t)} = 10$
 $Z_{final} - Z_{sp} = 30$

z, ft	z/B	l_{zi}	E_{si}, tsf	l_{za}	l_{za}/E_{si}
0.00	0.00	0.10			
2.00	0.10	0.23	610	0.163	0.0003
4.00	0.20	0.35	610	0.288	0.0005
6.00	0.30	0.48	610	0.414	0.0007
8.00	0.40	0.60	610	0.539	0.0009
10.00	0.50	0.73	610	0.664	0.0011
12.00	0.60	0.68	610	0.703	0.0012
14.00	0.70	0.63	610	0.654	0.0011
16.00	0.80	0.58	610	0.606	0.0010
18.00	0.90	0.53	610	0.557	0.0009
20.00	1.00	0.48	610	0.509	0.0008
22.00	1.10	0.44	610	0.460	0.0008
24.00	1.20	0.39	610	0.412	0.0007
26.00	1.30	0.34	610	0.364	0.0006
28.00	1.40	0.29	610	0.315	0.0005
30.00	1.50	0.24	610	0.267	0.0004
32.00	1.60	0.19	610	0.218	0.0004
34.00	1.70	0.15	610	0.170	0.0003
36.00	1.80	0.10	610	0.121	0.0002
38.00	1.90	0.05	610	0.073	0.0001
40.00	2.00	0.00	610	0.024	0.0000

Sand and Silt

$\Sigma l_{zi}/E_{si} = 0.01233$

1) l_{zp} = peak influence factor of soil layer i, Fig. 3-4 = $0.5+0.1(\Delta p/\sigma'_{vm})^{1/2}$ = 0.727

where, γ' (pcf) = effective unit weig **0.035** Using $\gamma' = \gamma \text{ sat} - \gamma \text{ water}$ (If soil is below GWT)
0.06 Using $\gamma' = \gamma \text{ moist}$ (If soil is above GWT)

Axisymmetric ($L/B=1$) - Spread/column footing -effective overburden pressure at depth l_{zp} , tsf
 $\sigma'_{vm} = 0.5\alpha B \times \gamma' + D \times \gamma'$ **0.487**

2) Elastic Modulus, E_{si} (ksf)

Convert SPT (blows/ft) to E_{si} (ksf) .After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978).

N (blows/ft) = **61** $L/B = 1$

Soil Type

s, sandy silts, slightly cohesive silt-sand mixt **610**
Clean, fine to medium sands and slightly silty sands **1067.5**
Coarse sands and sands with little gravel **1525**
Sandy gravel and gravel **1830**

E_{si} (ksf) = **962.5**

TEMPORARY BRIDGE

Settlement - Schmertmann Approximation for Spread Footing

**Based on USACOE, EM 1110-1-1904 "Settlement analysis," 1990 and NAVFAC Design Manual 7.01 "Soil Mechanics," 1986.*

This procedure provides settlement compatible with field measurements in many different areas.

* Assumes the distribution of vertical strain is compatible with a linear elastic half space subjected to uniform pressure.

$$\rho_i = C_1 * C_t * \Delta p * \Delta z_i \sum I_{zi} / E_{si}$$

Short Term	0.1132 ft	1.36 inches	<1	NOT OK
Long Term	0.1427 ft	1.71 inches	<1	NOT OK

where,

C_1 = correction to account for strain relief from embedment, $1 - 0.5\sigma'_{od}/\Delta p \geq 0.5$ 0.97

σ'_{od} = effective vertical overburden pressure at bottom of footing or depth D, tsf 0.17 = $\gamma' D$

where, γ_{moist} unit weight (pcf) 120

γ_{sat} unit weight (pcf) = 130.00

C_t = correction for time dependent increase in settlement, $1 + 0.2 \log_{10}(t / 0.1)$ 1.0 (short-term 1.260206 (long-term)

t = time, years (end of constructi 0.1 2 (long term)

Δp = net applied footing pressure, q - σ'_{od} , tsf 2.73

Δz_i = depth increment i, 0.1B, ft 1.0

I_{zi} = influence factor of soil layer i, fig. 3-4

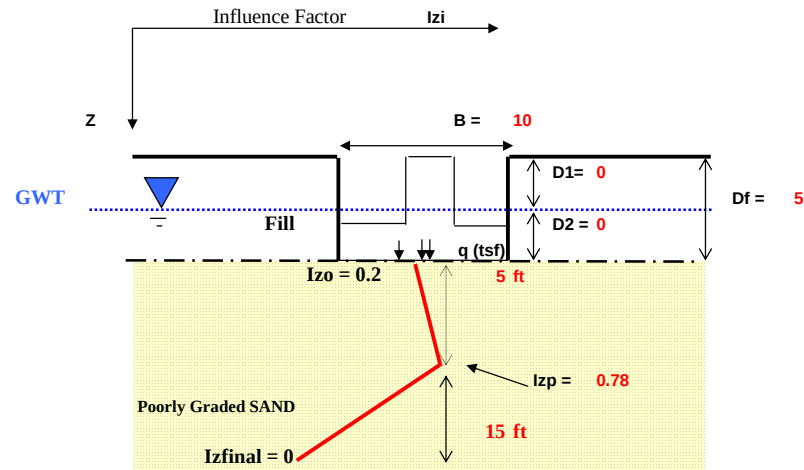
E_{si} = elastic modulus of soil layer i, tsf 481.25

q = applied pressure at foundation, tsf 2.90 5800 psf

B = 10

Df = 5

GWT depth (ft) = 0



$$I_{zo} = 0.10$$

$$I_{zp} = 0.78$$

$$I_{zp} - I_{zo} = 0.68$$

$$Z_{\text{final}} (\text{ft}) = 20$$

$$Z_{zp} (\text{ft}) = 5$$

$$Z_{\text{final}} - Z_{zp} = 15$$

z, ft	z/B	Izi	Esi, tsf	Iza	Iza/Esi	
0.00	0.00	0.10				Existing Fill
1.00	0.10	0.24	70	0.168	0.0024	
2.00	0.20	0.37	70	0.305	0.0044	
3.00	0.30	0.51	70	0.442	0.0063	
4.00	0.40	0.65	70	0.579	0.0083	
5.00	0.50	0.78	70	0.716	0.0102	Sand and Gravel
6.00	0.60	0.73	525	0.758	0.0014	
7.00	0.70	0.68	525	0.706	0.0013	
8.00	0.80	0.63	525	0.654	0.0012	
9.00	0.90	0.58	525	0.601	0.0011	
10.00	1.00	0.52	525	0.549	0.0010	
11.00	1.10	0.47	525	0.497	0.0009	
12.00	1.20	0.42	525	0.444	0.0008	
13.00	1.30	0.37	525	0.392	0.0007	
14.00	1.40	0.31	525	0.340	0.0006	
15.00	1.50	0.26	525	0.288	0.0005	
16.00	1.60	0.21	525	0.235	0.0004	
17.00	1.70	0.16	525	0.183	0.0003	
18.00	1.80	0.10	525	0.131	0.0002	
19.00	1.90	0.05	525	0.078	0.0001	
20.00	2.00	0.00	525	0.026	0.0000	

$$\sum Izi/E_{si} = 0.04278$$

1) $I_{zp} = \text{peak influence factor of soil layer i, fig. 3-4} = 0.5 + 0.1[\Delta p/\sigma'_{zp}]^{1/2} = 0.784$

where, γ' (pcf) = effective unit weight 0.034 Using $\gamma' = \gamma_{\text{sat}} - \gamma_{\text{water}}$ (If soil is below GWT)
 0.06 Using $\gamma' = \gamma_{\text{moist}}$ (If soil is above GWT)

Axisymmetric (L/B=1) - Spread/column footing - effective overburden pressure at depth I_{zp} , tsf
 $\sigma'_{zp} = 0.5 \times B \times \gamma' + D \times \gamma' = 0.338$

2) **Elastic Modulus, Esi (ksf)**

Convert SPT (blows/ft) to Esi (ksf) , After Schmertmann (1970) and Schmertmann, Hartman, and Brown (1978)

$$N (\text{blows/ft}) = 30$$

Soil Type

ts, sandy silts, slightly cohesive silt-sand mixture
 Clean, fine to medium sands and slightly silty sands
 Coarse sands and sands with little gravel
 Sandy gravel and gravel

L/B = 1

$$10N = 300$$

$$17.5N = 525$$

$$25N = 750$$

$$30N = 900$$

$$Esi (\text{Ksf}) = 962.5$$

Cranberry Highway/Mill Pond Dam over the Agawam River

Secant Walls



Calculation Cover Sheet

Project #: 22147.02 **Calculation #:** 1
Project: Mill Pond Dam Improvements **Date:** See Below
Subject/Task: Seepage and Slope Stability Analysis
Calculated by: OB **Reviewed by:** DRC

Revision Summary:

Revision #	Description	Date
0	Original Calculation	12/2023

Subject:

This calculation was performed to determine the seepage and slope stability of the Mill Pond Dam embankment during maximum pool, normal pool, drawdown, and earthquake conditions.

Information Provided/Used in the analysis:

1. Soil properties from the Mill Pond Dam Soil Parameters Table prepared by Pare Corporation in December 2021 and modified in January 2025.
2. Hydrologic and Hydraulic Data from HydroCAD model prepared by Pare Corporation in December 2021 and modified in January 2025.
3. Geometry of Mill Pond Dam from 22147.02 Permit Drawings prepared by Pare Corporation dated December 2025.
4. GeoStudio 2024.1.0

References:

1. 22147.02 Soil Parameters Table, Mill Pond Dam, Prepared by Pare Corporation in December 2021.
2. "U.S. Seismic Design Maps." USGS Earthquakes Hazard Program, accessed December, 2021 <http://earthquake.usgs.gov/designmaps>.

1.0 – MODEL INPUTS

Methodology:

Dam Embankment Geometry:

Critical Section: The critical sections were taken left and right of the primary spillway. These areas were assumed to be the critical section for the following reasons: Left of the primary spillway contains the section of the proposed upstream slope with the highest elevation difference and slope gradient. The same is true for the proposed downstream slope on the right side of the primary spillway.

Basis of geometry and soil parameters are based on Pare's Phase II Report dated April 2009 and the Permit Drawings December 2024. Decisive for the current design is a secant cut-off wall extending into less permeable soil strata in the foundation materials, therefore reducing seepage and increasing slope stability. The proposed impervious wall is required for phasing during construction and has been incorporated into the final design. The reinforced secant wall reduces risk of critical failure beyond the analysis herein, whereas the modelled scenarios are investigating section loss during normal pool, draw down from normal pool, design seismic event during normal pool, as well as maximum design flood.

Existing conditions are investigated in Phase II Inspection / Evaluation Report April 2009 and not included within the reviewed scenarios.

Soil Parameters:

Soil parameters were copied into GeoStudio from Pare's soil parameters table.

Boundary Conditions:

Pare defined the downstream slopes and areas as potential seepage faces in GeoStudio. Normal pool and max pool was set per Pare's HydroCAD model. For the proposed conditions model, downstream water was modeled at the water level within the Agawam River immediately downstream of the dam. The drawdown time from normal pool elevation was calculated using Pare's HydroCAD model.

Slope Stability Settings:

Slip surfaces were identified by GeoStudio using a Morgenstern-Price analysis with the "Grid and Radius" option selected.

Seismic:

Seismic parameters were sourced from the U.S. Seismic Design Maps USGS Earthquakes Hazard Program.

1.0 – MODEL RESULTS

See the attached GeoStudio Results Table

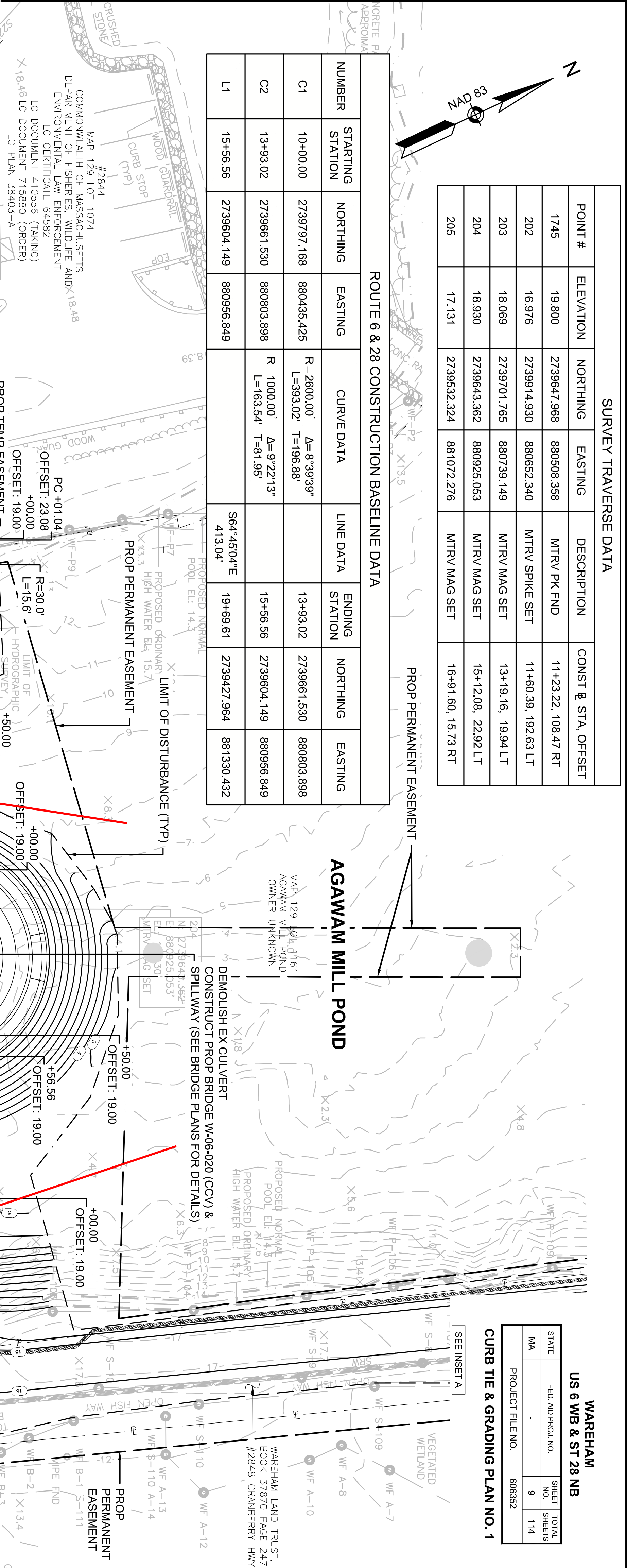
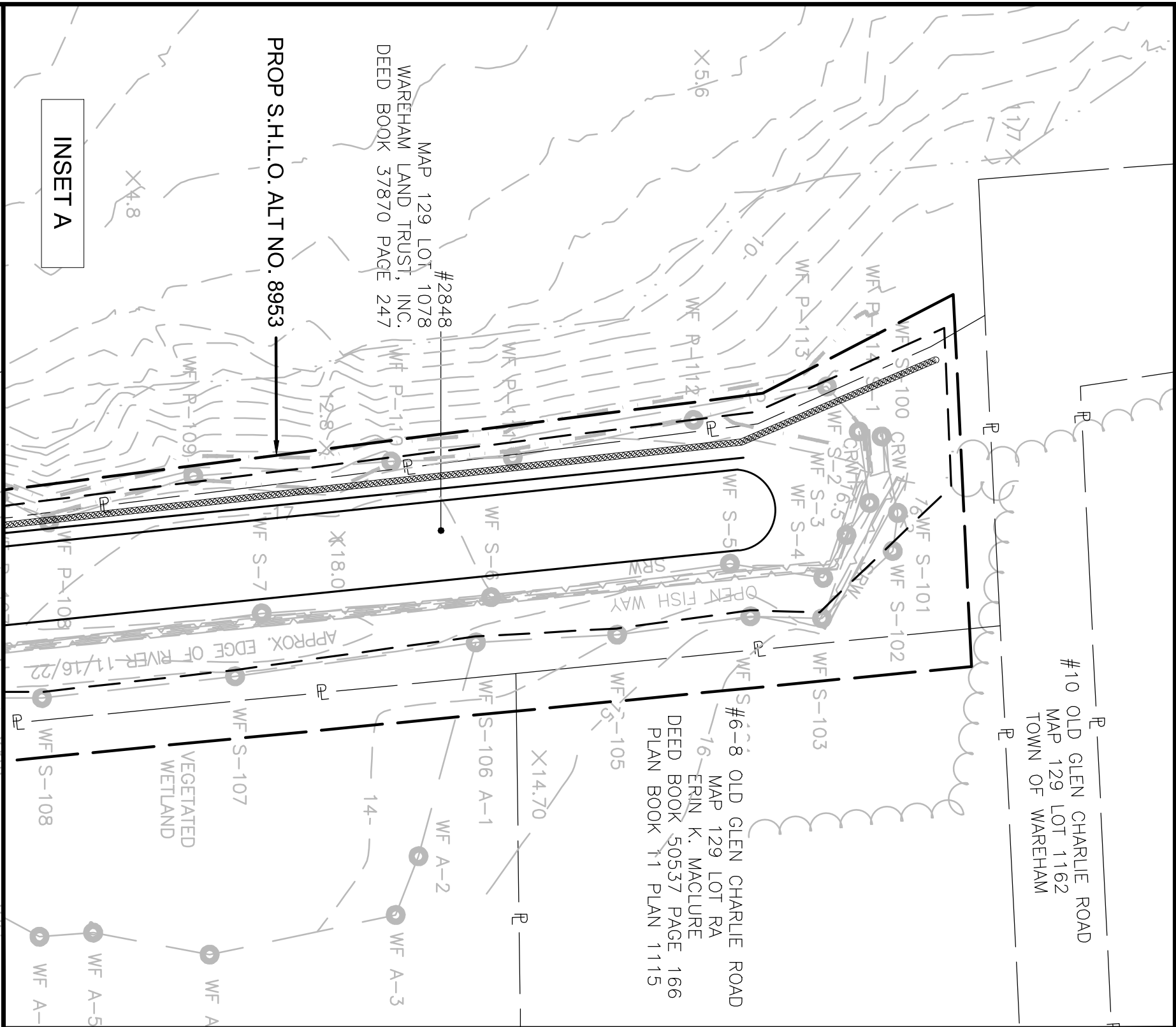
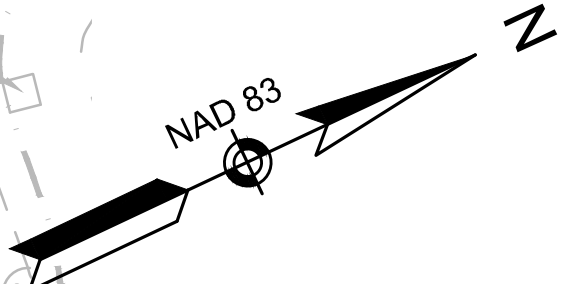
Calculation by:	Orell Baumann	Engineer II		4/28/2025
	Name	Position	Signature	Date
Checked by:	David Caouette	Managing Engineer		4/28/2025
	Name	Position	Signature	Date

SURVEY TRAVERSE DATA					
POINT #	ELEVATION	NORTHING	EASTING	DESCRIPTION	CONST & STA. OFFSET
1745	19.800	2739647.968	880508.358	MTRY PK FND	11+23.22, 108.47 RT
202	16.976	2739914.930	880652.340	MTRY SPIKE SET	11+60.39, 192.63 LT
203	18.069	2739701.765	880739.149	MTRY MAG SET	13+19.16, 19.94 LT
204	18.930	2739643.362	880925.053	MTRY MAG SET	15+12.08, 22.92 LT
205	17.131	2739532.324	881072.276	MTRY MAG SET	16+91.60, 15.73 RT

ROUTE 6 & 28 CONSTRUCTION BASELINE DATA					
NUMBER	STARTING STATION	NORTHING	EASTING	CURVE DATA	LINE DATA
C1	10+00.00	2739797.168	880435.425	R=2600.00 Δ=8°39'39" L=393.02° T=196.88'	13+93.02 2739661.530 880803.898
C2	13+93.02	2739661.530	880803.898	R=1000.00 Δ=9°22'13" L=163.54° T=81.95'	15+56.56 2739604.149 880956.849
L1	15+56.56	2739604.149	880956.849	S64°45'04"E 413.64'	19+69.61 2739427.964 881330.432

WAREHAM US 6 WB & ST 28 NB			
STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
MA	-	9	114
PROJECT FILE NO. 606352			

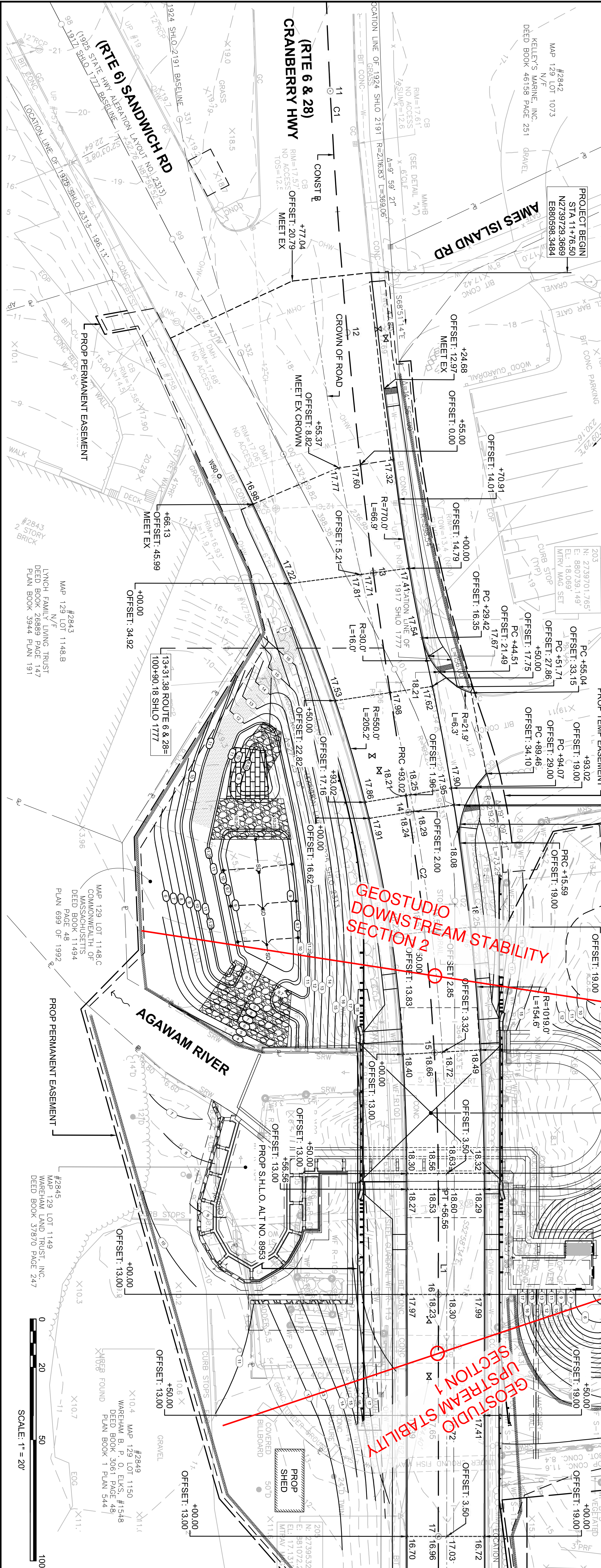
CURB TIE & GRADING PLAN NO. 1	
SEE INSET A	



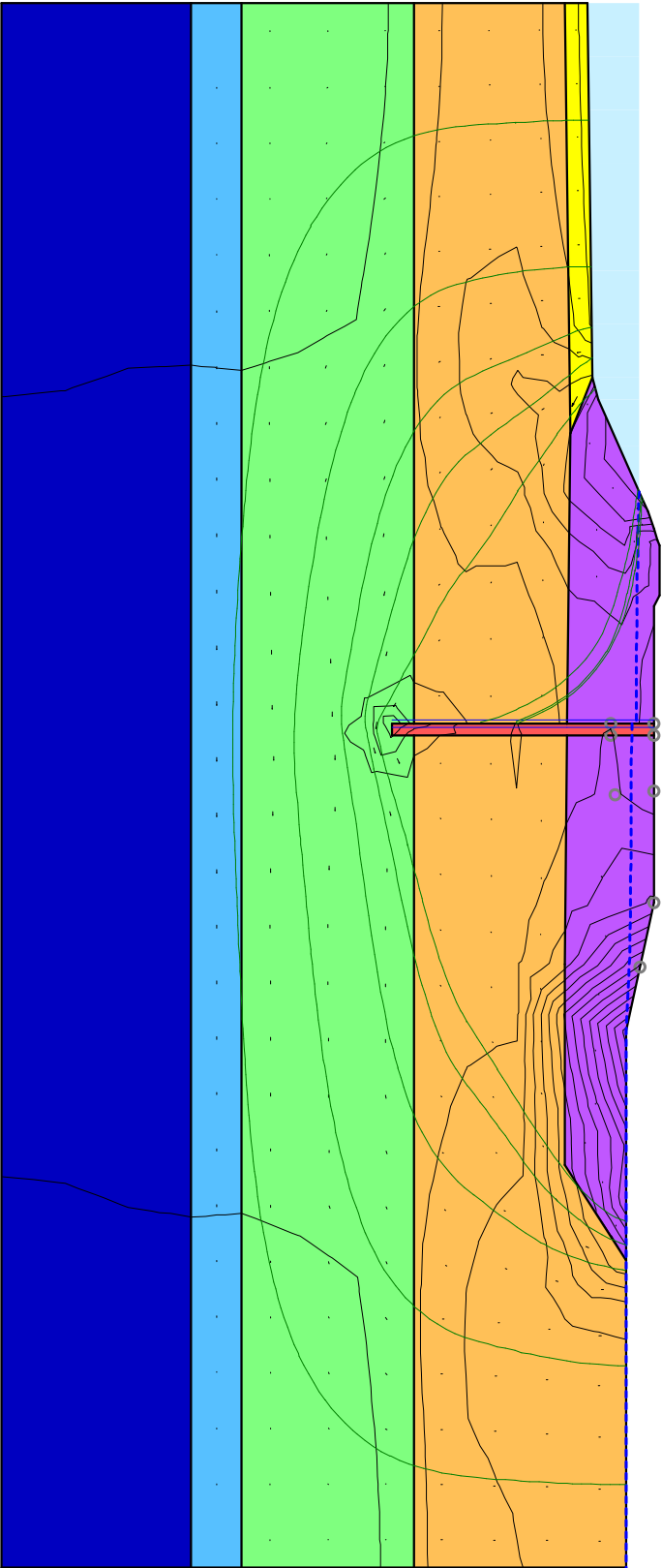
GEOSTUDIO
DOWNSTREAM STABILITY
SECTION 2

GEOSTUDIO
UPSTREAM STABILITY
SECTION 1

CONTINUED ON
SHEET NO. 10



Color	Name	Hydraulic Material Model	Sat Kx (ft/sec)	Ky/Kx Ratio	Volumetric Water Content	Compressibility (psi)
Blue	BEDROCK	Saturated Only	1e-09	1	0	0
Purple	FILL	Saturated / Unsaturated		0.25		
Light Green	GLACIAL TILL	Saturated / Unsaturated		1		
Yellow	POND MUCK	Saturated / Unsaturated		1		
Orange	SAND & SILT	Saturated / Unsaturated		1		
Red	SECANT PILE	Saturated / Unsaturated		1		
Light Blue	WEATHERED BEDROCK	Saturated / Unsaturated		1		

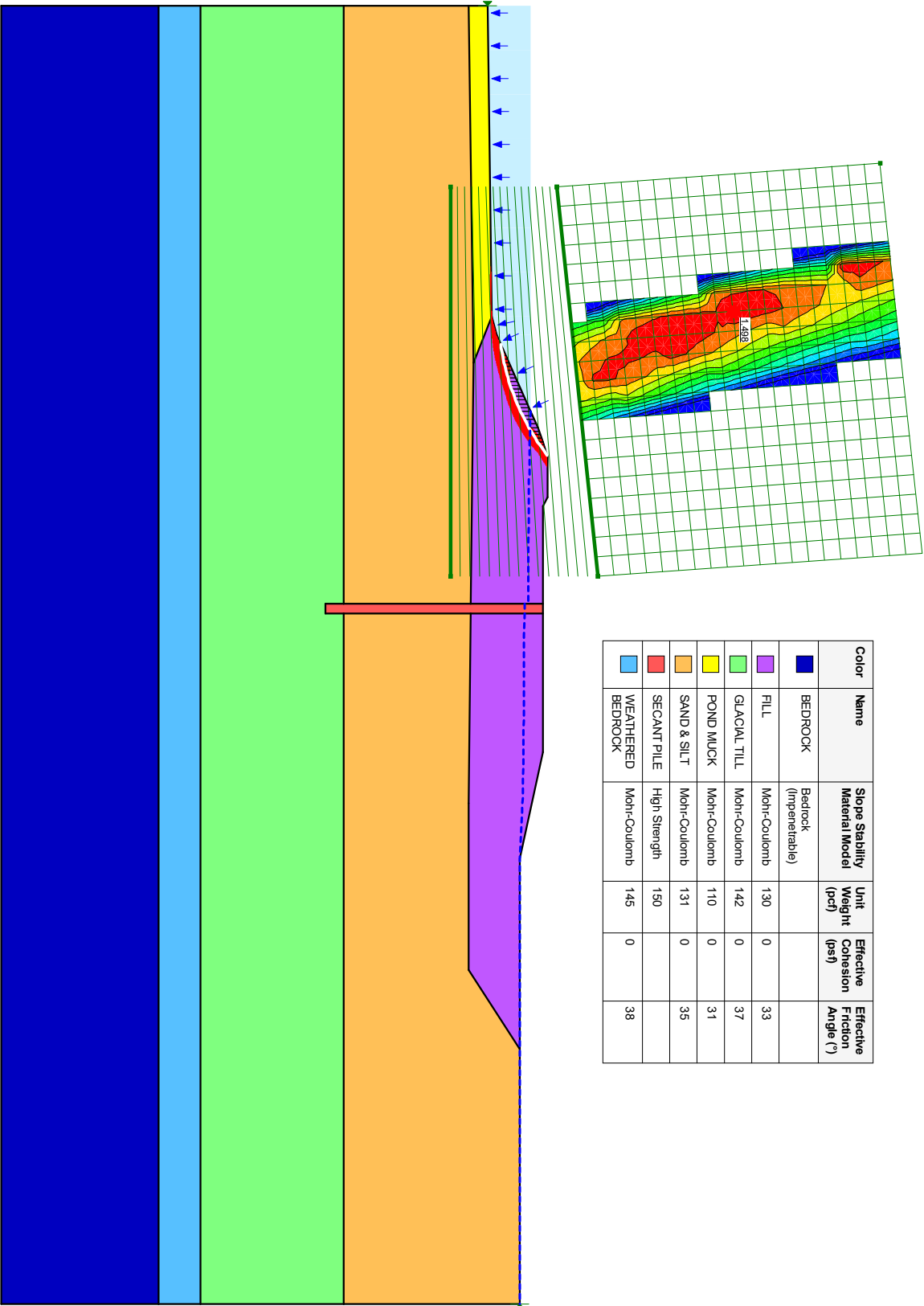


Section 1 - Normal Pool

MassDOT Wareham Mill Pond Dam - Slope Stability & Seepage.gsz

02/07/2025

1:400

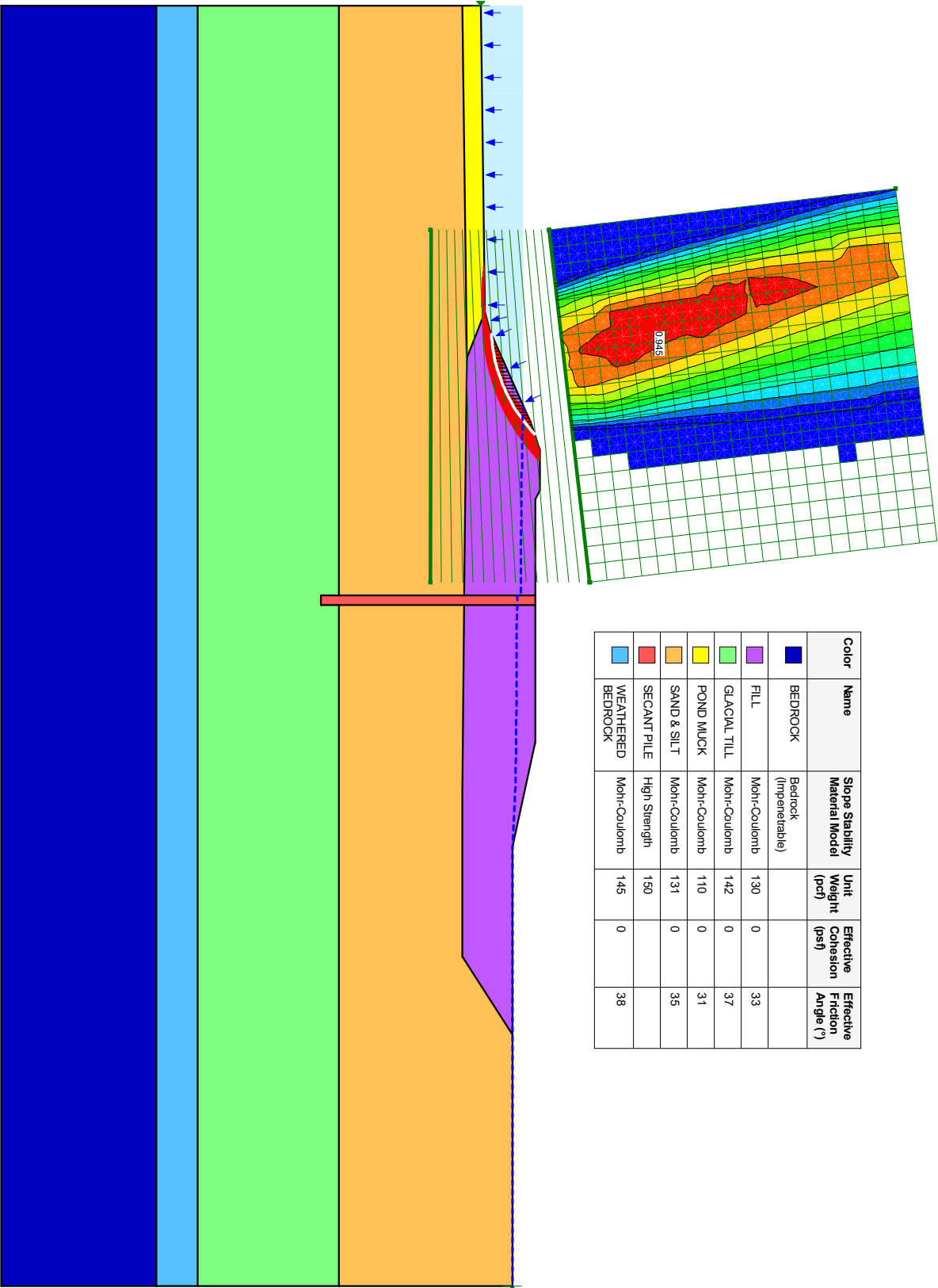


Location: Section 1 - Left of Primary Spillway
Factor of Safety: 1.498

Normal Pool - Upstream Slope
MassDOT Wareham Mill Pond Dam - Slope Stability & Seepage.gsz

02/07/2025

1:400



Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Blue	BEDROCK	Bedrock (Impenetrable)			
Purple	FILL	Mohr-Coulomb	130	0	33
Green	GLACIAL TILL	Mohr-Coulomb	142	0	37
Yellow	POND MUCK	Mohr-Coulomb	110	0	31
Orange	SAND & SILT	Mohr-Coulomb	131	0	35
Red	SECANT PILE	High Strength	150		
Light Blue	WEATHERED BEDROCK	Mohr-Coulomb	145	0	38

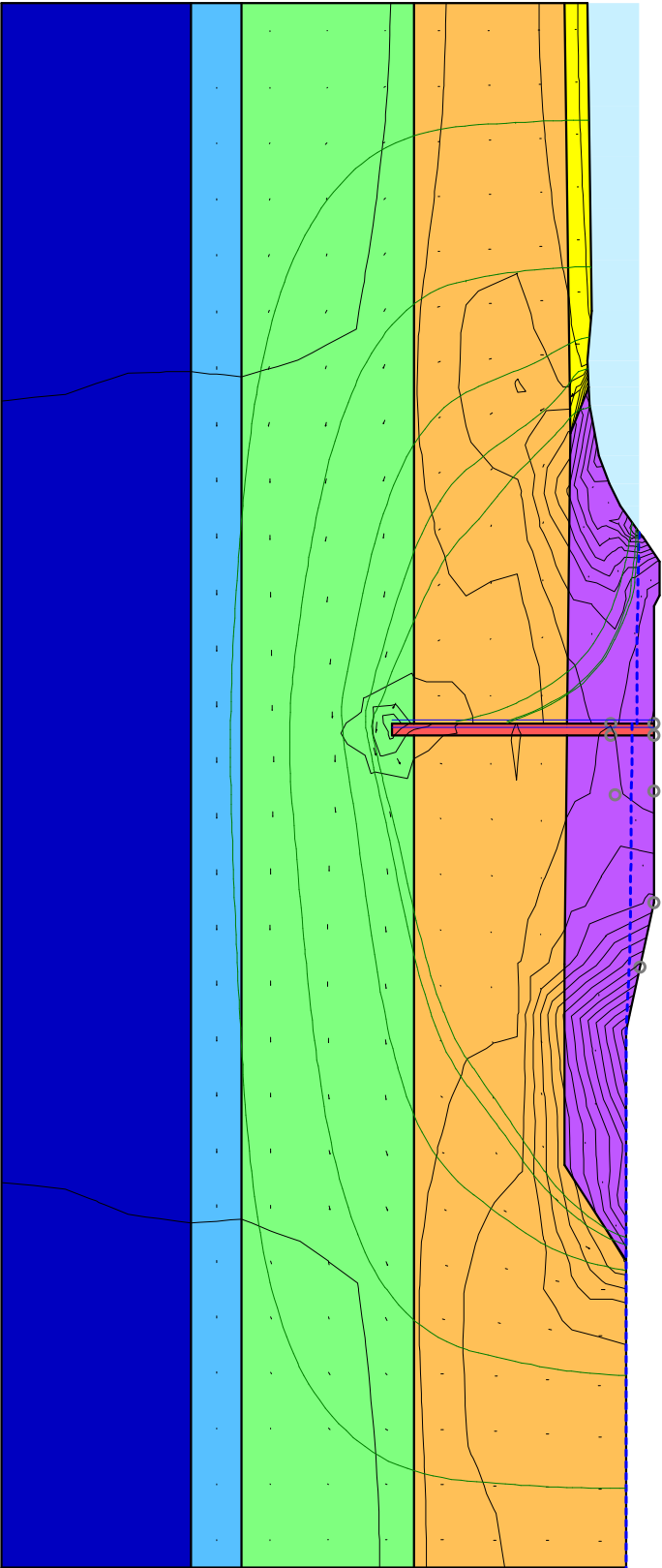
Location: Section 1 - Left of Primary Spillway
Factor of Safety: 0.945

Normal Pool - Upstream Slope - Sei 2475y
MassDOT Wareham Mill Pond Dam - Slope Stability & Seepage.gsz

02/07/2025

1:400

Color	Name	Hydraulic Material Model	Sat Kx (ft/sec)	Ky/Kx Ratio	Volumetric Water Content	Compressibility (psi)
Blue	BEDROCK	Saturated Only	1e-09	1	0	0
Purple	FILL	Saturated / Unsaturated		0.25		
Light Green	GLACIAL TILL	Saturated / Unsaturated		1		
Yellow	POND MUCK	Saturated / Unsaturated		1		
Orange	SAND & SILT	Saturated / Unsaturated		1		
Red	SECANT PILE	Saturated / Unsaturated		1		
Light Blue	WEATHERED BEDROCK	Saturated / Unsaturated		1		

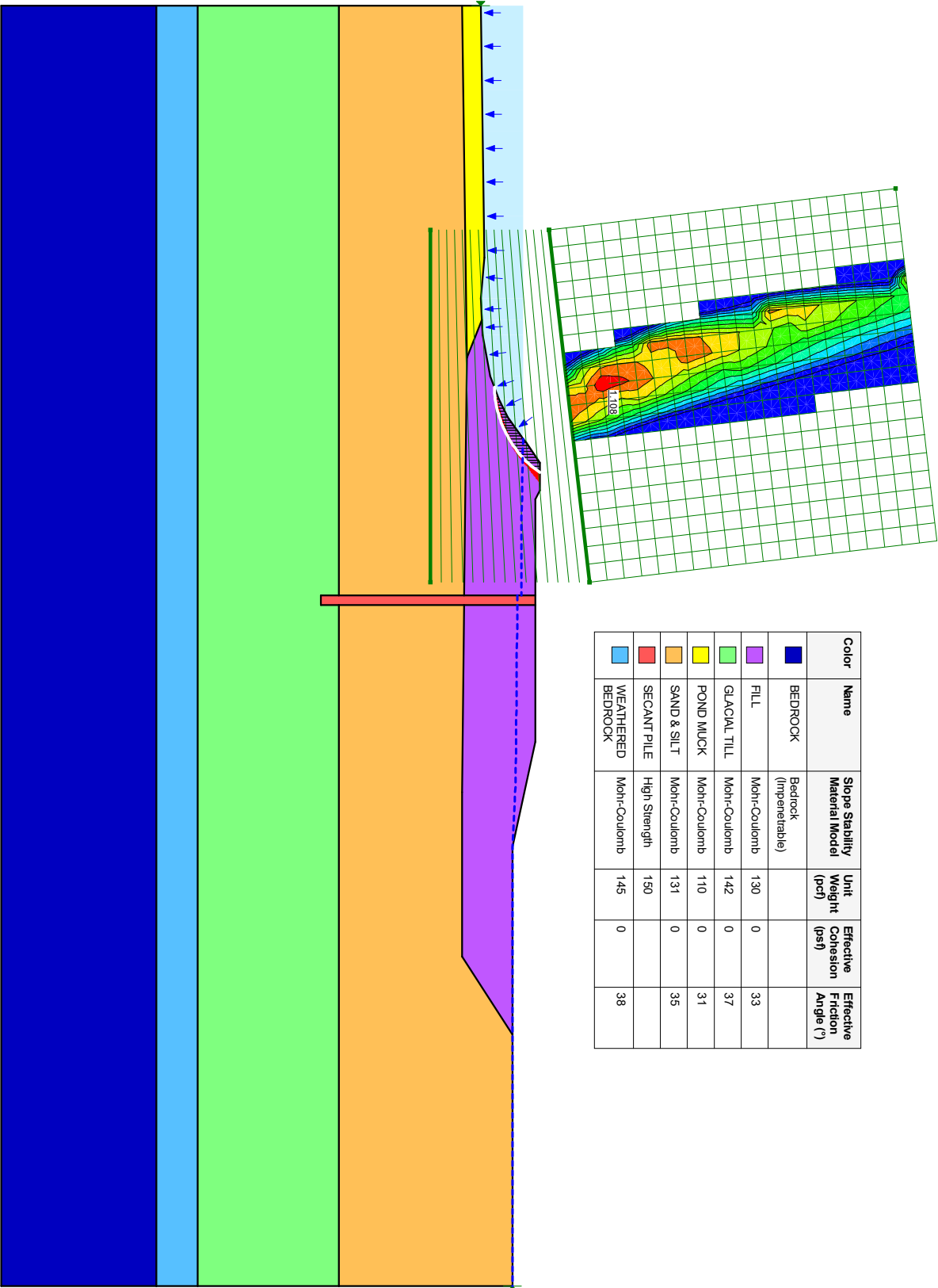


Section 1 - Normal Pool - PS

MassDOT Wareham Mill Pond Dam - Slope Stability & Seepage.gsz

02/07/2025

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Location: Section 1 - Left of Primary Spillway - PS
Factor of Safety: 1.108

Normal Pool - Upstream Slope - Post Sei
MassDOT Wareham Mill Pond Dam - Slope Stability & Seepage.gsz

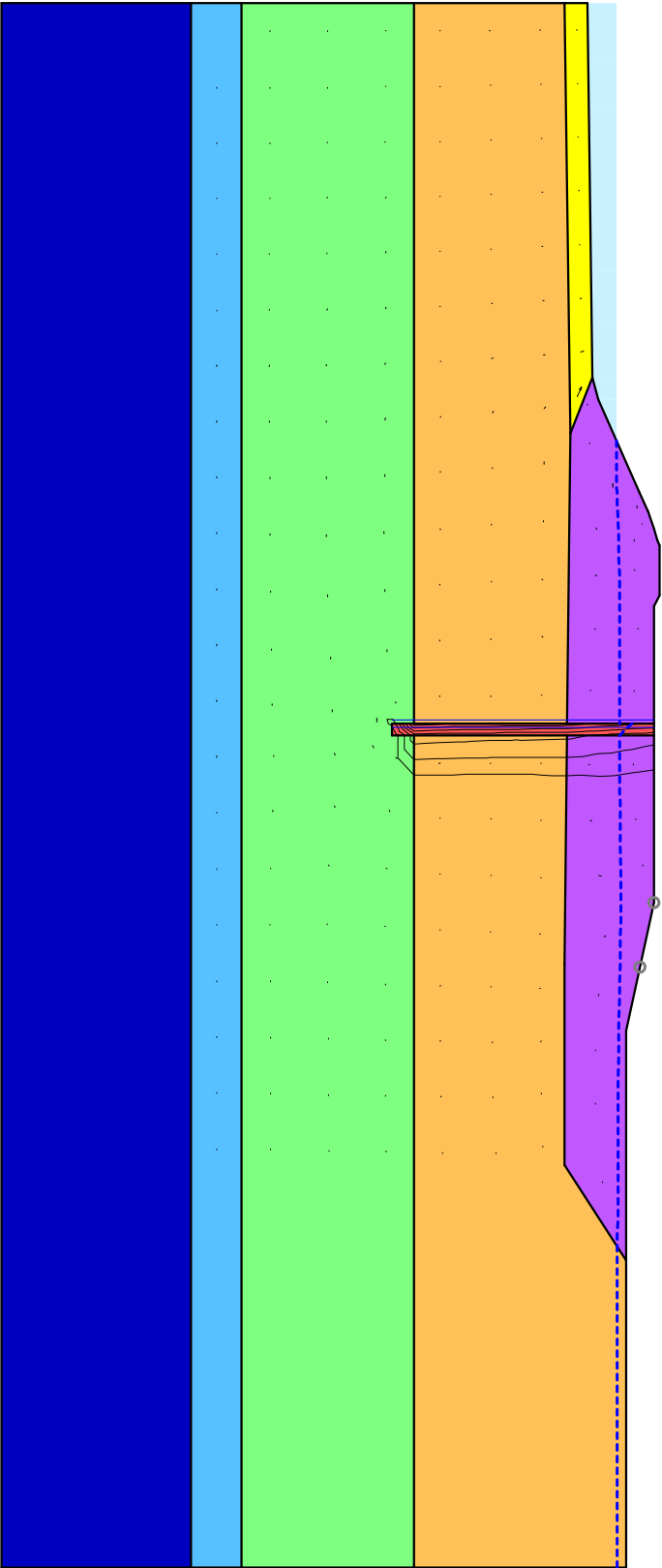
02/07/2025

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Drawdown by opening all spillways. Failure of dam is not considered.

Elapsed Time: 1.2 d

Color	Name	Hydraulic Material Model	Sat Kx (ft/sec)	Ky/Kx Ratio	Volumetric Water Content	Compressibility (psi)
Blue	BEDROCK	Saturated Only	1e-09	1	0	0
Purple	FILL	Saturated / Unsaturated		0.25		
Light Green	GLACIAL TILL	Saturated / Unsaturated		1		
Yellow	POND MUCK	Saturated / Unsaturated		1		
Orange	SAND & SILT	Saturated / Unsaturated		1		
Red	SECANT PILE	Saturated / Unsaturated		1		
Light Blue	WEATHERED BEDROCK	Saturated / Unsaturated		1		



Section 1 - Normal Pool - Rapid Drawdown

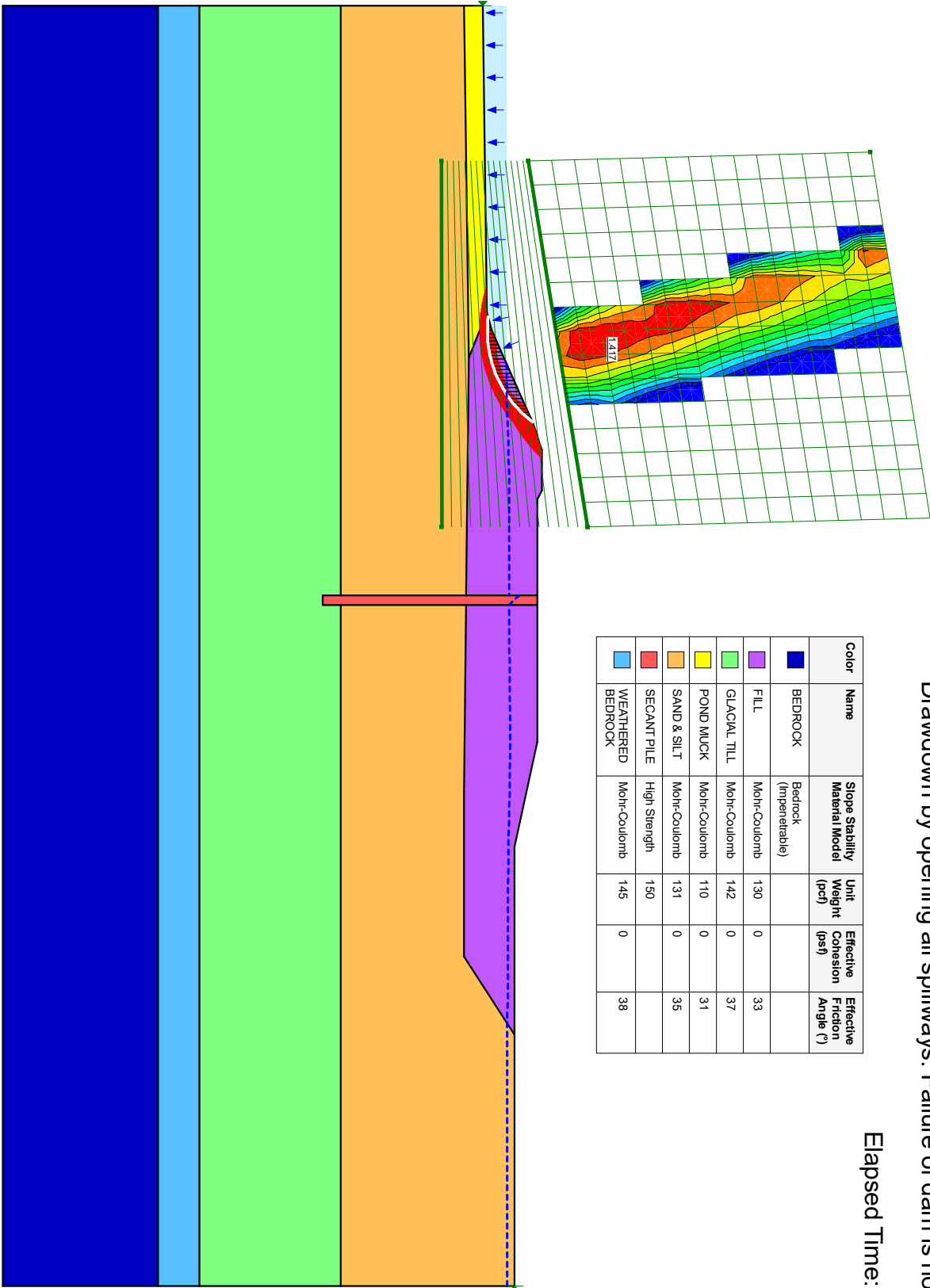
MassDOT Wareham Mill Pond Dam - Slope Stability & Seepage.gsz

02/07/2025

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Drawdown by opening all spillways. Failure of dam is not considered.

Elapsed Time: 1.2 d



Location: Section 1 - Left of Primary Spillway
Factor of Safety: 1.417

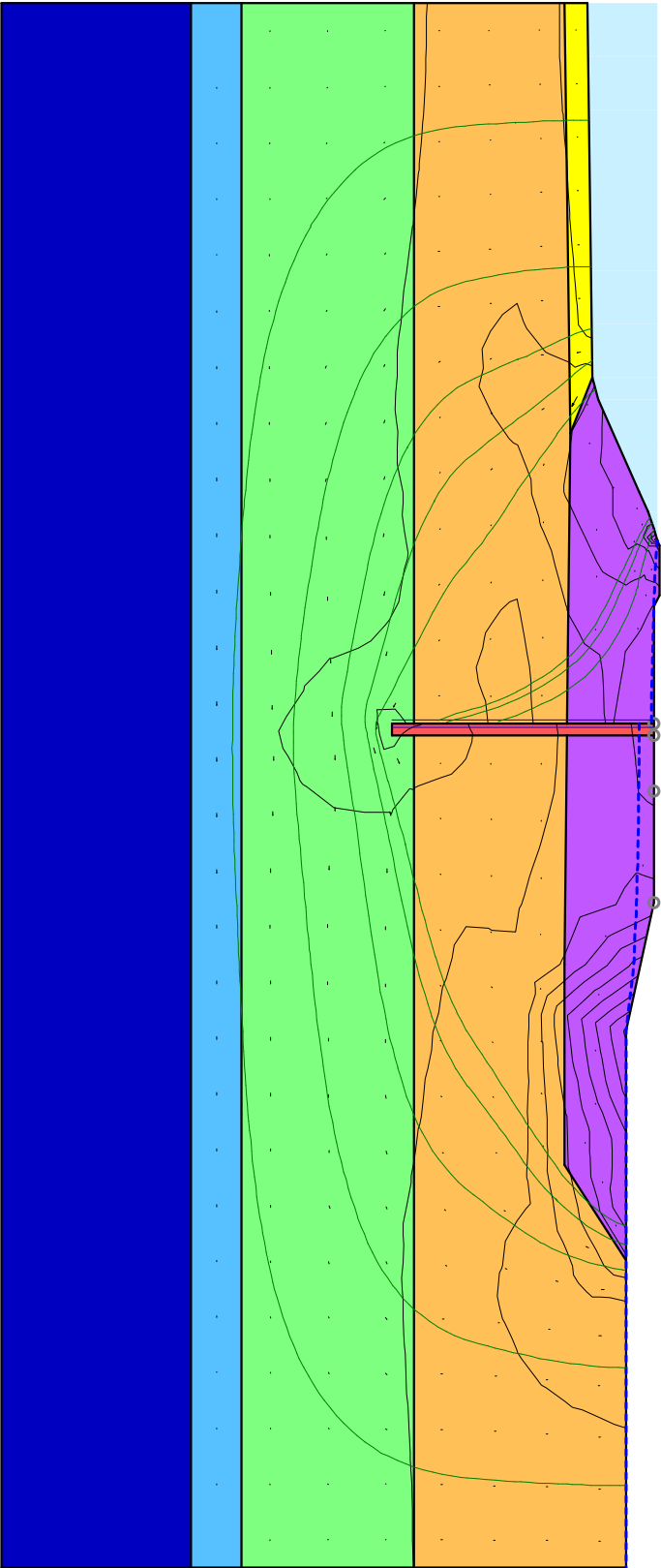
Rapid Drawdown - Upstream Slope

MassDOT Wareham Mill Pond Dam - Slope Stability & Seepage.gsz

02/07/2025

1:400

Color	Name	Hydraulic Material Model	Sat Kx (ft/sec)	Ky/Kx' Ratio	Volumetric Water Content	Compressibility (psi)
■	BEDROCK	Saturated Only	1e-09	1	0	0
■	FILL	Saturated / Unsaturated		0.25		
■	GLACIAL TILL	Saturated / Unsaturated		1		
■	POND MUCK	Saturated / Unsaturated		1		
■	SAND & SILT	Saturated / Unsaturated		1		
■	SECANT PILE	Saturated / Unsaturated		1		
■	WEATHERED BEDROCK	Saturated / Unsaturated		1		

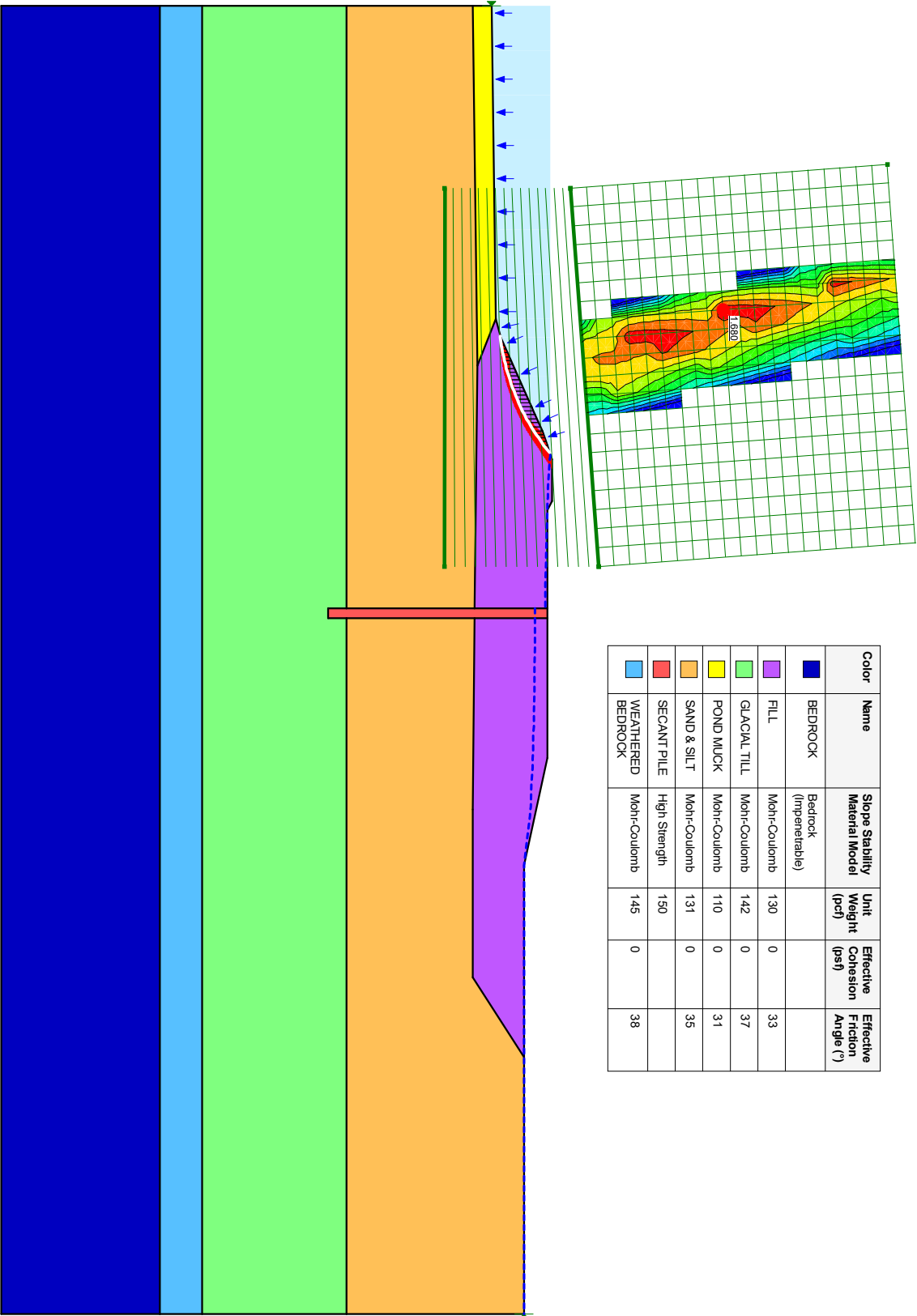


Section 1 - Flood Surgecharge

MassDOT Wareham Mill Pond Dam - Slope Stability & Seepage.gsz

02/07/2025

1:400



Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Blue	BEDROCK	Bedrock (Impenetrable)			
Purple	FILL	Mohr-Coulomb	130	0	33
Green	GLACIAL TILL	Mohr-Coulomb	142	0	37
Yellow	POND MUCK	Mohr-Coulomb	110	0	31
Orange	SAND & SILT	Mohr-Coulomb	131	0	35
Red	SECANT PILE	High Strength	150		
Light Blue	WEATHERED BEDROCK	Mohr-Coulomb	145	0	38

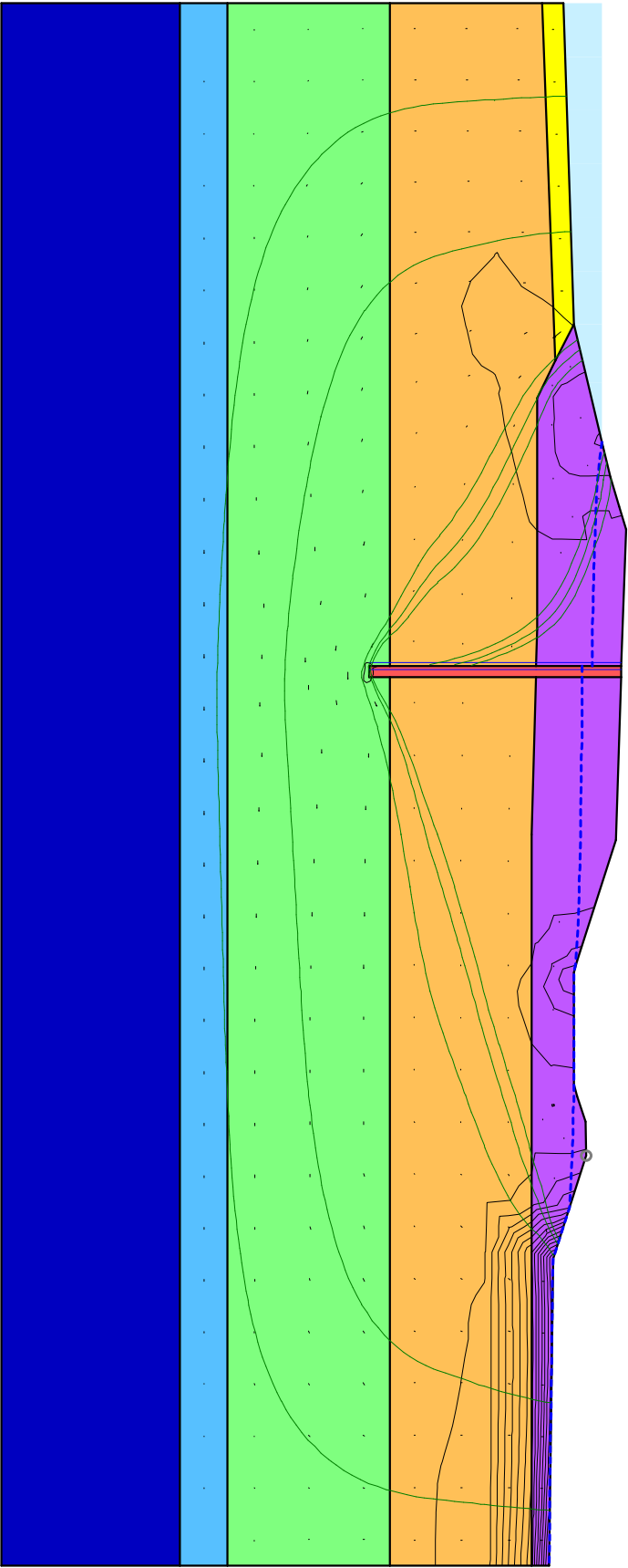
Location: Section 1 - Left of Primary Spillway
Factor of Safety: 1.680

Flood Surge - Upstream Slope
MassDOT Wareham Mill Pond Dam - Slope Stability & Seepage.gsz

02/07/2025

1:400

Color	Name	Hydraulic Material Model	Sat Kx (ft/sec)	Ky/Kx Ratio	Volumetric Water Content	Compressibility (/psi)
	BEDROCK	Saturated Only	1e-09	1	0	0
	FILL	Saturated / Unsaturated		0.25		
	GLACIAL TILL	Saturated / Unsaturated		1		
	POND MUCK	Saturated / Unsaturated		1		
	SAND & SILT	Saturated / Unsaturated		1		
	SECANT PILE	Saturated / Unsaturated		1		
	WEATHERED BEDROCK	Saturated / Unsaturated		1		

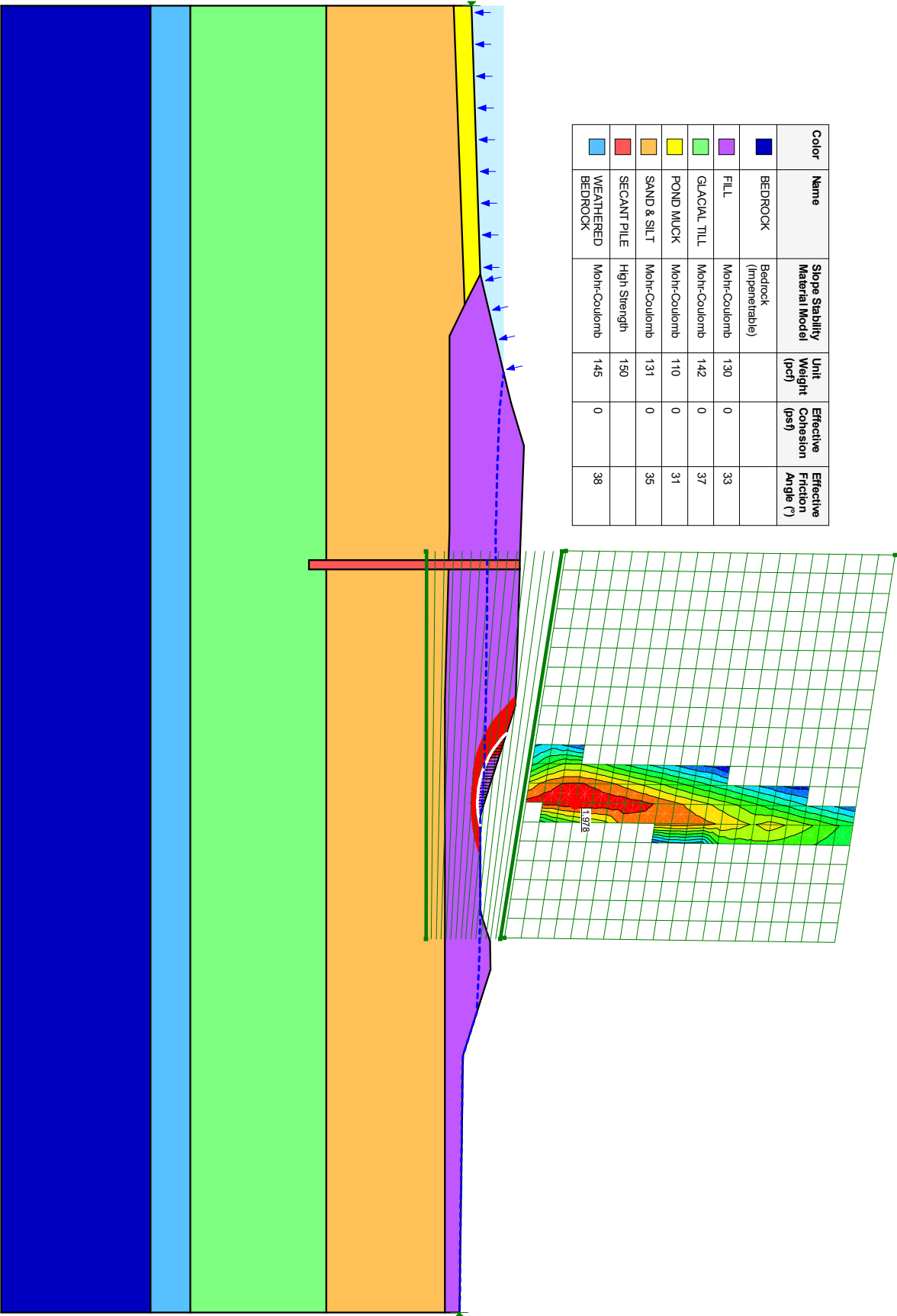


Section 2 - Normal Pool

MassDOT Wareham Mill Pond Dam - Slope Stability & Seepage.gsz

02/07/2025

1:400



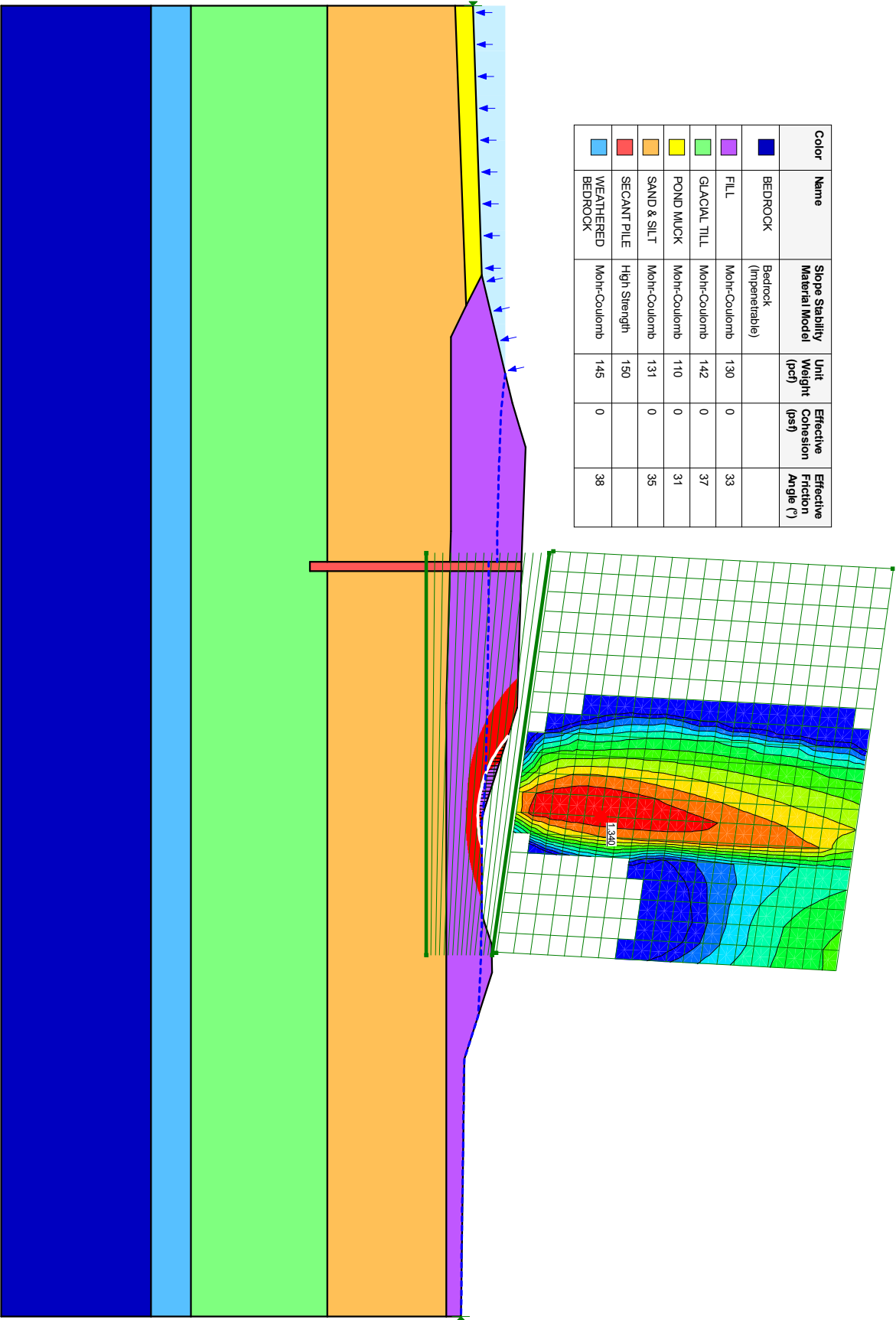
Location: Section 2 - Right of Primary Spillway
Factor of Safety: 1.978

Normal Pool - Downstream Slope

MassDOT Wareham Mill Pond Dam - Slope Stability & Seepage.gsz

02/07/2025

1:400



Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
Blue	BEDROCK	Bedrock (Impenetrable)			
Purple	FILL	Mohr-Coulomb	130	0	33
Green	GLACIAL TILL	Mohr-Coulomb	142	0	37
Yellow	POND MUCK	Mohr-Coulomb	110	0	31
Orange	SAND & SILT	Mohr-Coulomb	131	0	35
Red	SECANT PILE	High Strength	150		
Light Blue	WEATHERED BEDROCK	Mohr-Coulomb	145	0	38

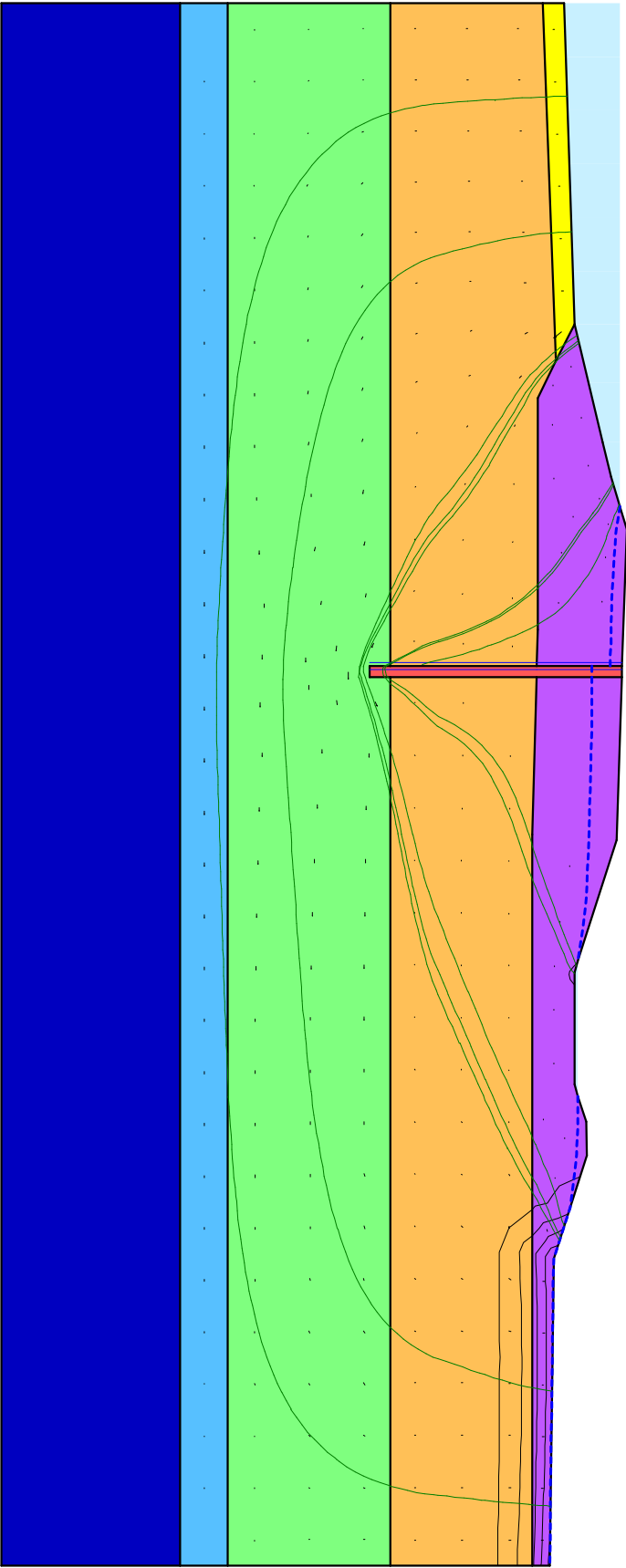
Location: Section 2 - Right of Primary Spillway
Factor of Safety: 1.340

Normal Pool - Downstream Slope - Sei 2475y
MassDOT Wareham Mill Pond Dam - Slope Stability & Seepage.gsz

02/07/2025

1:400

Color	Name	Hydraulic Material Model	Sat Kx (ft/sec)	Ky/Kx Ratio	Volumetric Water Content	Compressibility (/psi)
	BEDROCK	Saturated Only	1e-09	1	0	0
	FILL	Saturated / Unsaturated		0.25		
	GLACIAL TILL	Saturated / Unsaturated		1		
	POND MUCK	Saturated / Unsaturated		1		
	SAND & SILT	Saturated / Unsaturated		1		
	SECANT PILE	Saturated / Unsaturated		1		
	WEATHERED BEDROCK	Saturated / Unsaturated		1		

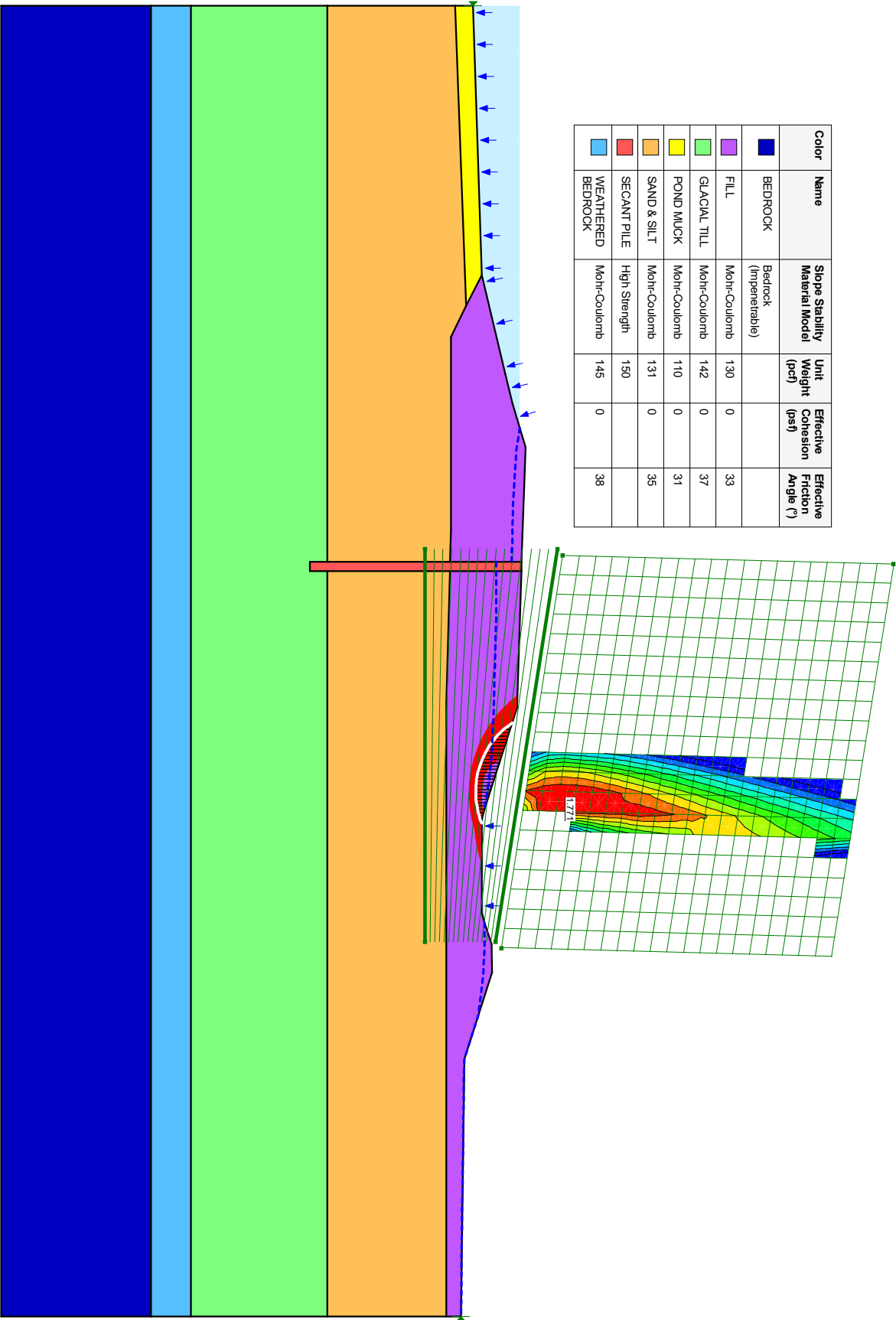


Section 2 - Flood Surge

MassDOT Wareham Mill Pond Dam - Slope Stability & Seepage.gsz

02/07/2025

1:400



Location: Section 2 - Right of Primary Spillway
Factor of Safety: 1.771

Flood Surge - Downstream Slope
MassDOT Wareham Mill Pond Dam - Slope Stability & Seepage.gsz

02/07/2025

1:400



Project: Mill Pond Dam
Subject: Soil Parameters
Computations By: OB
Checked By: BHN

Proj. No.: 22147.00
Date: 1/28/2025
Date: 1/30/2025

GEOTECHNICAL PROPERTIES:

Layer No.	Soil Type	Material Type	N160 (Blows/ft)	D _r (%)	γ_{dry} (PCF)	e	ϕ' (degrees)	ϕ_u (degrees)	C _u (psf)	γ_{sat} (PCF)	γ_m (PCF)	γ_b (PCF)	W _{sat}	W _{as}	K _{sat} (ft/sec)	Anisotropy (Kv/Kh)	At rest earth pressure, Ko	Active earth pressure, Ka	Passive Earth Pressure, Kp	Notes
1	POND MUCK	SM	25.0	52.5	110.0	0.75	31	N/A	N/A	136.7	123.4	74.3	0.28	0.020	6.63E-03	1.00	0.48	0.32	3.12	
2	FILL	SM	ND	ND	122.2	0.65	33	ND	ND	130.0	126.1	67.6	0.25	0.020	5.00E-04	0.25	0.46	0.29	3.39	
2	SAND AND SILT	SM	61.0	95.0	111.0	0.54	35	N/A	N/A	131.0	121.0	64.2	0.20	0.035	1.64E-03	1.00	0.43	0.27	3.69	
2A	GLACIAL TILL	SM	100	95.0	127.0	0.32	37	N/A	N/A	142.0	134.5	80.0	0.12	0.020	6.63E-03	1.00	0.40	0.25	4.02	
2B	WEATHERED BEDROCK	SM	100	95.0	127.0	0.32	37	N/A	N/A	142.0	134.5	80.0	0.12	N/A	N/A	1.00	0.40	0.25	4.02	

GEOTECHNICAL PROPERTIES FOOTNOTE LIST:

Layer No.	Soil Type	Material Type	N160 (Blows/ft)	D _r (%)	γ _{dry} (pcf)	e	φ ¹ (degrees)	φ _v (degrees)	c _u (psf)	γ _{sat} (pcf)	γ _m (pcf)	γ _b (pcf)	W _{sat}	W _{min}	K _{sat} (mbsec)	Anisotropy (Kv/Kh)	At rest earth pressure, Ko	Active earth pressure, Ka	Passive Earth Pressure, Kp	Comments
1	POND MUCK	1	2	4	7	7	7	NA	NA	11	12	13	1	15	25	24	16	17	18	
2	FILL	1	3	3	26	26	9	3	3	11	12	13	9	15	9	24	16	17	18	
2	SAND AND SILT	1	2	4	7	7	7	NA	NA	11	12	13	1	15	25	24	16	17	18	
2A	GLACIAL TILL	1	2	4	7	7	7	NA	NA	11	12	13	1	15	25	24	16	17	18	
2B	WEATHERED BEDROCK	1	2	4	7	7	7	NA	NA	11	12	13	1	15	25	24	16	17	18	

1. Grain Size and Moisture Content Analyses, Boring Logs, and Engineering Judgement
2. Average SPT Data from Subsurface Exploration Program. Corrections for rod and hammer efficiencies as well as effective stress were made. See Liquefaction Calculations
3. Not Determined
4. Das, B., "Principles of Geotechnical Engineering", 8th edition, Table 17.6
5. 98% of Maximum Dry Density of conducted Modified Proctor ASTM D698 to mimic specified as-built conditions
6. 98% of Maximum Dry Density of conducted Modified Proctor ASTM D1557 to mimic future as-placed conditions
7. United Facilities Criteria UFC 3-220-10N, 2022, Figure 8-1, Correlations were taken from determined USCS ClassificationMaterial Type and calculated Relative Density from Ref #4
8. US Army Corps of Engineers Engineer Research and Development Center, "Mechanical and Physical Properties of ASTM C33 Sand", pp.29-30
9. Engineering Judgement
10. Lab recorded void ratio from Direct Shear Tests, UU Tests, CU Tests, and engineering judgement
11. γ_{sat} = γ_{dry} + (e/(1+e))γ_w
Kulhawy and Mayne
12. moist = (γ_{dry}γ_{sat})/Z
13. γ_{soil}γ_{water} = γ_{sat}γ_{water}
14. W_{sat} = e/Gs
15. Rawls, W.J., Brakenek, D.L., Saxton, E., "Estimation of Soil Water Properties", Transactions of the ASAE Vol. 25, No.5, p. 1319, Table 2, and Grain Size Analyses
16. Das, B., "Principles of Geotechnical Engineering", 8th edition, Equation 13.5
17. Das, B., "Principles of Geotechnical Engineering", 8th edition, Equation 13.19
18. Das, B., "Principles of Geotechnical Engineering", 8th edition, Equation 13.56
19. Lab Constant Head Permeability ASTM D2434-22
20. Field Falling Head Test 2
21. Flexible Wall Permeability Test ASTM D5084
22. Field Falling Head Test 3
23. Seepage Model Calibration against observed Piezometer readings and Engineering Judgement
24. USBR Design Standards No. 13 Chapter 8: Seepage, model calibration against observed piezometer readings, and Engineering Judgement

Calculation Cover Sheet

Project #: 22147.02 **Calculation #:** 1
Project: Mill Pond Dam Improvements **Date:** See Below
Subject/Task: Secant / Soldier Pile Wall Stability Analysis
Calculated by: OB **Reviewed by:**

Revision Summary:

Revision #	Description	Date
0	Original Calculation	12/2024

Subject:

This calculation was performed to determine the Embedment Depth, Deflection, and Wall Stability of the Mill Pond Dam embankment and Cofferdam during all phases of construction and anticipated water levels during construction. Final Design is not emphasized herein, since conditions during construction far exceed construction requirements.

Information Provided/Used in the analysis:

1. Soil properties from the Mill Pond Dam Soil Parameters Table prepared by Pare Corporation in December 2021 and modified in January 2025.
2. Hydrologic and Hydraulic Data from HydroCAD model prepared by Pare Corporation in December 2021 and modified in January 2025.
3. Geometry of Mill Pond Dam from 22147.02 Permit Drawings prepared by Pare Corporation dated December 2025.
4. ShoringSuit Version 8.16g

References:

1. 22147.02 Soil Parameters Table, Mill Pond Dam, Prepared by Pare Corporation in December 2021.
2. As-Built construction plans and survey data.

1.0 – MODEL INPUTS

Methodology:

Dam Embankment Geometry:

Critical Section: The critical sections were taken left of the proposed primary spillway for the Secant Cut-off Wall with 2-foot diameter piles (including steel reinforcement every second pile), and through the proposed Secant-wall itself with 3-foot diameter piles (including steel reinforcement every second pile). Sections were determined by excavation depth at the downstream side during construction at the designated phase. The Soldier Pile Cofferdam was considered to remain impenetrable for water, requiring special precautions, including sealants in seams, geofabric on the waterside, and/or other. Below mudline the Soldier Pile Wall transitions to a Secant Wall design with reinforcements as a continuation of the Soldier Pile Wall support beams, preventing seepage or hydraulic failure under the lagging.

Profile Selection: Reinforcements (Steel profiles) were selected primarily on computed maximum bending moment and understanding of practical aspects in installation and spacing. Concrete profiles were selected primarily on deflection implications during varying water levels in the impoundment. Limited deflections were considered critical for the Secant Cut-off Wall to maintain functionality Route 6/28 throughout all construction phases

Excavation Support: Tiebacks or Braces were considered during the design process to reduce the section diameter of piles. Tiebacks were disregarded due to insufficient anchoring depth and/or overburden within the impoundment, whereas bracing heavily impacted construction applications in proximity to the wall. A cantilevered wall system was selected as the optimal solution for this project.

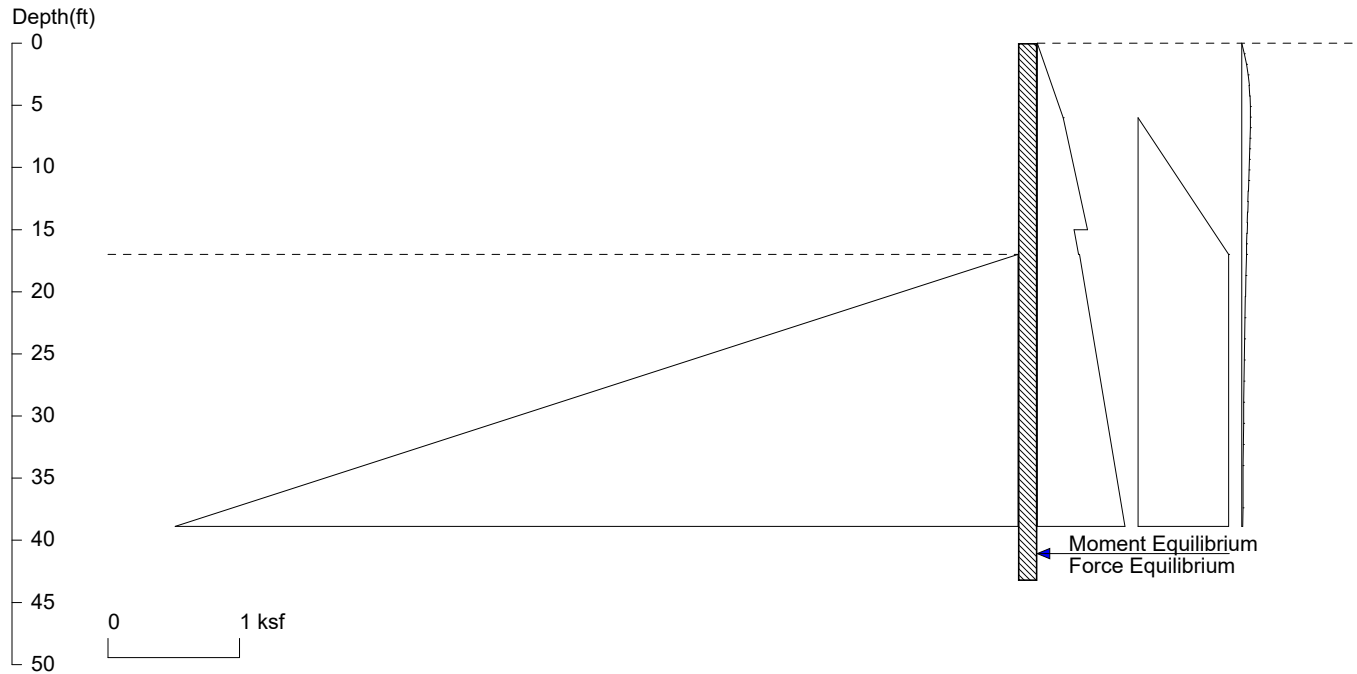
Soil Parameters:

Soil parameters were copied into GeoStudio from Pare's soil parameters table.

1.0 – MODEL RESULTS

See the attached Shoring Suite Results

Calculation by:	Orell Baumann	Engineer II		4/28/2025
	Name	Position	Signature	Date
Checked by:	David Caouette	Managing Engineer		4/28/2025
	Name	Position	Signature	Date



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Date: 12/17/2024

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Wall Height=17.0 Pile Diameter=2.0 Pile Spacing=3.3 Wall Type: 4. Secant/Tangent

PILE LENGTH: Min. Embedment=26.26 Min. Pile Length=43.26

MOMENT IN PILE: Max. Moment=426.96 per Pile Spacing=3.3 at Depth=29.16

PILE SELECTION:

Request Min. Section Modulus = $155.3 \text{ in}^3/\text{pile} = 2544.25 \text{ cm}^3/\text{pile}$, $F_y = 50 \text{ ksi} = 345 \text{ MPa}$, $F_b/F_y = 0.66$

HP18X135 has Section Modulus = 251.0 in³/pile=4113.14 cm³/pile. It is greater than Min. Requirements!

Top Deflection = 1.62(in) based on E (ksi)=29000.00 and I (in4)/pile=2196.0

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

Z1	P1	Z2	P2	Slope
*	Above	Base		
0.000	0.000	6.000	0.196	0.032643
6.000	0.196	15.00	0.381	0.020546
15.00	0.280	17.00	0.313	0.016376
*	Below	Base		
17.00	0.320	45.00	0.763	0.015819
*	Water	Pres.		
6.000	0.000	17.00	0.686	0.062400
17.00	0.686	153.0	0.686	0.000000
*	Sur-	charge		
0.000	0.000	0.850	0.019	0.022611
0.850	0.019	1.700	0.036	0.019682
1.700	0.036	2.550	0.049	0.015049
2.550	0.049	3.400	0.057	0.010189
3.400	0.057	4.250	0.063	0.006033
4.250	0.063	5.100	0.065	0.002874
5.100	0.065	5.950	0.066	0.000635
5.950	0.066	6.800	0.065	-0.000883

Proposal No. 606352-136278

6.800	0.065	7.650	0.063	-0.001880
7.650	0.063	8.500	0.061	-0.002513
8.500	0.061	9.350	0.059	-0.002895
9.350	0.059	10.200	0.056	-0.003105
10.200	0.056	11.050	0.053	-0.003198
11.050	0.053	11.900	0.050	-0.003209
11.900	0.050	12.750	0.048	-0.003166
12.750	0.048	13.600	0.045	-0.003085
13.600	0.045	14.450	0.043	-0.002980
14.450	0.043	15.300	0.040	-0.002858
15.300	0.040	16.150	0.038	-0.002727
16.150	0.038	17.000	0.036	-0.002591
17.000	0.036	18.700	0.032	-0.002386
18.700	0.032	20.400	0.028	-0.002120
20.400	0.028	22.100	0.025	-0.001872
22.100	0.025	23.800	0.022	-0.001646
23.800	0.022	25.500	0.020	-0.001444
25.500	0.020	27.200	0.017	-0.001265
27.200	0.017	28.900	0.016	-0.001108
28.900	0.016	30.600	0.014	-0.000970
30.600	0.014	32.300	0.012	-0.000851
32.300	0.012	34.000	0.011	-0.000747
34.000	0.011	37.400	0.009	-0.000618
37.400	0.009	40.800	0.007	-0.000481
40.800	0.007	44.200	0.006	-0.000378

PASSIVE PRESSURES: Pressures below will be divided by a Factor of Safety =1.3

Z1	P1	Z2	P2	Slope
*	Below	Base		
17.00	0.000	45.00	8.192	0.292581

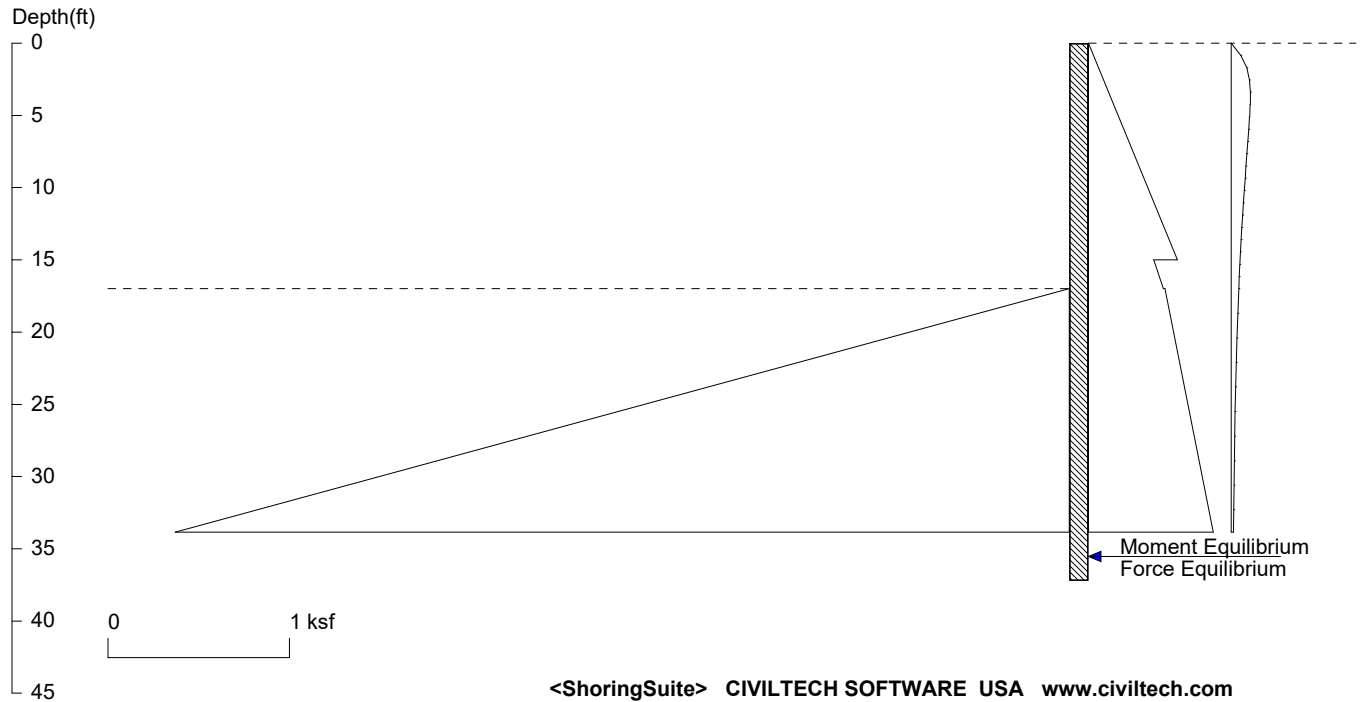
ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	3.33
2	17.00	2.00

PASSIVE SPACING:

No.	Z depth	Spacing
1	17.00	3.33

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft³; Deflection - in



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Date: 1/29/2025

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Wall Height=17.0

Pile Diameter=2.0

Pile Spacing=3.3

Wall Type: 4. Secant/Tangent

PILE LENGTH: Min. Embedment=20.22 Min. Pile Length=37.22

MOMENT IN PILE: Max. Moment=246.34 per Pile Spacing=3.3 at Depth=25.61

PILE SELECTION:Request Min. Section Modulus = 89.6 in³/pile=1467.94 cm³/pile, F_y= 50 ksi = 345 MPa, F_b/F_y=0.66HP18X135 has Section Modulus = 251.0 in³/pile=4113.14 cm³/pile. It is greater than Min. Requirements!Top Deflection = 0.91(in) based on E (ksi)=29000.00 and I (in⁴)/pile=2196.0**DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):**

Z1	P1	Z2	P2	Slope
*	Above	Base		
0.000	0.000	15.000	0.490	0.032643
15.000	0.360	17.000	0.413	0.026353
*	Below	Base		
17.000	0.421	45.000	0.863	0.015793
*	Sur-	charge		
0.000	0.000	0.850	0.052	0.061352
0.850	0.052	1.700	0.085	0.039057
1.700	0.085	2.550	0.100	0.017493
2.550	0.100	3.400	0.105	0.005052
3.400	0.105	4.250	0.104	-0.001024
4.250	0.104	5.100	0.100	-0.003884
5.100	0.100	5.950	0.096	-0.005210
5.950	0.096	6.800	0.091	-0.005785
6.800	0.091	7.650	0.086	-0.005974
7.650	0.086	8.500	0.081	-0.005953
8.500	0.081	9.350	0.076	-0.005814
9.350	0.076	10.200	0.071	-0.005605

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6.800	0.065	7.650	0.063	-0.001880
7.650	0.063	8.500	0.061	-0.002513
8.500	0.061	9.350	0.059	-0.002895
9.350	0.059	10.200	0.056	-0.003105
10.200	0.056	11.050	0.053	-0.003198
11.050	0.053	11.900	0.050	-0.003209
11.900	0.050	12.750	0.048	-0.003166
12.750	0.048	13.600	0.045	-0.003085
13.600	0.045	14.450	0.043	-0.002980
14.450	0.043	15.300	0.040	-0.002858
15.300	0.040	16.150	0.038	-0.002727
16.150	0.038	17.000	0.036	-0.002591
17.000	0.036	18.700	0.032	-0.002386
18.700	0.032	20.400	0.028	-0.002120
20.400	0.028	22.100	0.025	-0.001872
22.100	0.025	23.800	0.022	-0.001646
23.800	0.022	25.500	0.020	-0.001444
25.500	0.020	27.200	0.017	-0.001265
27.200	0.017	28.900	0.016	-0.001108
28.900	0.016	30.600	0.014	-0.000970
30.600	0.014	32.300	0.012	-0.000851
32.300	0.012	34.000	0.011	-0.000747
34.000	0.011	37.400	0.009	-0.000618
37.400	0.009	40.800	0.007	-0.000481
40.800	0.007	44.200	0.006	-0.000378
44.200	0.006	47.600	0.005	-0.000300
47.600	0.005	51.000	0.004	-0.000240

PASSIVE PRESSURES: Pressures below will be divided by a Factor of Safety =1.3

Z1	P1	Z2	P2	Slope
*	Below	Base		
22.00	0.000	45.00	6.729	0.292581
45.00	5.889	153.0	37.64	0.294044

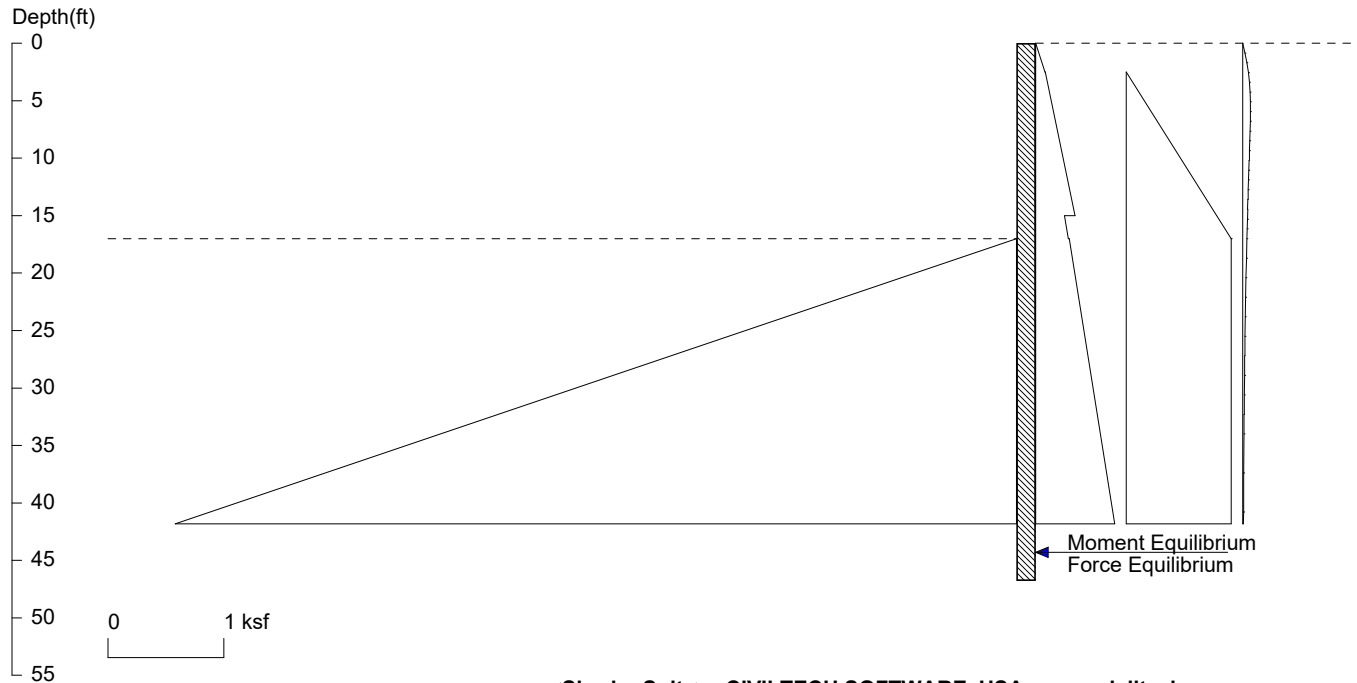
ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	3.33
2	17.00	2.00

PASSIVE SPACING:

No.	Z depth	Spacing
1	17.00	3.33

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft³; Deflection - in



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Date: 12/17/2024

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Wall Height=17.0

Pile Diameter=2.0

Pile Spacing=3.3

Wall Type: 4. Secant/Tangent

PILE LENGTH: Min. Embedment=29.77 Min. Pile Length=46.77

MOMENT IN PILE: Max. Moment=605.71 per Pile Spacing=3.3 at Depth=30.89

PILE SELECTION:Request Min. Section Modulus = 220.3 in³/pile=3609.36 cm³/pile, F_y = 50 ksi = 345 MPa, F_b/F_y=0.66HP18X135 has Section Modulus = 251.0 in³/pile=4113.14 cm³/pile. It is greater than Min. Requirements!Top Deflection = 2.55(in) based on E (ksi)=29000.00 and I (in⁴)/pile=2196.0**DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):**

Z1	P1	Z2	P2	Slope
*	Above	Base		
0.000	0.000	2.500	0.082	0.032643
2.500	0.082	15.00	0.338	0.020546
15.00	0.249	17.00	0.282	0.016376
*	Below	Base		
17.00	0.289	45.00	0.732	0.015839
45.00	0.784	153.0	2.869	0.019308
*	Water	Pres.		
2.500	0.000	17.00	0.905	0.062400
17.00	0.905	153.0	0.905	0.000000
*	Sur-	charge		
0.000	0.000	0.850	0.019	0.022611
0.850	0.019	1.700	0.036	0.019682
1.700	0.036	2.550	0.049	0.015049
2.550	0.049	3.400	0.057	0.010189
3.400	0.057	4.250	0.063	0.006033
4.250	0.063	5.100	0.065	0.002874
5.100	0.065	5.950	0.066	0.000635

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5.950	0.066	6.800	0.065	-0.000883
6.800	0.065	7.650	0.063	-0.001880
7.650	0.063	8.500	0.061	-0.002513
8.500	0.061	9.350	0.059	-0.002895
9.350	0.059	10.200	0.056	-0.003105
10.200	0.056	11.050	0.053	-0.003198
11.050	0.053	11.900	0.050	-0.003209
11.900	0.050	12.750	0.048	-0.003166
12.750	0.048	13.600	0.045	-0.003085
13.600	0.045	14.450	0.043	-0.002980
14.450	0.043	15.300	0.040	-0.002858
15.300	0.040	16.150	0.038	-0.002727
16.150	0.038	17.000	0.036	-0.002591
17.000	0.036	18.700	0.032	-0.002386
18.700	0.032	20.400	0.028	-0.002120
20.400	0.028	22.100	0.025	-0.001872
22.100	0.025	23.800	0.022	-0.001646
23.800	0.022	25.500	0.020	-0.001444
25.500	0.020	27.200	0.017	-0.001265
27.200	0.017	28.900	0.016	-0.001108
28.900	0.016	30.600	0.014	-0.000970
30.600	0.014	32.300	0.012	-0.000851
32.300	0.012	34.000	0.011	-0.000747
34.000	0.011	37.400	0.009	-0.000618
37.400	0.009	40.800	0.007	-0.000481
40.800	0.007	44.200	0.006	-0.000378
44.200	0.006	47.600	0.005	-0.000300

PASSIVE PRESSURES: Pressures below will be divided by a Factor of Safety =1.3

Z1	P1	Z2	P2	Slope
*	Below	Base		
17.00	0.000	45.00	8.192	0.292581
45.00	7.166	153.0	38.93	0.294138

ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	3.33
2	17.00	2.00

PASSIVE SPACING:

No.	Z depth	Spacing
1	17.00	3.33

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft³; Deflection - in

Z1	P1	Z2	P2	Slope
*	Above	Base		
0.000	0.000	15.00	0.490	0.032643
15.00	0.360	17.00	0.413	0.026353
17.00	0.414	22.00	0.498	0.016724
*	Below	Base		
22.00	0.506	45.00	0.859	0.015360
45.00	0.916	176.0	3.347	0.018558
*	Water	Pres.		
17.00	0.000	22.00	0.312	0.062400
22.00	0.312	176.0	0.312	0.000000
*	Sur-	charge		
0.000	0.000	1.100	0.024	0.022262
1.100	0.024	2.200	0.044	0.017726
2.200	0.044	3.300	0.057	0.011449
3.300	0.057	4.400	0.063	0.005954
4.400	0.063	5.500	0.065	0.002082
5.500	0.065	6.600	0.065	-0.000358
6.600	0.065	7.700	0.063	-0.001798

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7.700	0.063	8.800	0.060	-0.002602
8.800	0.060	9.900	0.057	-0.003013
9.900	0.057	11.000	0.053	-0.003184
11.000	0.053	12.100	0.050	-0.003206
12.100	0.050	13.200	0.046	-0.003138
13.200	0.046	14.300	0.043	-0.003015
14.300	0.043	15.400	0.040	-0.002862
15.400	0.040	16.500	0.037	-0.002691
16.500	0.037	17.600	0.034	-0.002515
17.600	0.034	18.700	0.032	-0.002338
18.700	0.032	19.800	0.029	-0.002166
19.800	0.029	20.900	0.027	-0.002000
20.900	0.027	22.000	0.025	-0.001844
22.000	0.025	24.200	0.021	-0.001628
24.200	0.021	26.400	0.018	-0.001373
26.400	0.018	28.600	0.016	-0.001157
28.600	0.016	30.800	0.014	-0.000975
30.800	0.014	33.000	0.012	-0.000822
33.000	0.012	35.200	0.010	-0.000695
35.200	0.010	37.400	0.009	-0.000590
37.400	0.009	39.600	0.008	-0.000502
39.600	0.008	41.800	0.007	-0.000428
41.800	0.007	44.000	0.006	-0.000367
44.000	0.006	48.400	0.005	-0.000294

PASSIVE PRESSURES: Pressures below will be divided by a Factor of Safety =1.3

Z1	P1	Z2	P2	Slope
*	Below	Base		
22.00	0.000	45.00	6.729	0.292581
45.00	5.892	176.0	44.40	0.293977

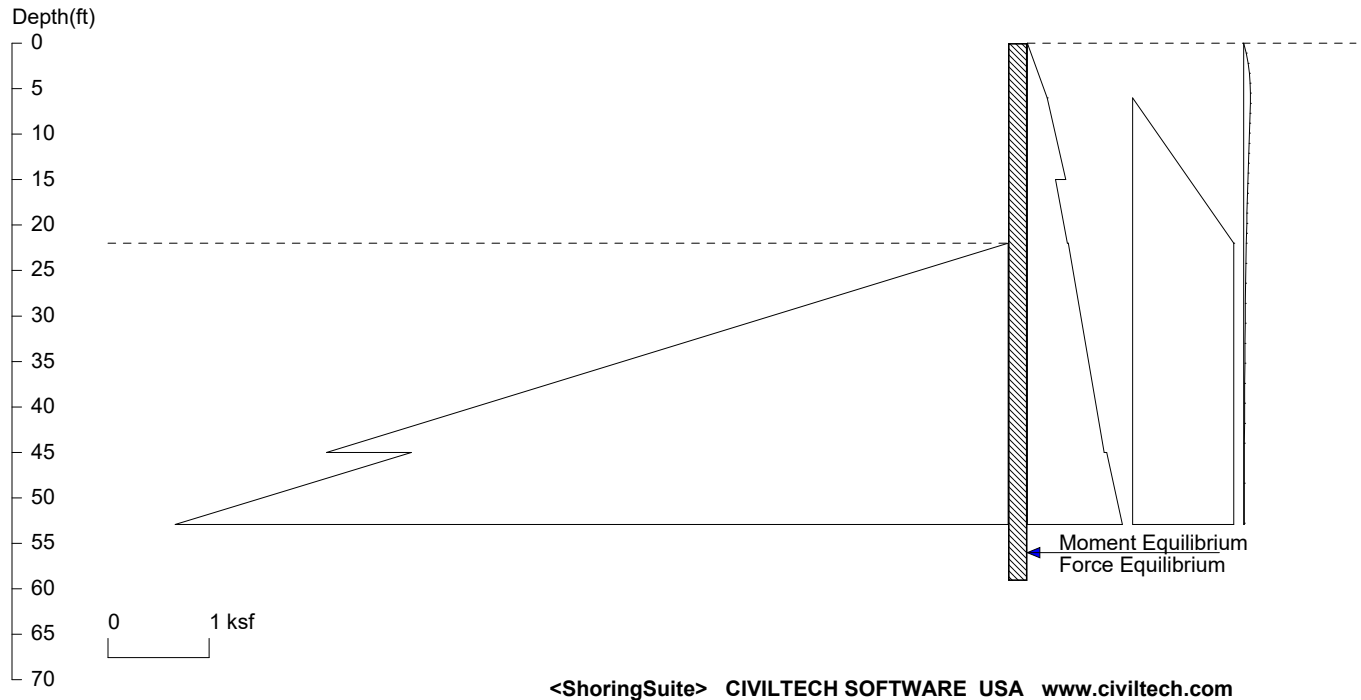
ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	3.33
2	22.00	2.00

PASSIVE SPACING:

No.	Z depth	Spacing
1	22.00	3.33

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft³; Deflection - in



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Wall Height=22.0 Pile Diameter=3.0 Pile Spacing=4.2 Wall Type: 4. Secant/Tangent

PILE LENGTH: Min. Embedment=37.12 Min. Pile Length=59.12

MOMENT IN PILE: Max. Moment=1343.01 per Pile Spacing=4.2 at Depth=39.34

PILE SELECTION:

Request Min. Section Modulus = 488.4 in³/pile=8002.85 cm³/pile, F_y= 50 ksi = 345 MPa, F_b/F_y=0.66

W27X217 has Section Modulus = 627.0 in³/pile=10274.65 cm³/pile. It is greater than Min. Requirements!

Top Deflection = 2.20(in) based on E (ksi)=29000.00 and I (in4)/pile=8910.0

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

Z1	P1	Z2	P2	Slope
*	Above	Base		
0.000	0.000	6.000	0.196	0.032643
6.000	0.196	15.00	0.381	0.020546
15.00	0.279	22.00	0.398	0.016921
*	Below	Base		
22.00	0.405	45.00	0.759	0.015411
45.00	0.785	176.0	3.340	0.019504
*	Water	Pres.		
6.000	0.000	22.00	0.998	0.062400
22.00	0.998	176.0	0.998	0.000000
*	Sur-	charge		
0.000	0.000	1.100	0.024	0.022262
1.100	0.024	2.200	0.044	0.017726
2.200	0.044	3.300	0.057	0.011449
3.300	0.057	4.400	0.063	0.005954
4.400	0.063	5.500	0.065	0.002082
5.500	0.065	6.600	0.065	-0.000358
6.600	0.065	7.700	0.063	-0.001798

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7.700	0.063	8.800	0.060	-0.002602
8.800	0.060	9.900	0.057	-0.003013
9.900	0.057	11.000	0.053	-0.003184
11.000	0.053	12.100	0.050	-0.003206
12.100	0.050	13.200	0.046	-0.003138
13.200	0.046	14.300	0.043	-0.003015
14.300	0.043	15.400	0.040	-0.002862
15.400	0.040	16.500	0.037	-0.002691
16.500	0.037	17.600	0.034	-0.002515
17.600	0.034	18.700	0.032	-0.002338
18.700	0.032	19.800	0.029	-0.002166
19.800	0.029	20.900	0.027	-0.002000
20.900	0.027	22.000	0.025	-0.001844
22.000	0.025	24.200	0.021	-0.001628
24.200	0.021	26.400	0.018	-0.001373
26.400	0.018	28.600	0.016	-0.001157
28.600	0.016	30.800	0.014	-0.000975
30.800	0.014	33.000	0.012	-0.000822
33.000	0.012	35.200	0.010	-0.000695
35.200	0.010	37.400	0.009	-0.000590
37.400	0.009	39.600	0.008	-0.000502
39.600	0.008	41.800	0.007	-0.000428
41.800	0.007	44.000	0.006	-0.000367
44.000	0.006	48.400	0.005	-0.000294
48.400	0.005	52.800	0.004	-0.000221
52.800	0.004	57.200	0.003	-0.000168
57.200	0.003	61.600	0.003	-0.000130

PASSIVE PRESSURES: Pressures below will be divided by a Factor of Safety =1.3

Z1	P1	Z2	P2	Slope
*	Below	Base		
22.00	0.000	45.00	6.729	0.292581
45.00	5.892	176.0	44.40	0.293977

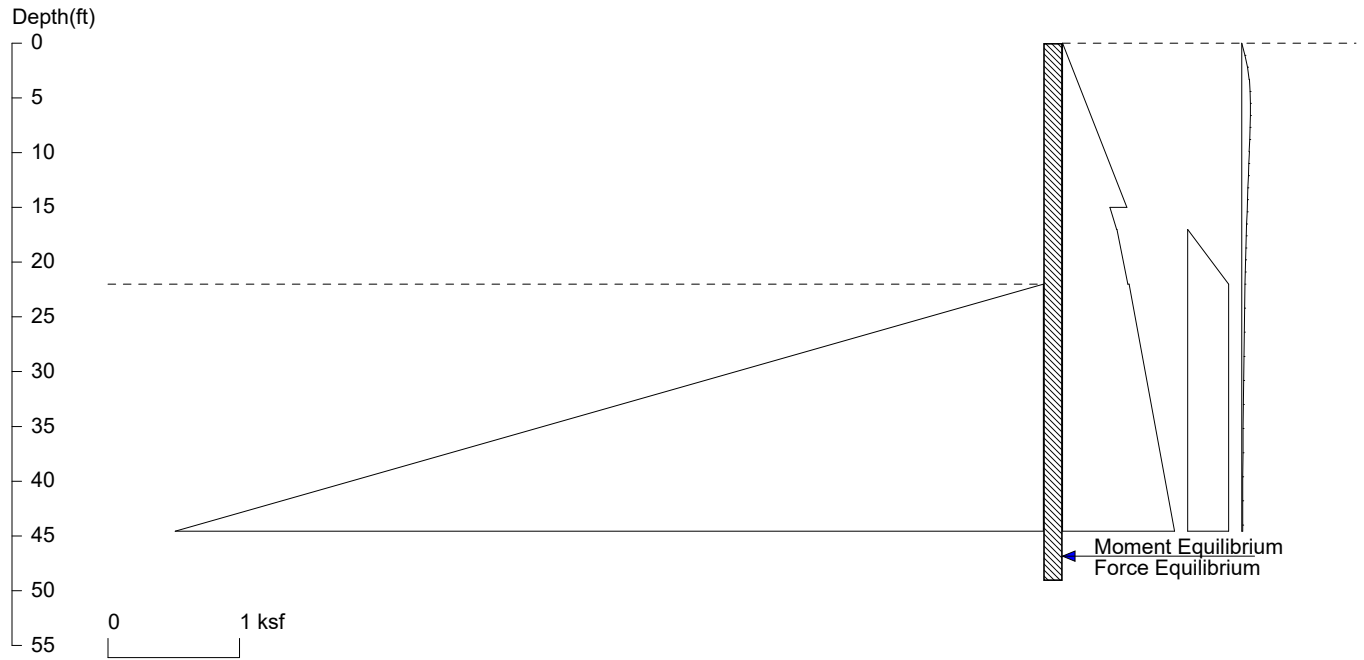
ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	4.20
2	22.00	3.00

PASSIVE SPACING:

No.	Z depth	Spacing
1	22.00	4.20

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft³; Deflection - in



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Wall Height=22.0 Pile Diameter=3.0 Pile Spacing=4.2 Wall Type: 4. Secant/Tangent

PILE LENGTH: Min. Embedment=27.10 Min. Pile Length=49.10

MOMENT IN PILE: Max. Moment=632.41 per Pile Spacing=4.2 at Depth=34.12

PILE SELECTION:

Request Min. Section Modulus = 230.0 in³/pile=3768.47 cm³/pile, F_y = 50 ksi = 345 MPa, F_b/F_y=0.66

W27X217 has Section Modulus = 627.0 in³/pile=10274.65 cm³/pile. It is greater than Min. Requirements!

Top Deflection = 0.90(in) based on E (ksi)=29000.00 and I (in⁴)/pile=8910.0

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

Z1	P1	Z2	P2	Slope
*	Above	Base		
0.000	0.000	15.00	0.490	0.032643
15.00	0.360	17.00	0.413	0.026353
17.00	0.414	22.00	0.498	0.016724
*	Below	Base		
22.00	0.506	45.00	0.859	0.015360
45.00	0.916	176.0	3.347	0.018558
*	Water	Pres.		
17.00	0.000	22.00	0.312	0.062400
22.00	0.312	176.0	0.312	0.000000
*	Sur-	charge		
0.000	0.000	1.100	0.024	0.022262
1.100	0.024	2.200	0.044	0.017726
2.200	0.044	3.300	0.057	0.011449
3.300	0.057	4.400	0.063	0.005954
4.400	0.063	5.500	0.065	0.002082
5.500	0.065	6.600	0.065	-0.000358
6.600	0.065	7.700	0.063	-0.001798

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7.700	0.063	8.800	0.060	-0.002602
8.800	0.060	9.900	0.057	-0.003013
9.900	0.057	11.000	0.053	-0.003184
11.000	0.053	12.100	0.050	-0.003206
12.100	0.050	13.200	0.046	-0.003138
13.200	0.046	14.300	0.043	-0.003015
14.300	0.043	15.400	0.040	-0.002862
15.400	0.040	16.500	0.037	-0.002691
16.500	0.037	17.600	0.034	-0.002515
17.600	0.034	18.700	0.032	-0.002338
18.700	0.032	19.800	0.029	-0.002166
19.800	0.029	20.900	0.027	-0.002000
20.900	0.027	22.000	0.025	-0.001844
22.000	0.025	24.200	0.021	-0.001628
24.200	0.021	26.400	0.018	-0.001373
26.400	0.018	28.600	0.016	-0.001157
28.600	0.016	30.800	0.014	-0.000975
30.800	0.014	33.000	0.012	-0.000822
33.000	0.012	35.200	0.010	-0.000695
35.200	0.010	37.400	0.009	-0.000590
37.400	0.009	39.600	0.008	-0.000502
39.600	0.008	41.800	0.007	-0.000428
41.800	0.007	44.000	0.006	-0.000367
44.000	0.006	48.400	0.005	-0.000294
48.400	0.005	52.800	0.004	-0.000221

PASSIVE PRESSURES: Pressures below will be divided by a Factor of Safety =1.3

Z1	P1	Z2	P2	Slope
*	Below	Base		
22.00	0.000	45.00	6.729	0.292581
45.00	5.892	176.0	44.40	0.293977

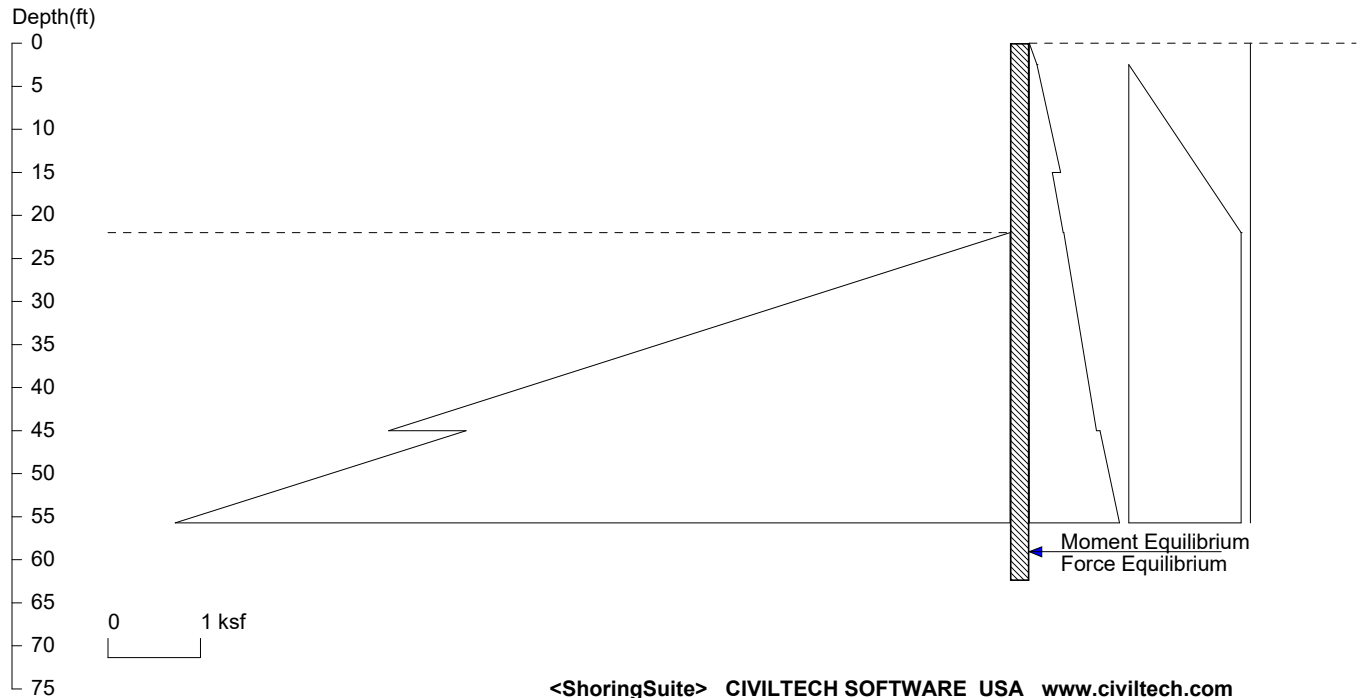
ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	4.20
2	22.00	3.00

PASSIVE SPACING:

No.	Z depth	Spacing
1	22.00	4.20

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft³; Deflection - in



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Wall Height=22.0

Pile Diameter=3.0

Pile Spacing=4.2

Wall Type: 4. Secant/Tangent

PILE LENGTH: Min. Embedment=40.45 Min. Pile Length=62.45

MOMENT IN PILE: Max. Moment=1664.17 per Pile Spacing=4.2 at Depth=40.90

PILE SELECTION:Request Min. Section Modulus = 605.2 in³/pile=9916.62 cm³/pile, $F_y = 50 \text{ ksi} = 345 \text{ MPa}$, $F_b/F_y=0.66$ W27X217 has Section Modulus = 627.0 in³/pile=10274.65 cm³/pile. It is greater than Min. Requirements!Top Deflection = 2.92(in) based on $E \text{ (ksi)}=29000.00$ and $I \text{ (in}^4\text{)}/\text{pile}=8910.0$ **DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):**

Z1	P1	Z2	P2	Slope
*	Above	Base		
0.000	0.000	2.500	0.082	0.032643
2.500	0.082	15.00	0.338	0.020546
15.00	0.248	22.00	0.366	0.016857
*	Below	Base		
22.00	0.373	45.00	0.728	0.015447
45.00	0.766	176.0	3.350	0.019728
*	Water	Pres.		
2.500	0.000	22.00	1.217	0.062400
22.00	1.217	176.0	1.217	0.000000
*	Sur-	charge		
0.000	0.000	1.100	0.000	0.000000
1.100	0.000	2.200	0.000	0.000000
2.200	0.000	3.300	0.000	0.000000
3.300	0.000	4.400	0.000	0.000000
4.400	0.000	5.500	0.000	0.000000
5.500	0.000	6.600	0.000	0.000000
6.600	0.000	7.700	0.000	0.000000

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7.700	0.000	8.800	0.000	0.000000
8.800	0.000	9.900	0.000	0.000000
9.900	0.000	11.000	0.000	0.000000
11.000	0.000	12.100	0.000	0.000000
12.100	0.000	13.200	0.000	0.000000
13.200	0.000	14.300	0.000	0.000000
14.300	0.000	15.400	0.000	0.000000
15.400	0.000	16.500	0.000	0.000000
16.500	0.000	17.600	0.000	0.000000
17.600	0.000	18.700	0.000	0.000000
18.700	0.000	19.800	0.000	0.000000
19.800	0.000	20.900	0.000	0.000000
20.900	0.000	22.000	0.000	0.000000
22.000	0.000	24.200	0.000	0.000000
24.200	0.000	26.400	0.000	0.000000
26.400	0.000	28.600	0.000	0.000000
28.600	0.000	30.800	0.000	0.000000
30.800	0.000	33.000	0.000	0.000000
33.000	0.000	35.200	0.000	0.000000
35.200	0.000	37.400	0.000	0.000000
37.400	0.000	39.600	0.000	0.000000
39.600	0.000	41.800	0.000	0.000000
41.800	0.000	44.000	0.000	0.000000
44.000	0.000	48.400	0.000	0.000000
48.400	0.000	52.800	0.000	0.000000
52.800	0.000	57.200	0.000	0.000000
57.200	0.000	61.600	0.000	0.000000
61.600	0.000	66.000	0.000	0.000000

PASSIVE PRESSURES: Pressures below will be divided by a Factor of Safety =1.3

Z1	P1	Z2	P2	Slope
*	Below	Base		
22.00	0.000	45.00	6.729	0.292581
45.00	5.892	176.0	44.40	0.293977

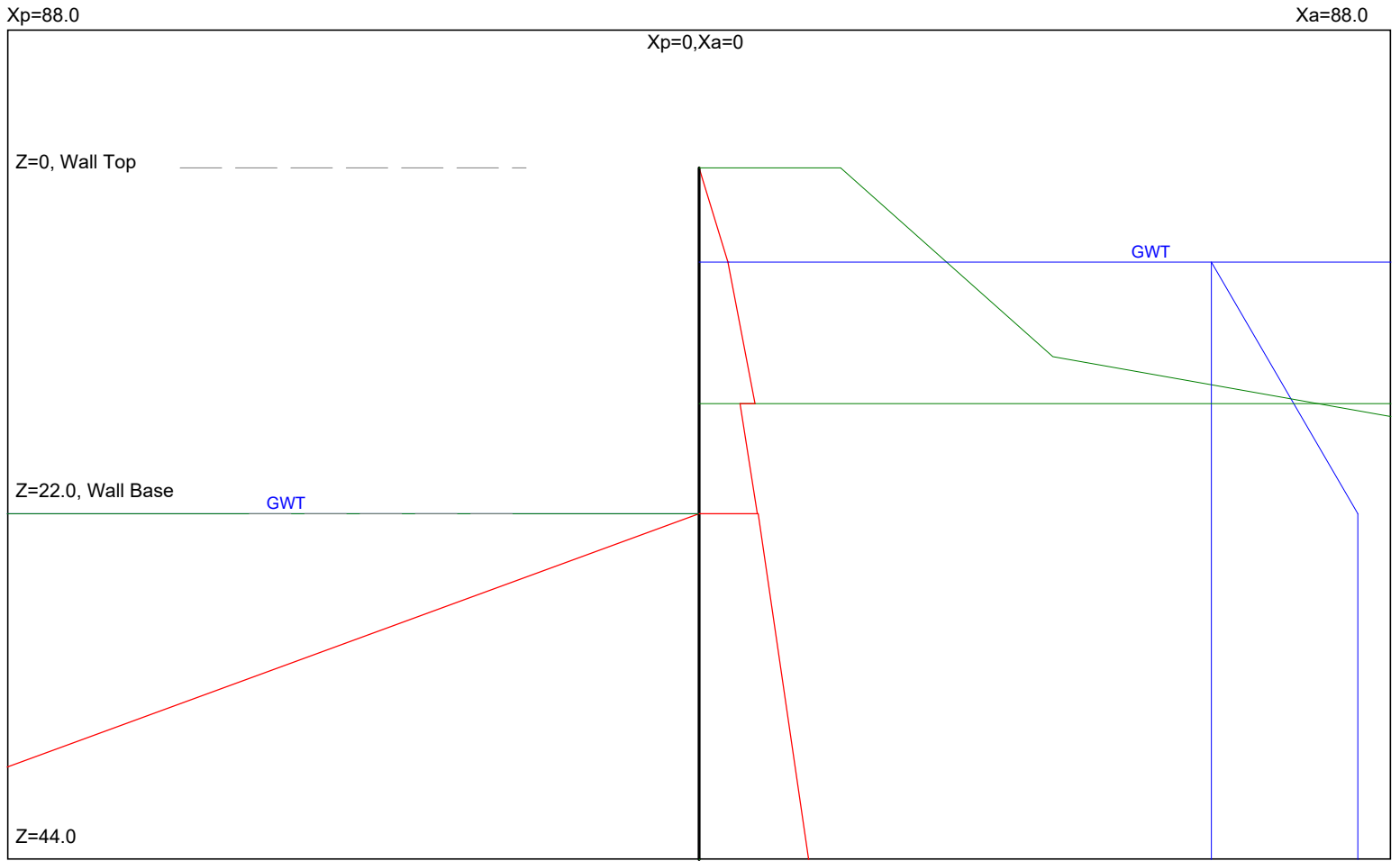
ACTIVE SPACING:

No.	Z depth	Spacing
1	0.00	4.20
2	22.00	3.00

PASSIVE SPACING:

No.	Z depth	Spacing
1	22.00	4.20

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft³; Deflection - in



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UNITS: DEPTH/DISTANCE: ft, UNIT WEIGHT: pcf, FORCE: kip/ft, PRESSURE: ksf, SLOPE: kcf

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* INPUT DATA *

Wall Height=22.0 Total Soil Types= 3

Soil No.	Weight	Saturate	Phi	Cohesion	Nspt	Type	Description
1	102.0	126.6	31	0.0	8	4	Fill
2	112.0	132.0	38.00	0.0	60	4	Sand and Silt
3	127.0	142.0	35	0.0	60	4	Glacial Till

Ground Surface at Active Side:

Line	Z1	Xa1	Z2	Xa2	Soil No.	Description
1	0.0	0.0	0.0	18.0	1	Fill
2	0.0	18.0	12.0	45.0	1	Fill
3	12.0	45.0	16.0	90.0	1	Fill
4	16.0	90.0	16.0	800.0	1	Fill
5	15.0	0.0	15.0	800.0	2	Sand and Silt
6	45.0	0.0	45.0	800.0	3	Glacial Till

Water Table at Active Side:

Point	Z-water	X-water
1	6.0	0.0
2	6.0	800.0

Ground Surface at Passive Side:

Line	Z1	Xp1	Z2	Xp2	Soil No.	Description
1	22.0	0.0	22.0	800.0	2	Sand and Silt

Water Table at Passive Side:

Point	Z-water	X-water
1	22.0	0.0
2	22.0	800.0

Wall Friction Options: 1.* No wall friction

Wall Batter Angle = 0

Apparent Pressure Conversion: 1.* Default (Terzaghi and Peck)*

Water Density = 62.4

Water Pressure: 1.* No seepage at wall tip

* OUTPUT RESULTS *

Total Force above Base= 5.55 per one linear foot (or meter) width along wall height

Total Static Force above Base= 5.55

Driving Pressure above Base - Output to Shoring - Multiplier of Pressure = 1

Z1	Pa1	Z2	Pa2	Slope	Coef.
0.00	0.00	6.00	0.20	0.0326	0.3200
6.00	0.20	15.00	0.38	0.0205	0.3200
15.00	0.28	22.00	0.40	0.0169	0.2431

Driving Pressure below Base - Output to Shoring - Multiplier of Pressure = 1

Z1	Pa1	Z2	Pa2	Slope	Ka or Ko
22.00	0.40	44.00	0.75	0.0156	0.2238

Passive Pressure below Base - Output to Shoring - Multiplier of Pressure = 1

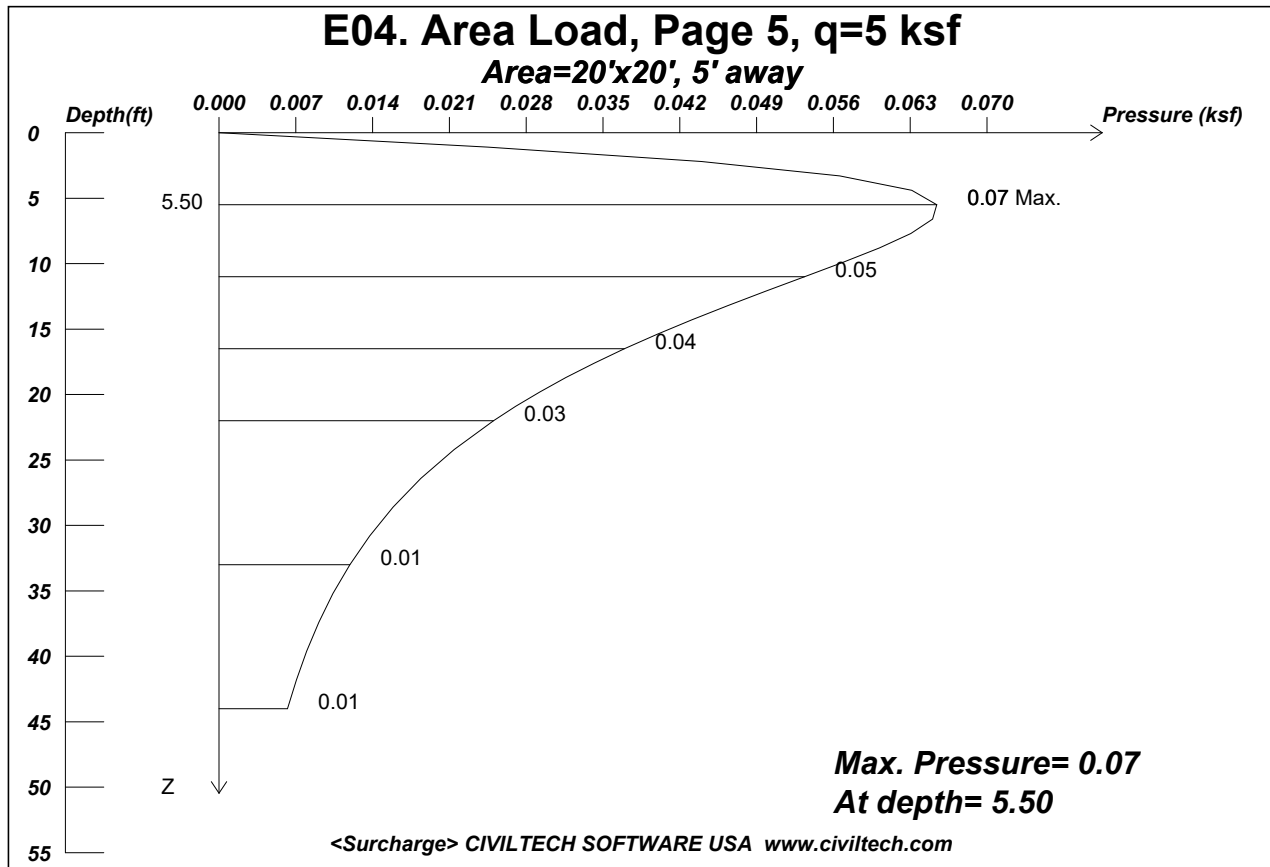
Z1	Pp1	Z2	Pp2	Slope	Kp
22.00	0.00	44.00	6.44	0.293	4.2037

Water Pressure - Output to Shoring - Multiplier of Pressure = 1

No	Z1	Pw1	Z2	Pw2	kw1
0	6.00	0.00	22.00	1.00	0.06
1	22.00	1.00	44.00	1.00	0.00

UNITS: DEPTH/DISTANCE: ft, UNIT WEIGHT: pcf, FORCE: kip/ft, PRESSURE: ksf, SLOPE: kcf

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Wall Height, $H=22$ Load Depth at Surface, $D=0$

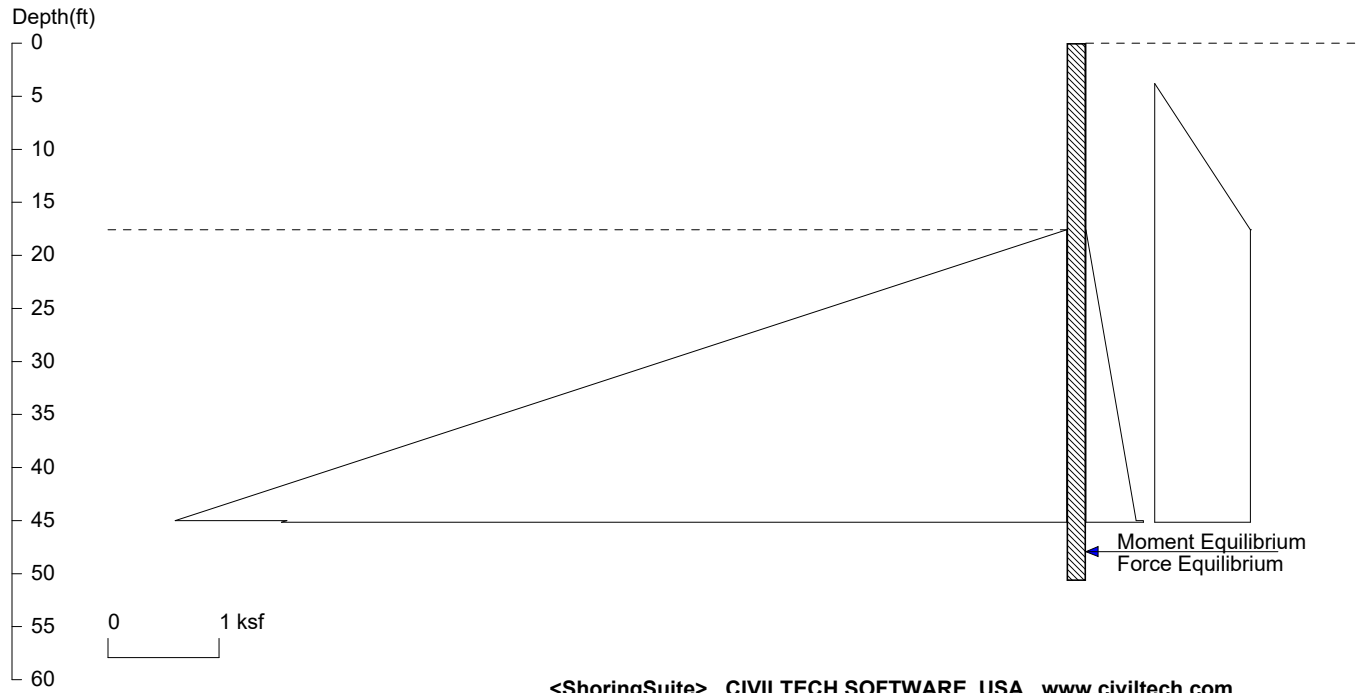
Load Factor of Surcharge Loading = 1

Rigid Wall Condition -- No movement or deflection of the wall are allowed.

Max. Pressure = 0.065 at depth = 5.50

X	Width	Length	Area Load
5.0	20.0	20.0	.25

UNITS: LENGTH/DEPTH: ft, Qpoint: kip, Qline: kip/ft, Qstrip/Qarea/PRESSURE: ksf



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Wall Height=17.6 Pile Diameter=2.0 Pile Spacing=6.7 Wall Type: 2. Soldier Pile, Drilled

PILE LENGTH: Min. Embedment=33.08 Min. Pile Length=50.68

MOMENT IN PILE: Max. Moment=659.62 per Pile Spacing=6.7 at Depth=33.11

PILE SELECTION:Request Min. Section Modulus = 239.9 in³/pile=3930.63 cm³/pile, F_y = 50 ksi = 345 MPa, F_b/F_y=0.66W18X211J has Section Modulus = 419.0 in³/pile=6866.15 cm³/pile. It is greater than Min. Requirements!Top Deflection = 1.53(in) based on E (ksi)=29000.00 and I (in⁴)/pile=4330.0**DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):**

Z1	P1	Z2	P2	Slope
*	Above	Base		
0.000	0.000	3.800	0.000	0.000000
3.800	0.000	17.600	0.000	0.000000
*	Below	Base		
17.600	0.000	45.000	0.454	0.016557
45.000	0.516	158.400	2.962	0.021568
*	Water	Pres.		
3.800	0.000	17.600	0.861	0.062400
17.600	0.861	158.400	0.861	0.000000

PASSIVE PRESSURES: Pressures below will be divided by a Factor of Safety =2

Z1	P1	Z2	P2	Slope
*	Below	Base		
17.600	0.000	45.000	8.017	0.292581
45.000	7.013	158.400	40.364	0.294101

ACTIVE SPACING:

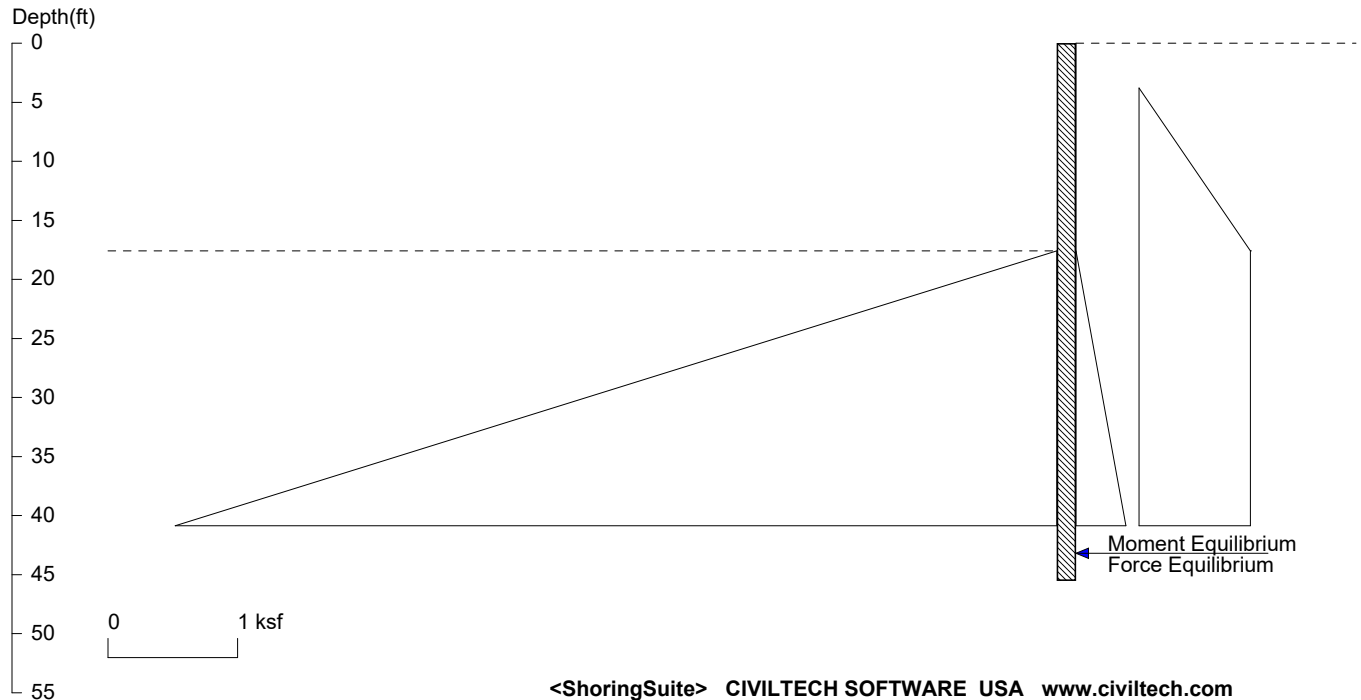
No.	Z depth	Spacing
1	0.00	6.66
2	17.60	2.00

PASSIVE SPACING:

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No.	Z depth	Spacing
1	17.60	4.00

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft³; Deflection - in



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Wall Height=17.6 Pile Diameter=2.0 Pile Spacing=6.7 Wall Type: 2. Soldier Pile, Drilled

PILE LENGTH: Min. Embedment=27.93 Min. Pile Length=45.53

MOMENT IN PILE: Max. Moment=568.50 per Pile Spacing=6.7 at Depth=30.46

PILE SELECTION:

Request Min. Section Modulus = $206.7 \text{ in}^3/\text{pile} = 3387.62 \text{ cm}^3/\text{pile}$, $F_y = 50 \text{ ksi} = 345 \text{ MPa}$, $F_b/F_y = 0.66$

W18X211J has Section Modulus = 419.0 in³/pile=6866.15 cm³/pile. It is greater than Min. Requirements!

Top Deflection = 1.16(in) based on E (ksi)=29000.00 and I (in4)/pile=4330.0

DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):

Z1	P1	Z2	P2	Slope
*	Above	Base		
0.000	0.000	3.800	0.000	0.000000
3.800	0.000	17.600	0.000	0.000000
*	Below	Base		
17.600	0.000	45.000	0.454	0.016557
45.000	0.516	158.400	2.962	0.021568
*	Water	Pres.		
3.800	0.000	17.600	0.861	0.062400
17.600	0.861	158.400	0.861	0.000000

PASSIVE PRESSURES: Pressures below will be divided by a Factor of Safety =1.5

Z1	P1	Z2	P2	Slope
*	Below	Base		
17.600	0.000	45.000	8.017	0.292581
45.000	7.013	158.400	40.364	0.294101

ACTIVE SPACING:

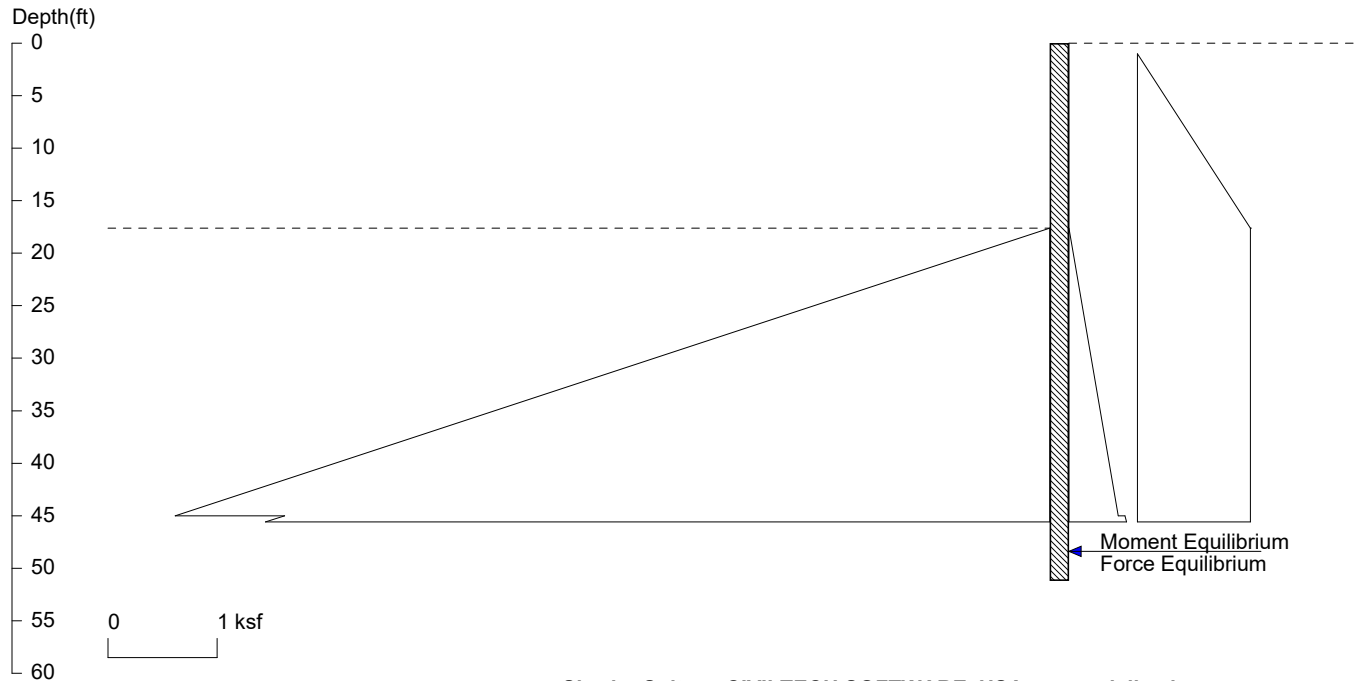
No.	Z depth	Spacing
1	0.00	6.66
2	17.60	2.00

PASSIVE SPACING:

Proposal No. 606352-136278

No.	Z depth	Spacing
1	17.60	4.00

UNITS: Width,Spacing,Diameter,Length,and Depth - ft; Force - kip; Moment - kip-ft
Friction,Bearing,and Pressure - ksf; Pres. Slope - kip/ft³; Deflection - in



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Wall Height=17.6 Pile Diameter=2.0 Pile Spacing=6.7 Wall Type: 2. Soldier Pile, Drilled

PILE LENGTH: Min. Embedment=33.60 Min. Pile Length=51.20

MOMENT IN PILE: Max. Moment=989.57 per Pile Spacing=6.7 at Depth=33.07

PILE SELECTION:Request Min. Section Modulus = 359.8 in³/pile=5896.76 cm³/pile, F_y = 50 ksi = 345 MPa, F_b/F_y=0.66W18X211J has Section Modulus = 419.0 in³/pile=6866.15 cm³/pile. It is greater than Min. Requirements!Top Deflection = 2.50(in) based on E (ksi)=29000.00 and I (in⁴)/pile=4330.0**DRIVING PRESSURES (ACTIVE, WATER, & SURCHARGE):**

Z1	P1	Z2	P2	Slope
*	Above	Base		
0.000	0.000	1.000	0.000	0.000000
1.000	0.000	17.60	0.000	0.000000
*	Below	Base		
17.60	0.000	45.00	0.454	0.016557
45.00	0.516	158.4	2.962	0.021568
*	Water	Pres.		
1.000	0.000	17.60	1.036	0.062400
17.60	1.036	158.4	1.036	0.000000

PASSIVE PRESSURES: Pressures below will be divided by a Factor of Safety =1.5

Z1	P1	Z2	P2	Slope
*	Below	Base		
17.60	0.000	45.00	8.017	0.292581
45.00	7.013	158.4	40.36	0.294101

ACTIVE SPACING:

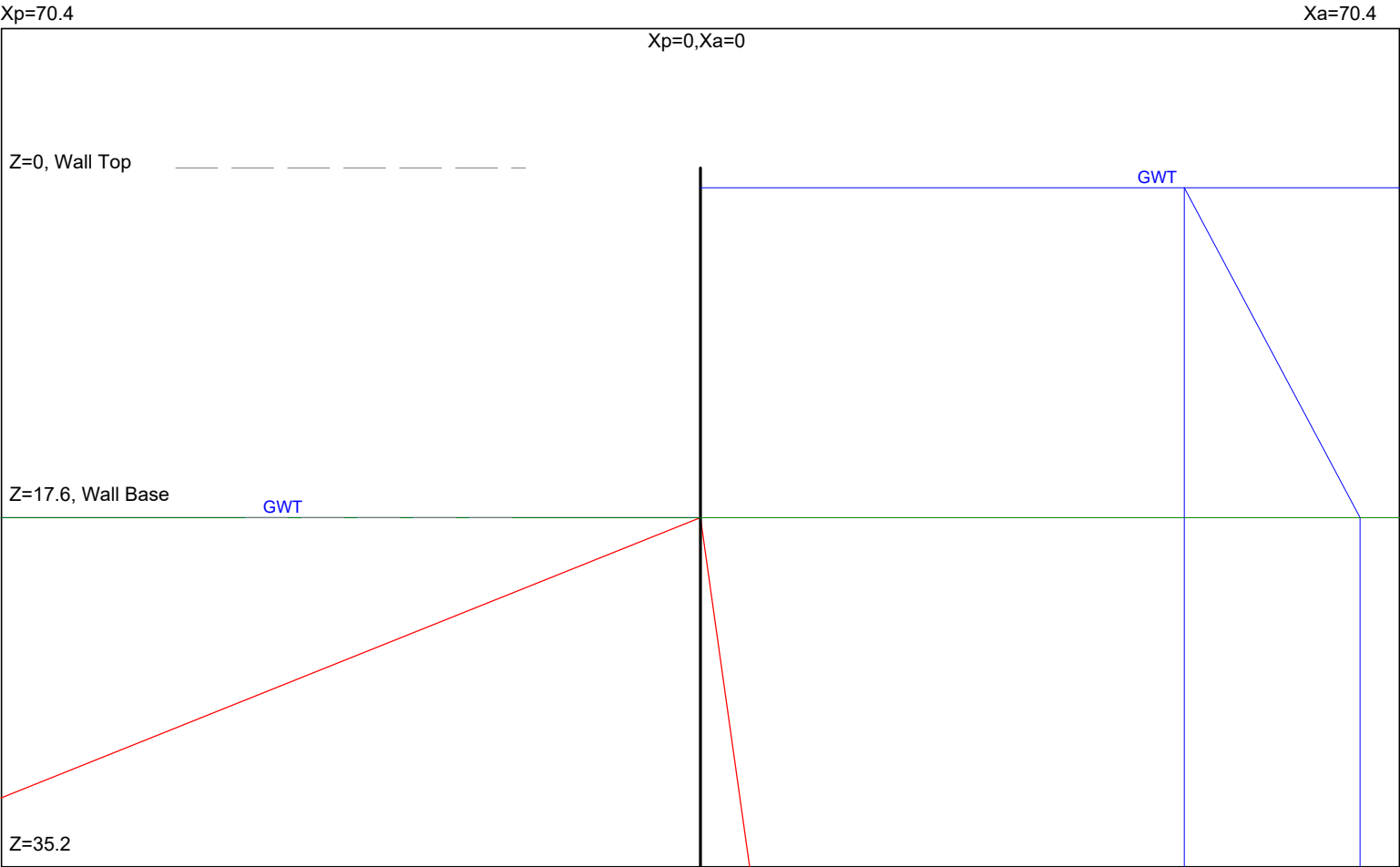
No.	Z depth	Spacing
1	0.00	6.66
2	17.60	2.00

PASSIVE SPACING:

Proposal No. 606352-136278

No.	Z depth	Spacing
1	17.60	4.00

UNITS: Width, Spacing, Diameter, Length, and Depth - ft; Force - kip; Moment - kip-ft
Friction, Bearing, and Pressure - ksf; Pres. Slope - kip/ft³; Deflection - in



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UNITS: DEPTH/DISTANCE: ft, UNIT WEIGHT: pcf, FORCE: kip/ft, PRESSURE: ksf, SLOPE: kcf

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*** INPUT DATA ***

Wall Height=17.6 Total Soil Types= 3

Soil No.	Weight	Saturate	Phi	Cohesion	Nspt	Type	Description
1	102.0	126.6	31	0.0	8	4	Fill
2	112.0	132.0	38	0.0	60	4	Sand and Sil
3	127.0	142.0	35	0.0	60	4	Glacial Till

Ground Surface at Active Side:

Line	Z1	Xa1	Z2	Xa2	Soil No.	Description
1	17.6	0.0	17.6	800.0	2	Sand and Sil
2	45.0	0.0	45.0	800.0	3	Glacial Till

Water Table at Active Side:

Point	Z-water	X-water
1	1.0	0.0
2	1.0	800.0

Ground Surface at Passive Side:

Line	Z1	Xp1	Z2	Xp2	Soil No.	Description
1	17.6	0.0	17.6	800.0	2	Sand and Sil
2	45.0	0.0	45.0	800.0	3	Glacial Till

Water Table at Passive Side:

Point	Z-water	X-water
1	17.6	0.0
2	17.6	800.0

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Wall Friction Options: 1.* No wall friction

*** OUTPUT RESULTS ***

Total Force above Base= 0.00 per one linear foot (or meter) width along wall height

Total Static Force above Base= 0.00

Driving Pressure above Base - Output to Shoring - Multiplier of Pressure = 1

Z1	Pa1	Z2	Pa2	Slope	Coef.
0.00	0.00	1.00	0.00	0.0000	0.0000
1.00	0.00	17.60	0.00	0.0000	0.0000

Driving Pressure below Base - Output to Shoring - Multiplier of Pressure = 1

Z1	Pa1	Z2	Pa2	Slope	Ka or Ko
17.60	0.00	35.20	0.29	0.0166	0.2379

Passive Pressure below Base - Output to Shoring - Multiplier of Pressure = 1

Z1	Pp1	Z2	Pp2	Slope	Kp
17.60	0.00	35.20	5.15	0.293	4.2037

Water Pressure - Output to Shoring - Multiplier of Pressure = 1

No	Z1	Pw1	Z2	Pw2	kw1
0	1.00	0.00	17.60	1.04	0.06
1	17.60	1.04	35.20	1.04	0.00

UNITS: DEPTH/DISTANCE: ft, UNIT WEIGHT: pcf, FORCE: kip/ft, PRESSURE: ksf, SLOPE: kcf

Date: 12/17/2024 File Name: Z:\JOBS\22 Jobs\22147.02 MassDOT Wareham Mill Pond Dam-MA\CALCS\Cofferdam Phasing\Final\Shoring Suite\Seca



Engineers | Scientists | Planners

PARECORP.COM



MEMORANDUM

DATE: December 14, 2023

TO:

CC:

FROM: Sean Markey, E.I.T., Pare Corporation
Brian Nehring, P.E., Pare Corporation

RE: Cofferdam Phasing Geotechnical Feasibility Study
Rehabilitation of Cranberry Highway (Route 28) / Mill Pond Dam over Agawam River
Wareham, MA
(Pare Project No. 22147.02)

Project Background

Pare understands that the sum of the conditions listed in geotechnical report have been modified. The proposed drawdown of Mill Pond will no longer be applied due to public concerns. Pare has been requested to review the following geotechnical considerations for the proposed cofferdam phasing:

- Impoundment is not permitted to be drawn down as recommended in the project geotechnical report.
- Proposed cofferdam to be reviewed with higher water level.
- Proposed steel sheeting along the center line of the existing dam to be reviewed for stability, seepage, and installation feasibility.
- Construction considerations for dewatering.
- Review geotechnical implications for removal of flow by-pass channels through existing primary and auxiliary spillway without drawdown of Mill Pond.

Alternatives Study

Pare has been requested to perform an alternatives study on the performance of a cofferdam/support of excavation design to provide general excavation support and control seepage during construction. Information provided in the 2021 Geotechnical Report, dated December 2021 and revised September 2023, was used for the feasibility study. Three conditions were analyzed using GeoStudio 2021 R2 SEEP/W and SLOPE/W programming:

1. Existing Conditions
2. Proposed Sheet Pile
3. Shallow Refusal Sheet Pile



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HOLYOKE, MA 01040
413.507.3448

Analysis 1 - Existing Conditions:

The Existing Conditions analysis was performed to confirm the parameters and site conditions used in all three analyses are representative of those encountered during the subsurface investigations. To do so, a cross-section (perpendicular to the sheet pile length) was drawn as the representative section for analysis. The cross-section location is illustrated in Figure 1 below. Borings included in the analysis are borings B-1, B-3, BB17-1, and BB17-3. Depths of soil strata were chosen based on the selected borings and the Subsurface Profile provided in the Figures Section of the previous Phase II and Geotechnical Reports (2009, 2021). Elevations were based off the MassDOT 'Boring Plan of Proposed Bridge' provided in the Figures Section of the Geotechnical Report and the Existing Condition Drawing on Sheet 15 of the MassDOT 'Sketch Plans of Proposed Bridge.' Soil parameter values were selected from the 2021 Geotechnical Report, Holtz and Kovacs *An Introduction to Geotechnical Engineering* (1981), and ASAE Vol.25, No. 5 Estimation of Soil Water Properties (1982). "Drainage" surfaces were selected across the spillway side of dam. Soil parameters and existing boundary conditions were adjusted during the analysis in attempt to match the water pressure head at 0 feet to the encountered water level readings in borings B-1 and B-3 from the 2021 Geotechnical Report. Model was able to match the water pressure head at 0 feet to the water level reading in boring B-3, but not in boring B-1.

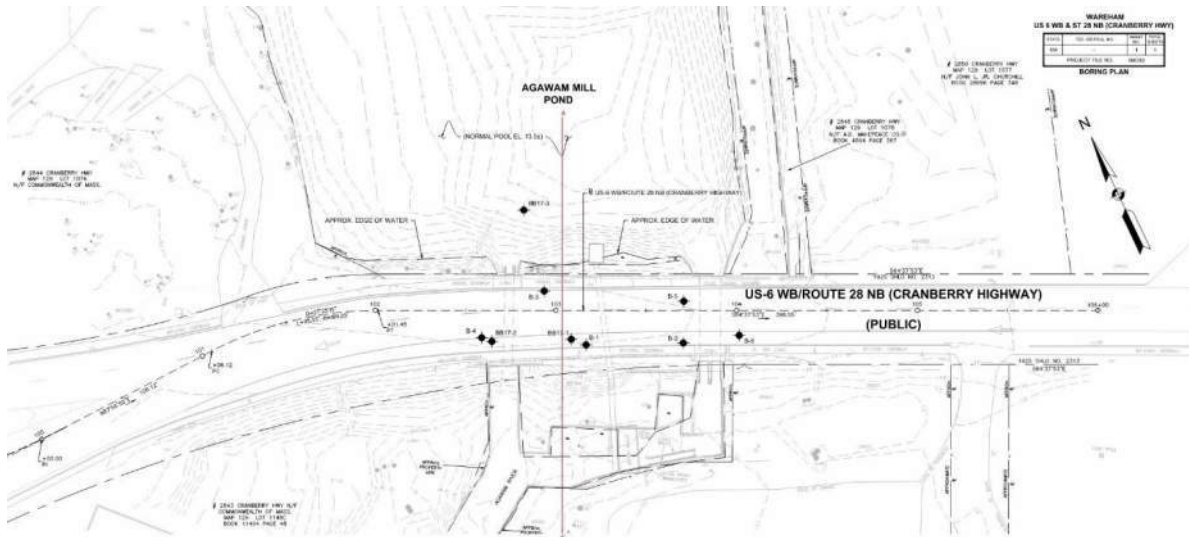


Figure 1. Cross Section A-A' Location

Existing Conditions Soil Parameters

TABLE 1: GEOSTUDIO 2021 R2 SOIL PARAMETERS								
Material	Material Model	Vol. Water Content Function Estimation Method	Saturated Water Content	Diameter (mm) at 10% passing ¹	Diameter (mm) at 60% passing ¹	Liquid Limit (%) ¹	Saturated K _x (ft/sec)	Residual Water Content
FILL	Saturated / Unsaturated	Grain-Size Data ²	0.1284	0.05	0.58	0	4.218E-4	0.041
POND MUCK	Saturated / Unsaturated	Sample Functions (Sand)	0.28	NA	NA	NA	6.63E-3	0.02
SAND & SILT	Saturated / Unsaturated	Grain-Size Data ²	0.1129	0.2	0.6	0	1.755E-3	0.035



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TABLE 1: GEOSTUDIO 2021 R2 SOIL PARAMETERS								
Material	Material Model	Vol. Water Content Function Estimation Method	Saturated Water Content	Diameter (mm) at 10% passing ¹	Diameter (mm) at 60% passing ¹	Liquid Limit (%) ¹	Saturated K _x (ft/sec)	Residual Water Content
GLACIAL TILL	Saturated / Unsaturated	Sample Functions (Sand)	0.25	NA	NA	NA	6.63E-3	0.02
WEATHERED BEDROCK ³	Saturated / Unsaturated	Sample Functions (Sand)	0.25	NA	NA	NA	6.63E-3	0.02
BEDROCK	Saturated Only	NA	0	NA	NA	NA	1.0E-9	NA

1. Only used when Grain-Size Data estimation method is selected.

2. Sieve analysis results from Pare Geotechnical Report #14199.14 & #22147.02 used for input.

3. In accordance with Pare's Geotechnical Report #14199.14 & #22147.02, "Weathered Bedrock" and "Glacial Till" assumed to have similar physical properties.

'Van Genuchten' was selected as the estimation method for the Hydraulic Conductivity Function for each material. Ky'/K_x' Ratio for all materials was selected as 1.

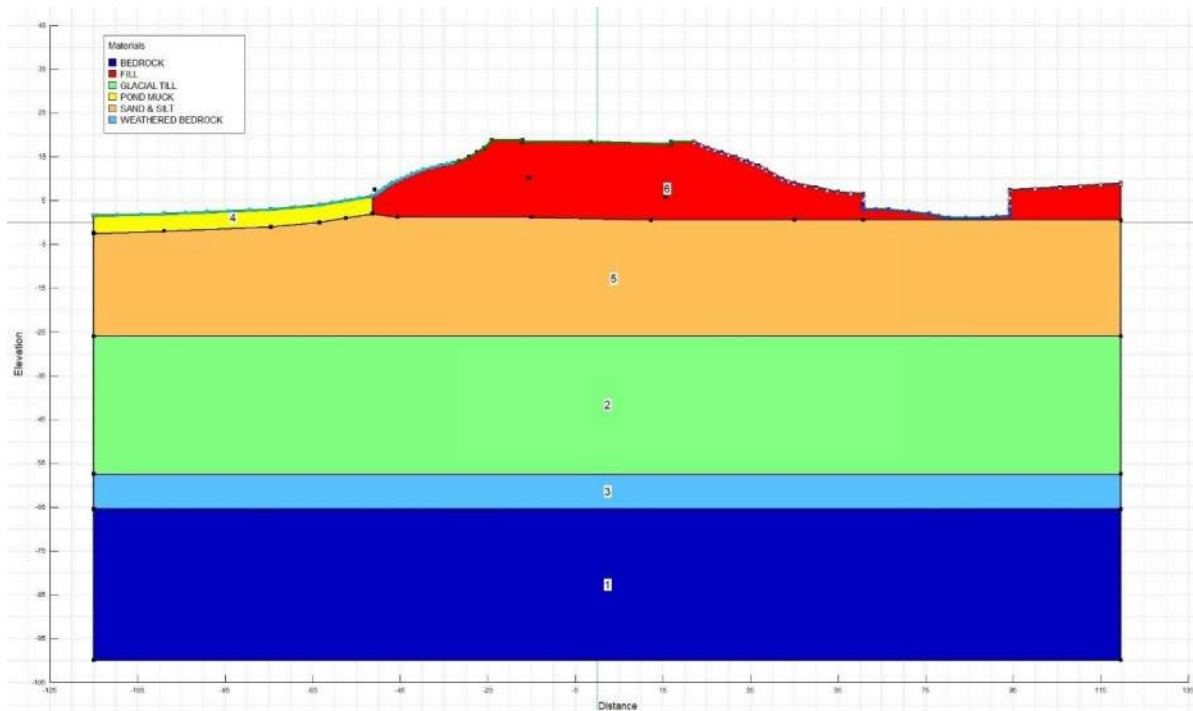


Figure 2. Existing Conditions

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Results:

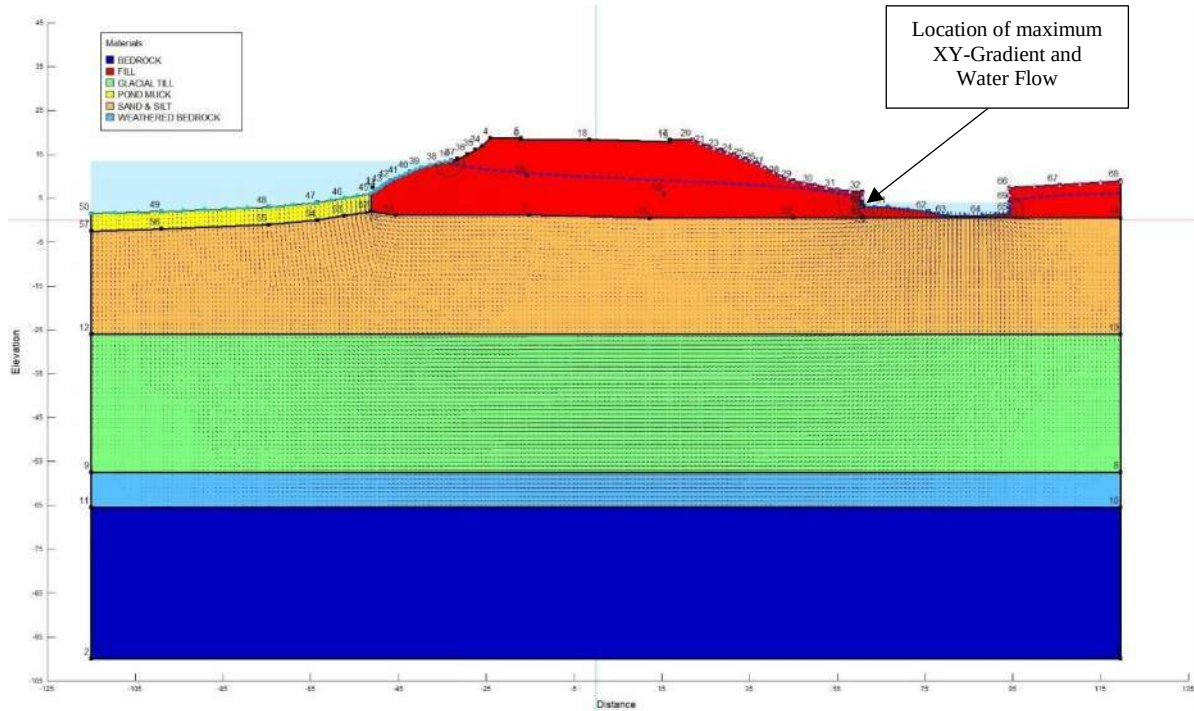


Figure 3. Existing Conditions Results

TABLE 2: GEOSTUDIO 2021 R2 EXISTING CONDITION RESULTS		
Maximum XY-Gradient	Max Water Flow Rate at a Single Point (ft ³ /sec per ft)	Water Influx (gal/min)
1.02874	6.54182E-4	55

Assumptions:

- Concrete culverts/spillways not completely impervious and affect local groundwater readings during drilling operations; therefore, borings next to culverts/spillways with suspicious groundwater readings not chosen for cross section calibration (e.g. down stream water levels higher in dam borings than those drilled closer to the pond).
- Groundwater level reading for boring B-1 assumed not to be reliable in analysis calibration due to wash and drive drilling methods.
- “Pond Muck” stratum 4 feet in depth across analysis area, in accordance with what was encountered in boring BB17-3.
- “Fill” stratum stretches laterally across existing dam structures and associate appurtenances in excavated direction. “Fill” stratum stretches laterally to Mill Pond to where slope of ground surface decreases and not past boring BB17-3 where it was not encountered.

- Existing spillway and fish channel concrete slabs and walls not impervious in analysis due to weathering and corrosion.
- Pond water level set at “Normal Pool” elevations of 13.6 feet.
- “Weathered Bedrock” and “Glacial Till” have similar physical properties, in accordance with the 2009 Phase II and 2021 Geotechnical Report.

Restrictions:

- Subsurface investigations performed in 2008, 2010, 2017, and 2018: groundwater levels highly variable depending on seasonal weather and precipitation levels; different loggers and drilling consultants with their own classifications/descriptions.
- Wash and Drive drilling methods pump water into borings, so groundwater readings at the end of drilling may not be representative of actual groundwater conditions.
- Analysis assumes consistent soil conditions throughout soil strata.
- Concrete culverts neglected during analysis for the three conditions tested.
- “Boulder” and “Wood” soil strata encountered during subsurface investigations neglected during analysis due to inconsistencies across borings (not assumed continuous across existing dam structure).



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Analysis 2 - Proposed Sheet Pile Design:

The proposed sheet pile installation depth was estimated, rather than calculated for constructability. The proposed sheet pile was estimated to be installed to a depth below the excavation surface that is twice the length of the exposed length of pile. The bottom of the sheet pile was designed to an elevation of -47 feet NAVD88. A cofferdam was installed at 76 feet downstream from the sheet pile to restrict the surficial flowing water to enter the excavation. The bottom of the cofferdam was installed to a depth of -25 feet NAVD88. Impervious Boundaries were set on all surfaces of the sheet pile and cofferdam. Soil Parameters from the Existing Conditions Analysis were applied to the Proposed Sheet Pile Design. The following parameters were selected for the sheet pile material:

TABLE 3: GEOSTUDIO 2021 R2 SHEET PILE PARAMETERS								
Material	Material Model	Vol. Water Content Function Estimation Method	Saturated Water Content	Diameter (mm) at 10% passing	Diameter (mm) at 60% passing	Liquid Limit (%)	Saturated Kx (ft/sec)	Residual Water Content
SHEET PILE	Saturated Only	NA	0	NA	NA	NA	1.0E-10	NA

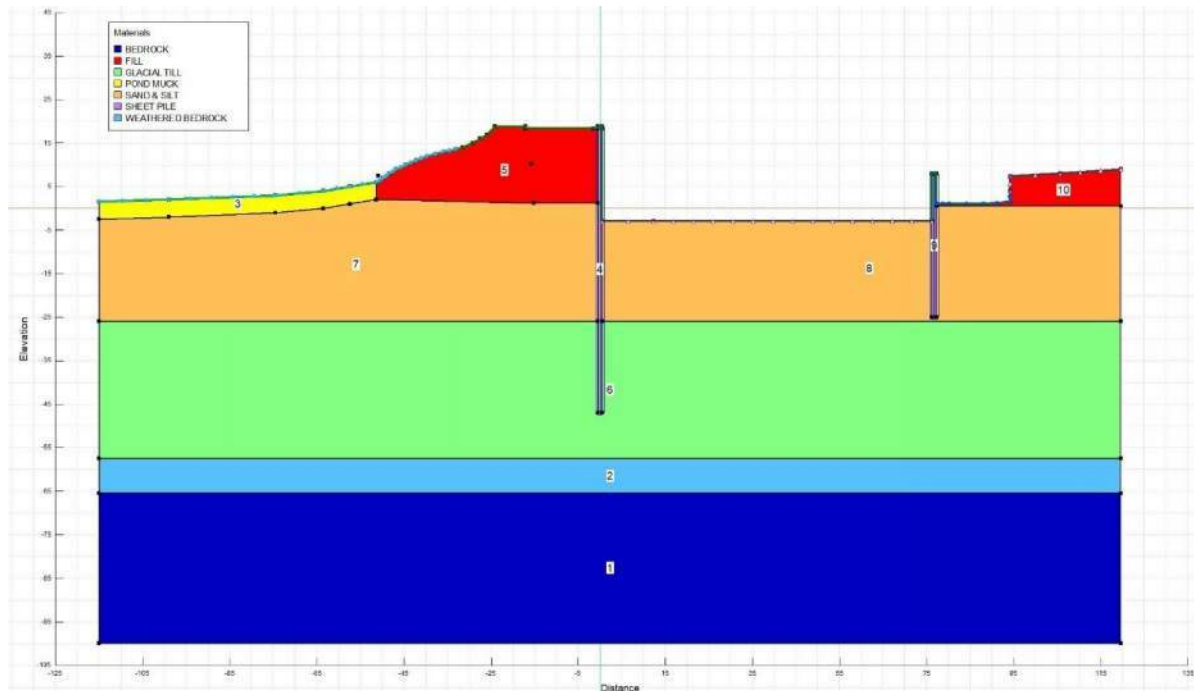


Figure 4. Proposed Sheet Pile Design Conditions

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Results:

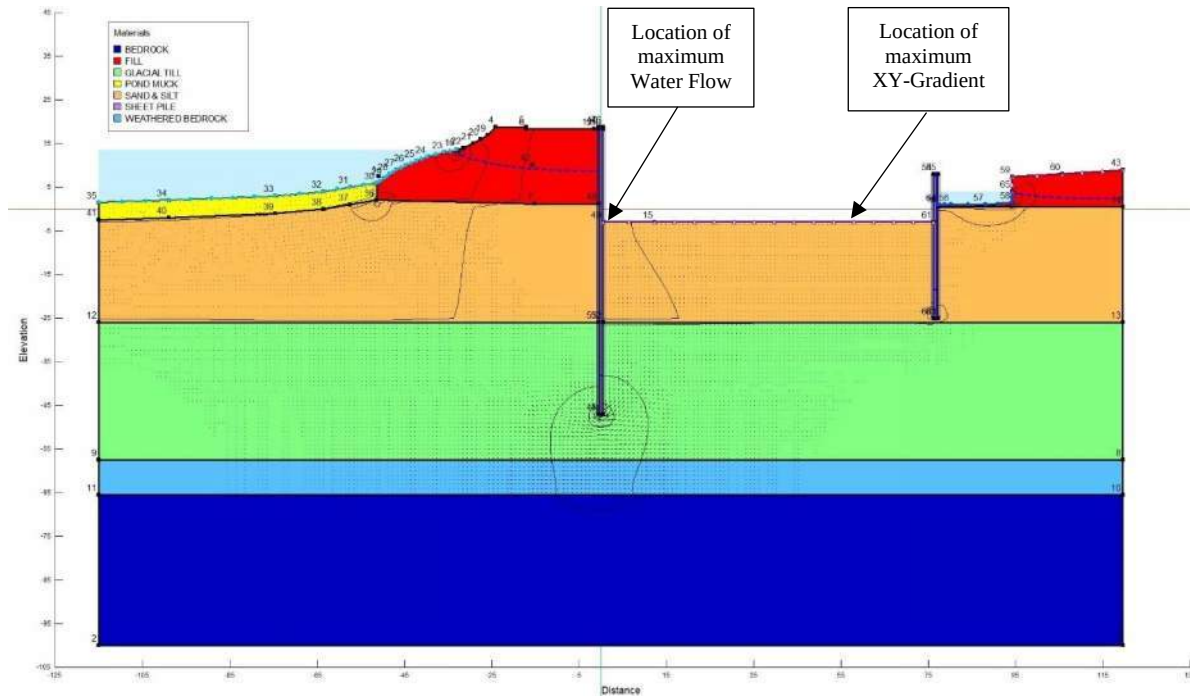


Figure 5. Proposed Sheet Pile Design Results

TABLE 4: GEOSTUDIO 2021 R2 PROPOSED SHEET PILE DESIGN RESULTS		
Maximum XY-Gradient	Max Water Flow Rate at a Single Point (ft ³ /sec per ft)	Water Influx Across Entire Excavation (gal/min)
0.201209	3.47362E-4	2150

Assumptions:

- Sheet piles will be able to push aside boulders, obstructions, and split-spoon sampler refusal materials to reach an installation elevation of -4.5 feet NAVD88.
- Sheet piles are completely impervious.
- Excavation anticipated to reach an elevation of -3 feet NAVD88, or 1 foot below the bottom of foot elevation, in accordance with the MassDOT ‘Sketch Plans of Proposed Bridge.’
- Excavation to -3 feet NAVD88 to extend at least 103 feet horizontally downstream from the sheet pile, and about 255 feet parallel to the sheet pile (from the Key Plan in MassDOT’s “Sketch Plans of Proposed Bridge”).
- Applicable assumptions from Analysis 1 – Existing Conditions are applied to Analysis 2.

Restrictions:

◆

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- Stability analysis was not performed but can be done if additional time allows. Only SEEP/W analysis was performed.
- Analysis was only performed for Phase 1 design of the Phased Bridge Construction.
- Applicable restrictions from Analysis 1 – Existing Conditions are applied to Analysis 2.



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Analysis 3 - Shallow Refusal Sheet Pile Design:

During the subsurface investigations, split-spoon sampling refusal was encountered at depths shallower than the anticipated required sheet pile depth. In our experience, sheet pile installation may hit refusal at depths where drilling split-spoon sampling refusal was encountered. We anticipate sheet piles to encounter refusal at elevations of about -4.5 feet NAVD88 (where boring BB17-1 encountered blow count of $n=107/4''$), or shallower. Therefore, we performed a third analysis of the anticipated construction scenario. Previous conditions from the Proposed Sheet Pile Design and Existing Conditions were applied, except that the sheet pile was installed to an elevation of -4.5 feet NAVD88, instead of -47 feet NAVD88.

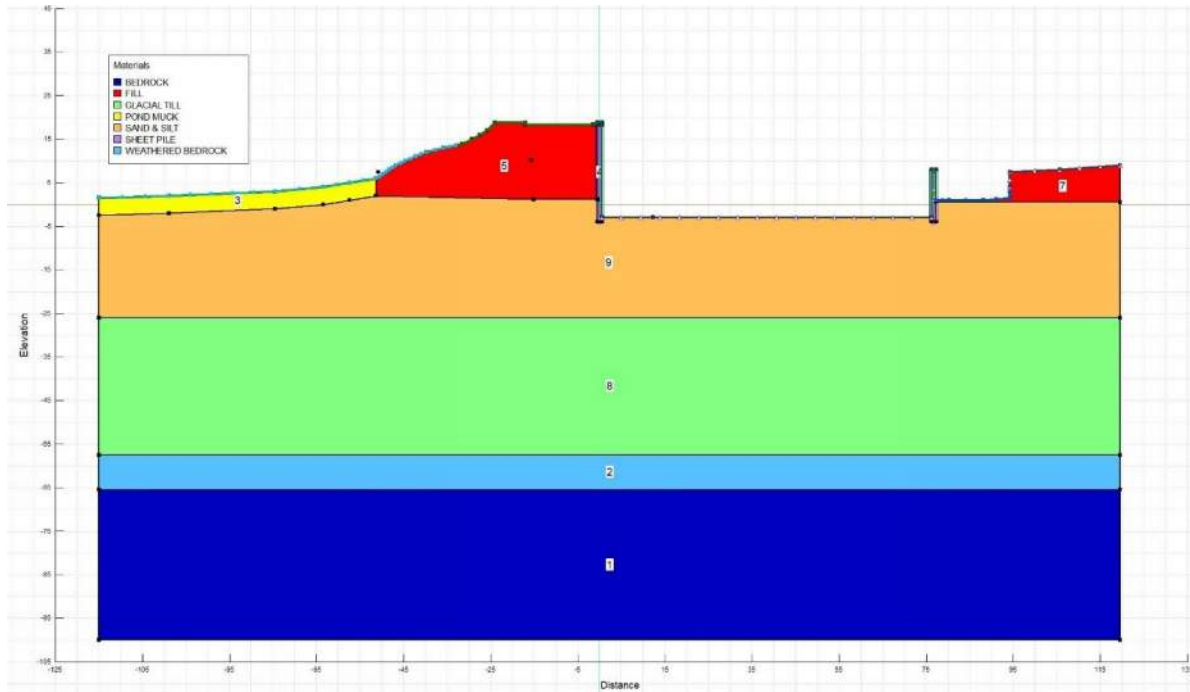


Figure 6. Shallow Refusal Sheet Pile Design Conditions

Results:

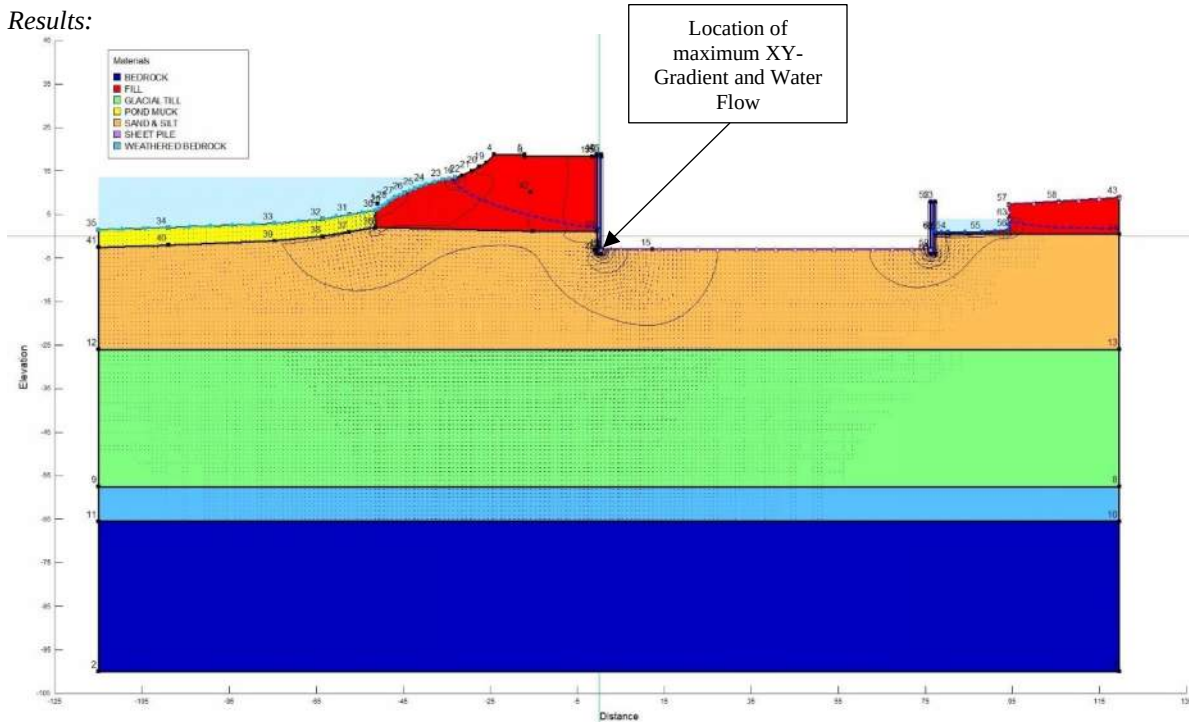


Figure 7. Shallow Refusal Sheet Pile Design Results

TABLE 5: GEOSTUDIO 2021 R2 SHALLOW REFUSAL SHEET PILE DESIGN RESULTS		
Maximum XY-Gradient	Max Water Flow Rate at a Single Point (ft ³ /sec per ft)	Water Influx Across Entire Excavation (gal/min)
2.81715	1.86925E-3	3375

Assumptions:

- Sheet piles will be able to push aside boulders and obstructions to reach an installation elevation of -4.5 feet NAVD88.
- Applicable assumptions from Analysis 1 – Existing Conditions and Analysis 2 – Proposed Sheet Pile Design are applied to Analysis 3.

Restrictions:

- Applicable restrictions from Analysis 1 – Existing Conditions and Analysis 2 – Proposed Sheet Pile Design are applied to Analysis 3.

◆

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Discussion:

The preliminary phasing review utilized GeoStudio 2021 R2 program analysis which only tests a single cross-section line through the plans. The calculations assume for the purposes of this review that this is the critical cross-section for the proposed structure. Additional sections should be analyzed dependent upon final design and site variability.

The following is a list of site conditions that may impact the installation of the proposed phasing that should be reviewed by the project team:

- Steel sheeting capable of breaking through existing concrete culverts.
- If very dense soils were accurately classified as till or are weathered bedrock.

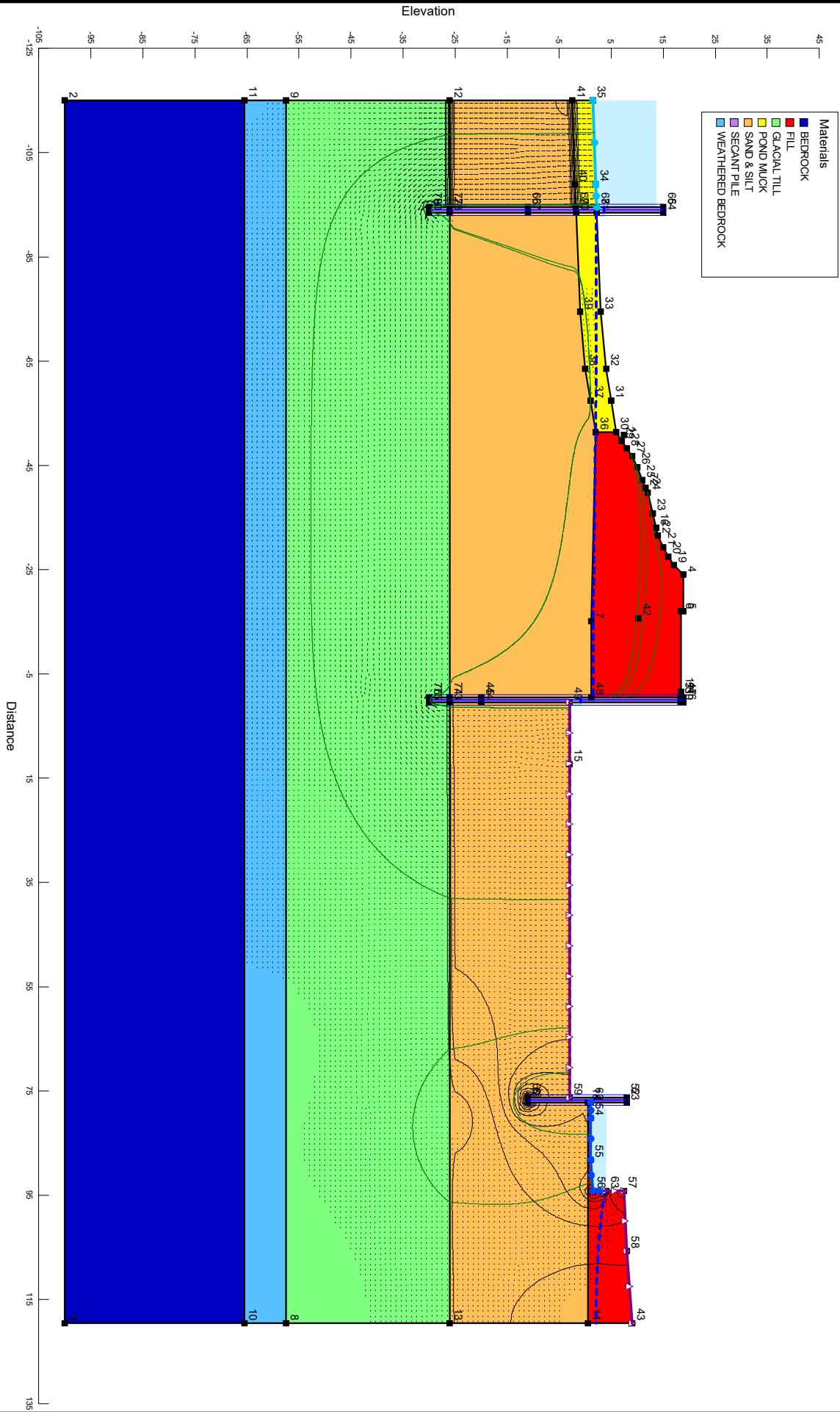
Pare anticipates sheet piles will hit shallow refusal during installation due to the boulders and very dense Sand & Silt and Glacial Deposits encountered. Shallow refusal will result in a larger quantity of water influow into the excavation, and thus require larger dewatering efforts. The shallow refusal will result in an anticipated lower quality of stability and support of excavation feasibility. Additional measures, such as secant piles, soldier piles, and/or soil nails may be necessary to help support the excavation from slope failure and seepage.



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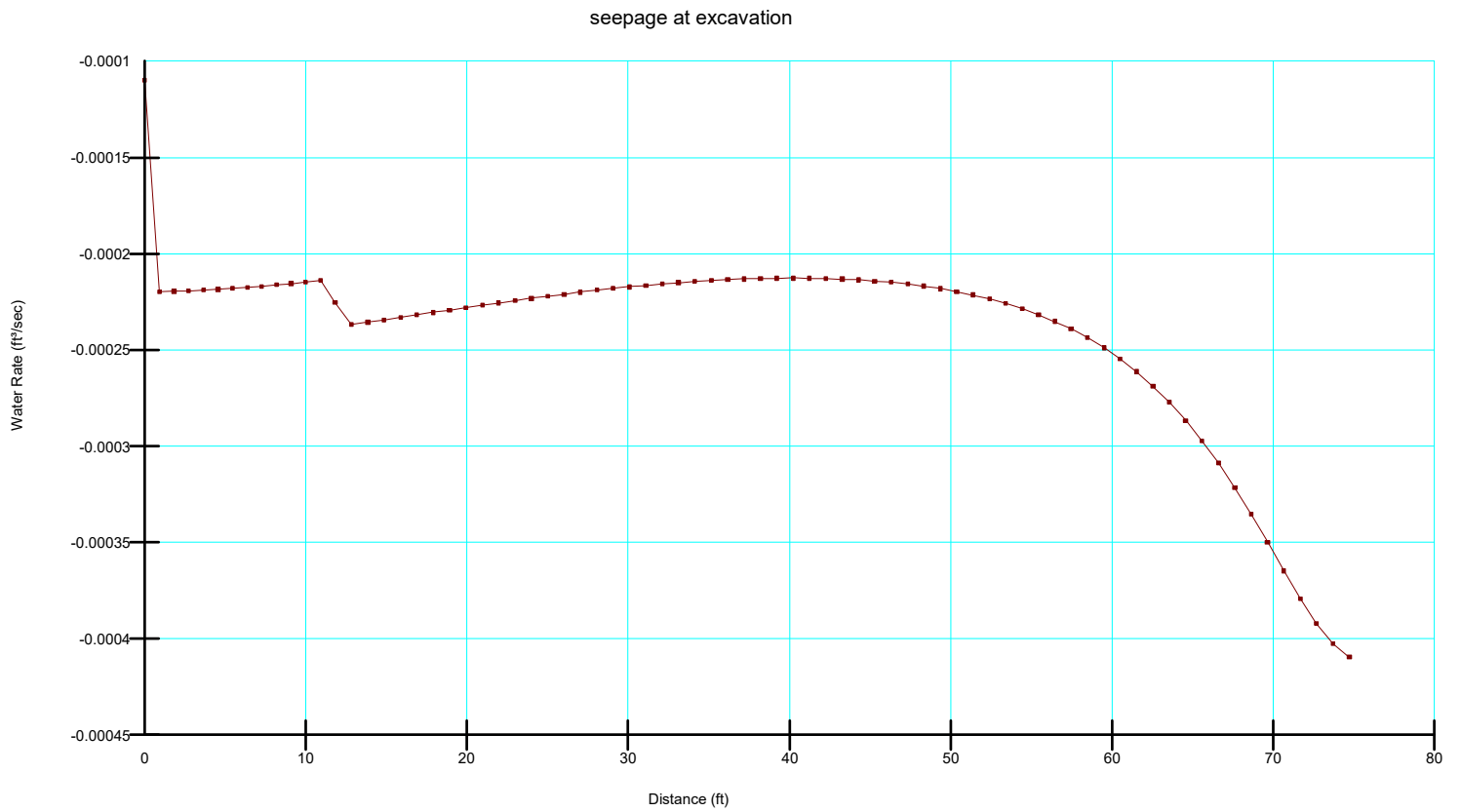


Steady-State Seepage (8)

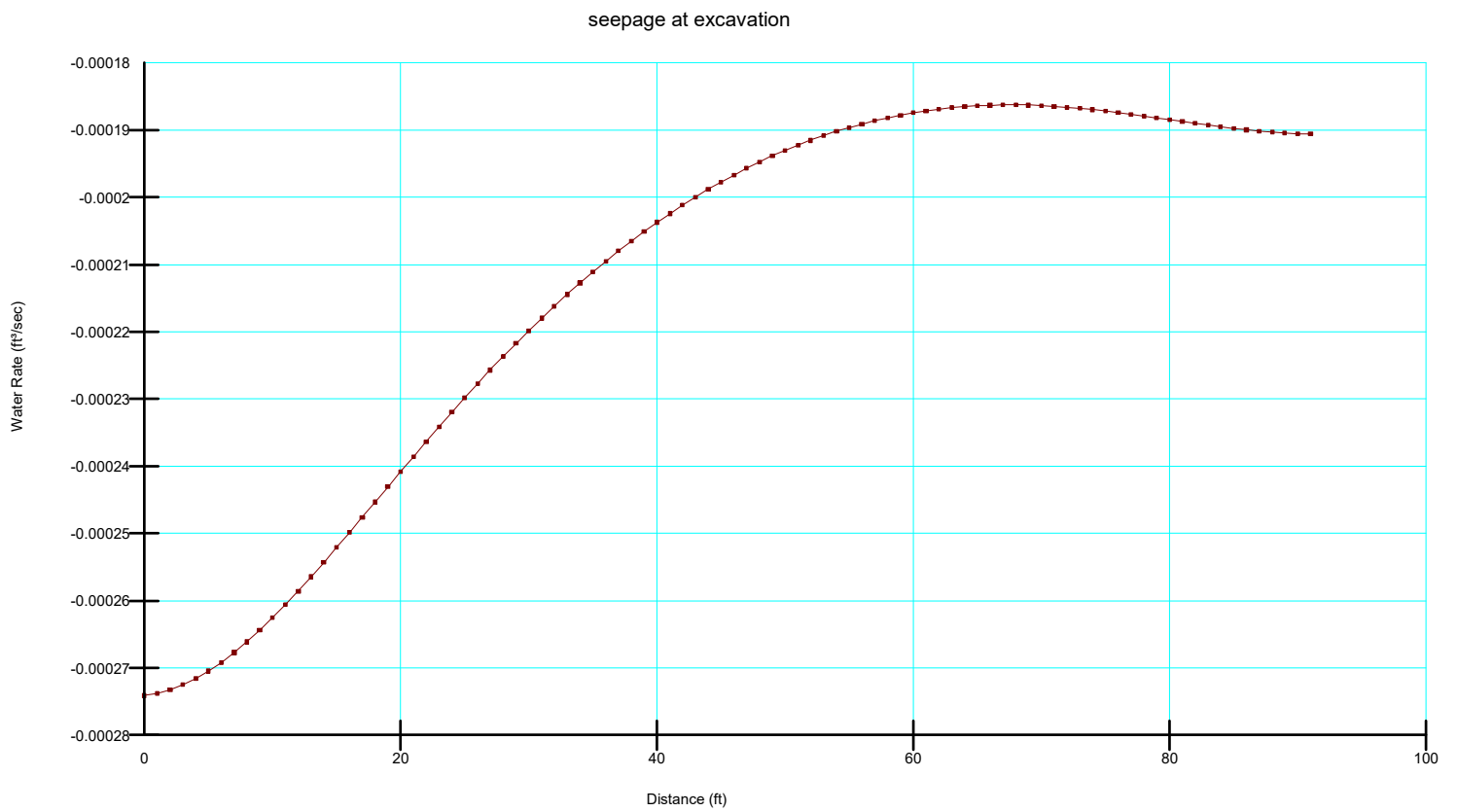
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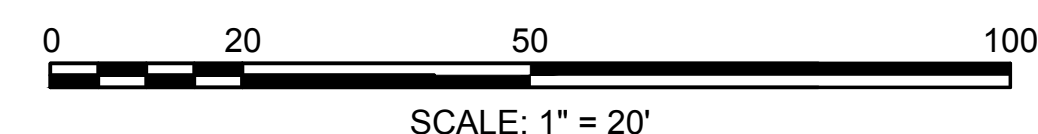
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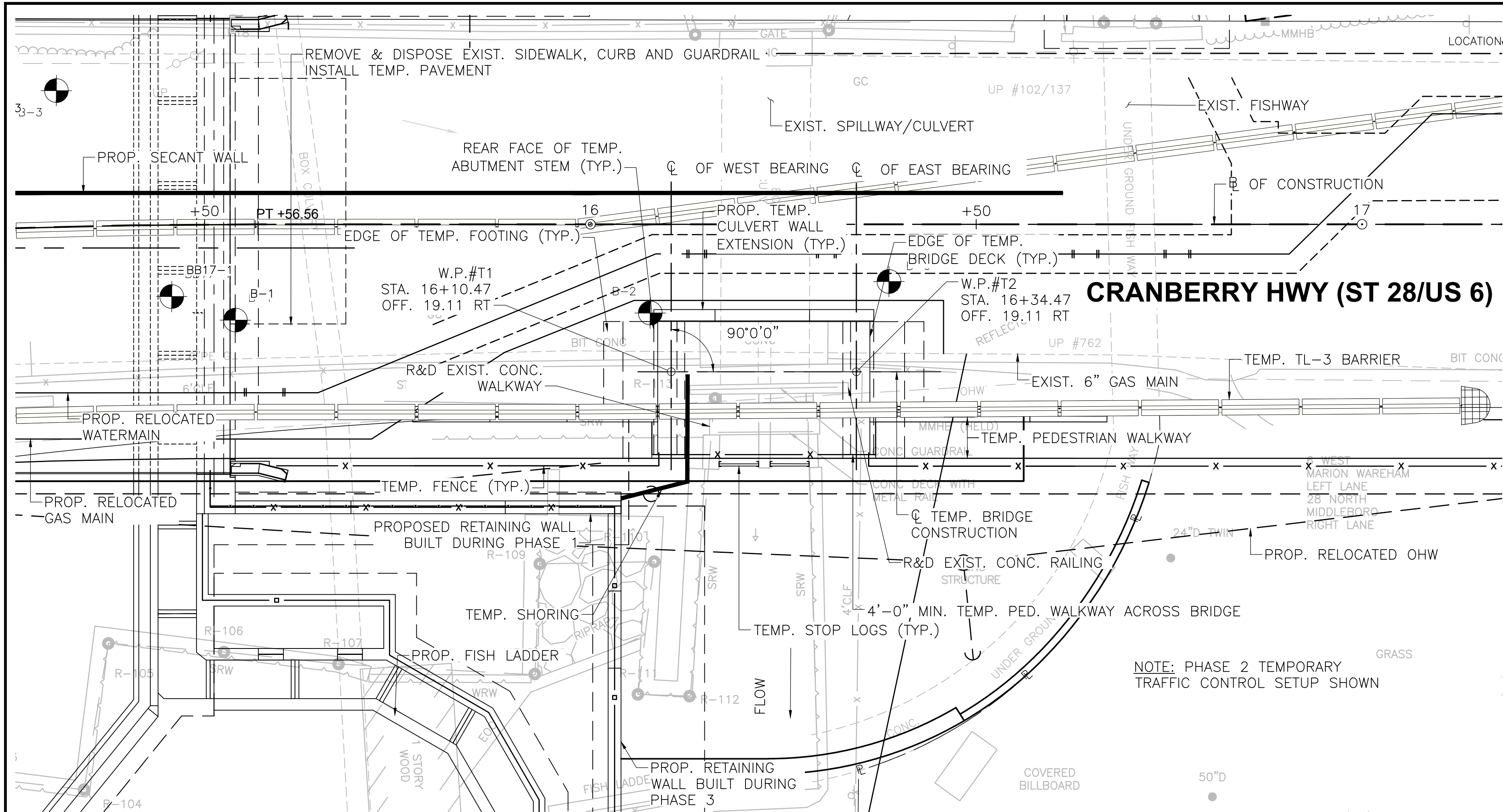






CONSTRUCTION PLAN NO. 1

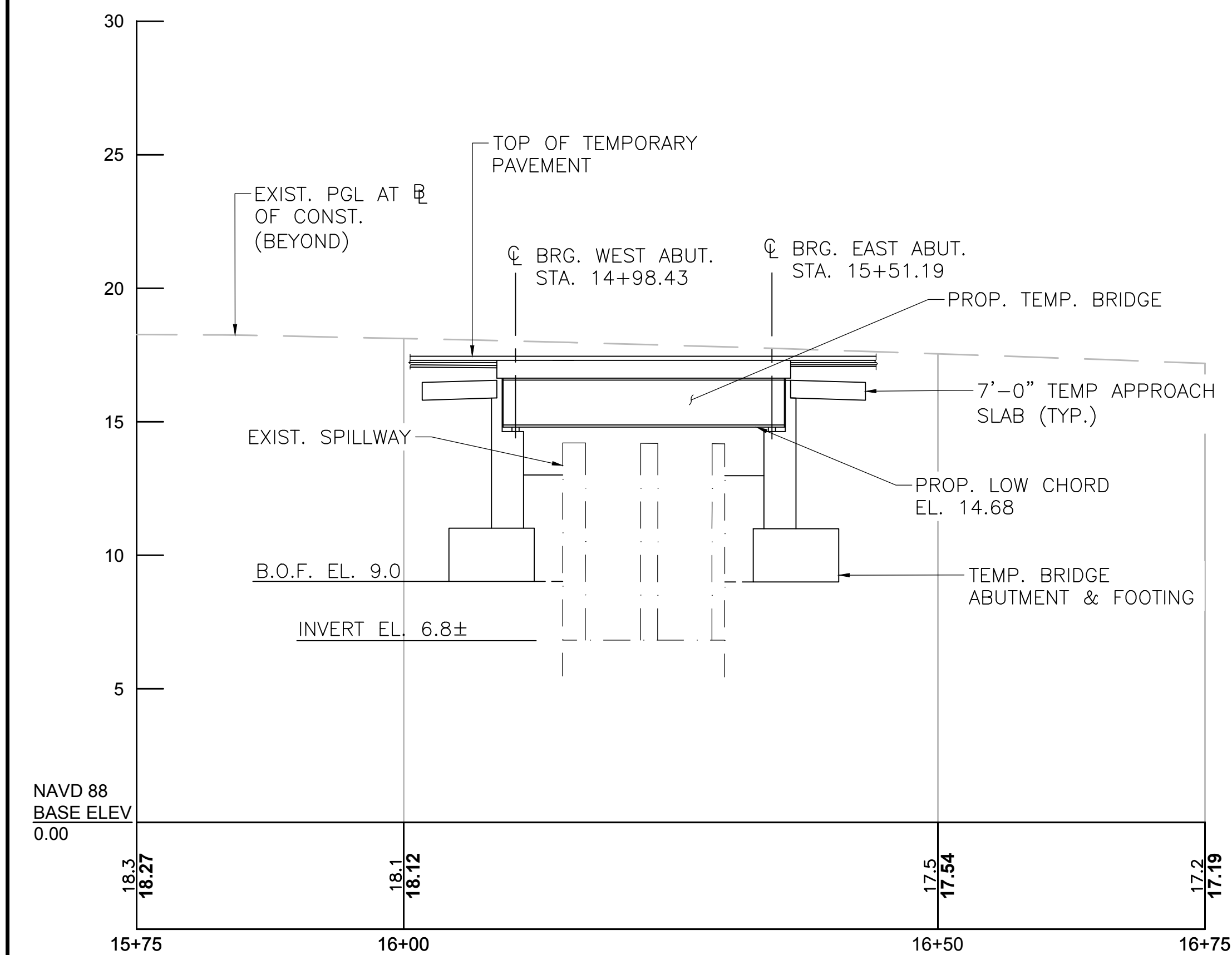




KEY PLAN

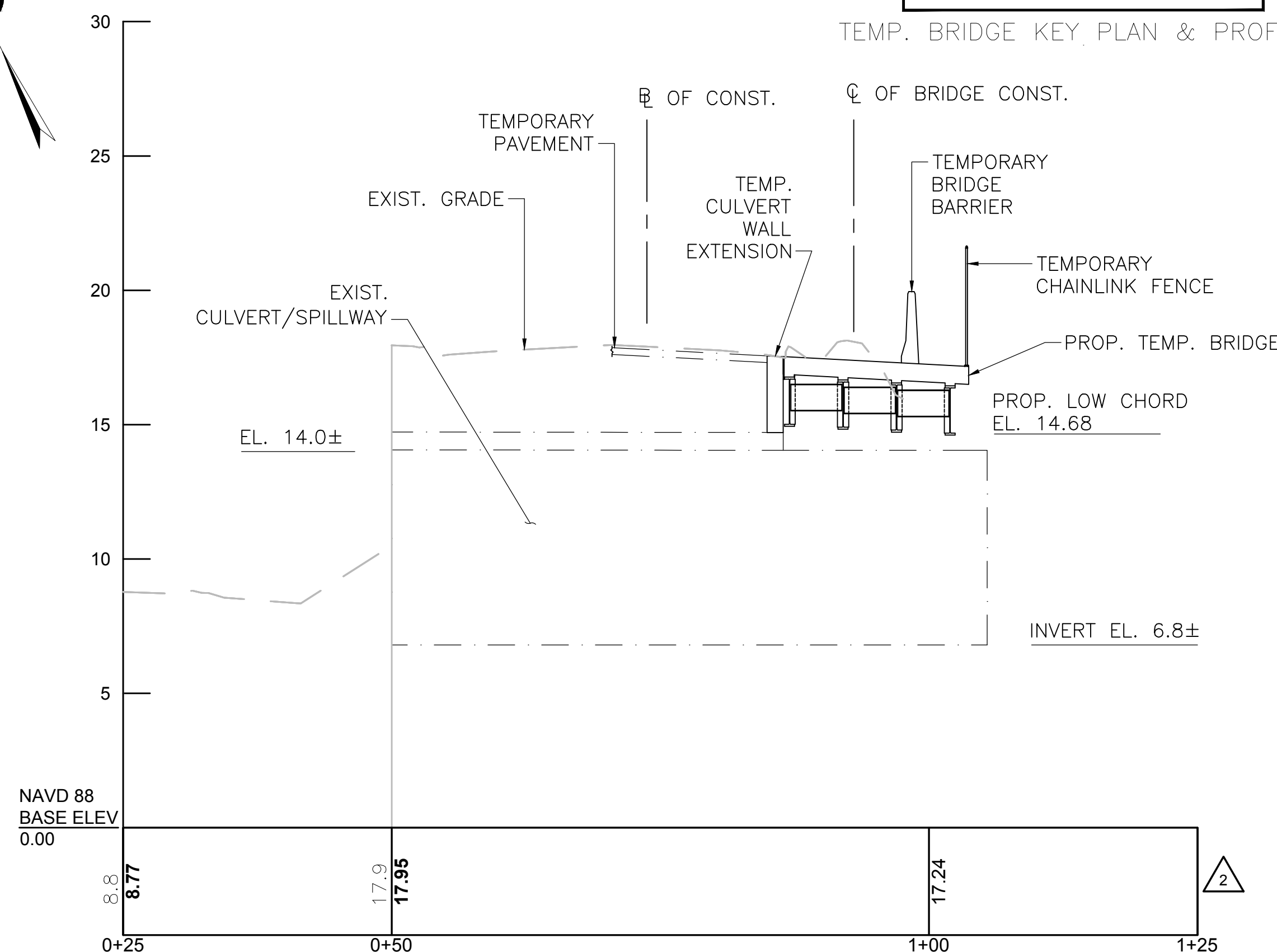
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CONSTRUCTION WORKING POINTS			
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T2	16+34.47	N2739553.636	E881019.163



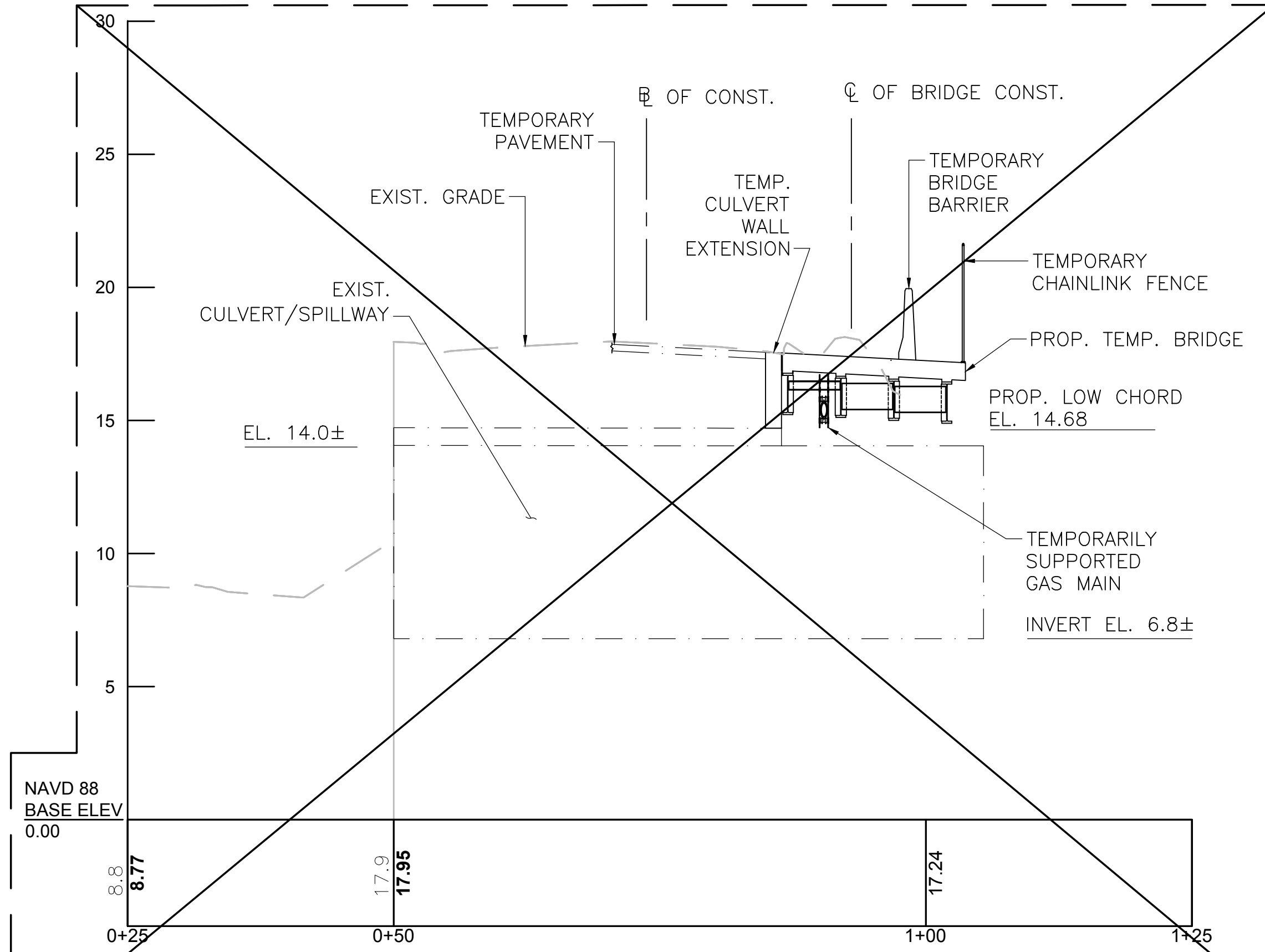
PROFILE - US 6 WB & ST 28 NB

HORIZONTAL 1"=10'
VERTICAL 1"=4'



PROFILE - SLAB/SPILLWAY

HORIZONTAL 1"=10'
VERTICAL 1"=4'



PROFILE - SLAB/SPILLWAY

HORIZONTAL 1"=10'
VERTICAL 1"=4'

SEPT. 25, 2026	2 REVISED PROFILE
AUGUST 8, 2026	ISSUED FOR CONSTRUCTION
DATE	DESCRIPTION
THIS SHEET IS APPROVED FOR CONSTRUCTION BY MASSDOT	
AUTHORIZED SIGNATORY: STATE BRIDGE ENGINEER	
USE ONLY PRINTS OF LATEST DATE	