STATE OF CONNECTICUT
DEPARTMENT OF TRANSPORTATION

M E M O R A N D U M

Subject: Project No. 15-248
Rte. 1 over Stillman Pond
Bridge #00325

City of: Bridgeport

Date: November 14, 2018

to: Mary Baker
Trans. Principal Engineer
Bureau of Engineering
and Construction

from: *for* Leo L. Fontaine
Trans. Principal Engineer
Bureau of Engineering
and Construction

Michael McDonnell

Digitally signed by Michael McDonnell
DN: C=US, E=michael.mcdonnell@ct.gov, O=Connecticut Department of Transportation, OU=Soils and Foundations, CN=Michael McDonnell
Date: 2018.11.14 16:28:55-05'00'

1. Transmitted are the following:
   - ☐ Roadway Geotechnical Report
   - ☒ Structure Geotechnical Report (REVISED)
   - ☐ Plans:
   - ☐ Correspondence:

2. This transmittal is being made:
   - ☒ In response to your request
   - ☐ Initiated by this office

3. Comments: This geotechnical report has been revised due to changes to the bottom of footing elevations for Retaining Wall 102 and Wingwall 2A. (11-9-2018)

4. Please take the following action:
   - ☐ Please review and forward to
   - ☒ Please review for incorporation into the design of the project
   - ☐ For your use and information

Attachment

Amy Hare/aeh

cc: Bryan Reed – Bao Chuong – Nick Martin
Michael McDonnell – Amy Hare
Eric Tallarita
Scott Bushee – Vitalij Staroverov

Project Description :

This project involves the rehabilitation of Bridge No. 00325, which carries U.S. Route 1 over Stillman Pond, two abandoned railroad spurs, and an access road in the City of Bridgeport (See attached Project Location Plan). The 75-foot long single-span bridge consists of a reinforced concrete arch supported by reinforced concrete abutments bearing on spread footings partially on rock. The bridge was built in 1910 and rehabilitated in 1935. It has a curb-to-curb width of 50 feet and carries 2 lanes of traffic in each direction. Stillman Pond is carried in a concrete lined channel beneath the bridge. The foundation dimensions of the channel walls are unknown.

The proposed rehabilitation will consist of an arch relining utilizing a skewed corrugated steel plate arch on CIP arch pedestals. The corrugated steel-plate arch will be installed beneath the existing structure, with the annular space between the two structures filled with CSLM. It is anticipated that a portion of the arch pedestal foundations will be founded on spread footings poured directly on bedrock and a portion of the arch pedestal foundations will be founded on micropiles drilled into bedrock due to a highly variable bedrock surface. The relining arch, arch pedestals, headwalls and the arch pedestal foundations will be Contractor-designed. New cast-in-place wingwalls, retaining walls, and the replacement of the channel wall for the Stillman Pond channel will be State-designed. The recommendations for those walls are enclosed in this report.

Geotechnical Information and Site Conditions:

Surficial Geology:

Published USGS mapping indicates the natural surficial soils at the site consist of sand and gravel. USGS maps are attached.

Existing borings, including those furnished with the request for this report, were completed during an earlier subsurface investigation for this project in the mid 1990’s. Two existing roadway borings (RB-3 and RB-4) were taken behind the existing bridge abutments and show at least 12 feet of loose to medium dense fill consisting of gravelly sand with ash and cinders.

Bedrock Geology:

Published USGS mapping indicates the bedrock underlying the site consists of Cooks Pond Schist. Cooks Pond Schist is defined as fine-grained, rusty weathering schist. The site lies just west of what is mapped as the Derby Hill Member of the Orange Formation. The Derby Hill member is described as thin-bedded fine to medium grained “pinstripe” schist and gneiss. USGS maps are attached.

Observations:

A subsurface investigation was performed in August, 2015, a geophysical investigation was performed in December, 2015, and test pits were performed in April of 2018. The August investigation included 4 structure borings; 1 behind each abutment and 1 through each abutment to determine the foundation type and thickness. The borings behind the abutments generally encountered fill bearing on gravelly sand bearing on bedrock. Boring location plans, finalized boring logs, and rock core data sheets are attached to this report.

The December investigation included two seismic refraction lines developing two bedrock profiles. The geophysical report is attached and includes refraction line locations and profiles. The April investigation included four test pits to depths varying from 8 to 14 feet below existing ground surface to determine the existence and depths of the existing retaining wall and wingwall footings. The test pit summary package is attached and the test pit summary is included below. Locations of the test pits can be found on the boring location plan.

The general soil matrix at the site is as follows:

Thickness (Ft)	**Description**
16-21	**Miscellaneous Fill** – Loose to medium loose SAND, silt, and gravel in varying percentages mixed with slag/cinders and brick fragments.
4 - 7	**Gravelly Sand** – Very dense SAND with some gravel, little to trace silt.
2 - 6	**Weathered Bedrock** – Weak, highly weathered to residual Schist and Gneiss.
	Bedrock – Strong, moderately to slightly weathered interbedded Schist and Gneiss

.

Groundwater Observations – Stillman Pond flows at approximately elevation 11 feet within the channel. Groundwater measurements in borings B-3 and B-4 were consistent with this elevation.

Test Pit Summary:

Test Pit	Structure	Survey Shot	Northing	Easting	Elevation	Notes
TP-1	RW-102	Top	131452.59	483924.84	22.98	Footing encountered.
		Bottom	131454.34	483925.23	17.65	Footing encountered.
TP-2	WW-2A	NA				Excavation reached approximately 14' below existing ground surface along wingwall. Groundwater was not encountered.
TP-3	WW-2B	NA				Excavation reached approximately 13' below existing ground surface along wingwall. Groundwater was encountered at approximately 9' below existing ground surface.
TP-4	RW-103	Top	131373.47	483931.98	16.36	Footing encountered.
		Bottom	131372.49	483932.08	13.64	Footing encountered.

Recommendations:

Design Recommendations:

- The Corrugated Arch System, including the arch pedestals and pedestal foundations will be Contractor-designed. The pedestal foundations are anticipated to be partially founded on spread footings on bedrock and partially on a pile cap supported by micropiles socketed into rock.
- The Contractor is required to design, install and test the micropiles as per the attached special provision which modifies the Standard Specifications Section 7.06 “Micropiles” and requires the Contractor to design for the strength and service limit states for both axial and lateral load demands defined by the arch pedestal designer.
- See the Appendices for all subsurface data to utilize for the Contractor’s design.
- The following are the minimum micropile requirements that shall be shown on the Contract plans:
  - All micropiles will have permanent casing left in place and seated 6 inches into competent bedrock.
  - The plans and quantities should call for one (1) verification test pile and a minimum of two (2) proof tests along the length of the micropile-supported section of the retaining wall, using the appropriate geotechnical resistance factor as per AASHTO. The verification test should be a non-production/sacrificial pile and will be installed and tested prior to any production piles being installed. This additional time for testing should be noted in the Construction Sequence and Schedule. The pile layout plan in the structure sheets should note the locations of the test piles.

- The following are the estimated micropile requirements to be utilized by CTDOT Bridge Design:
    - Estimated casing properties: 10.75”OD, 0.5” thickness, and 45 ksi strength
    - Estimated reinforcing bar properties: a #18 bar, 60 ksi strength
    - Estimated concrete strength: 4 ksi.
    - Estimated spacing of micropiles between 30 inches and 5 feet.
    - Estimated rock socket diameter: 9.75 inches.
    - Estimated Maximum Strength Pile Load is 82 kips/pile.
    - Estimated Maximum Service Limit Pile Load is 54 kips/pile.
    - Estimated Ultimate Pile Capacity is 149 kips/pile.
    - The estimated bond zone length in bedrock is 20 feet (based upon a grout to ground bond strength of 13 ksf.)
- Include pay items: “Micropiles” as a lump sum pay item as defined by the attached special provision.

<u>Wingwalls and Retaining Walls Recommendations:</u>

Wingwall 1A is a cast-in-place retaining wall with a maximum retained height of approximately 22 feet and a length of 16 feet and will be founded at the proposed bottom of footing elevation of 12.5 feet.

- Found the wingwall footing on 1 foot of Granular Fill.
- Use the attached Geotechnical Wall Design Parameters sheet to design the wall.
- Anticipated immediate settlement is ½ inch. Post-construction settlement is anticipated to be negligible.

Wingwall 2A is a two-stepped cast-in-place retaining wall with a maximum retained height of 20.5 feet, secondary retained height of 16.5 feet with lengths of 22 and 16 feet, respectively. The steps will be founded at the proposed bottom of footing elevations of 14 and 18 feet.

- Found the wingwall step 1 on 1 foot of Granular Fill. Found the wingwall Step 2 on a minimum of 2 feet of Granular Fill.
- Use the attached Geotechnical Wall Design Parameters sheet to design the walls.
- Anticipated immediate settlement is ½ inch. Post-construction settlement is anticipated to be negligible.

Wingwall 1B is a three-stepped cast-in-place retaining wall with a maximum retained height of 22.5 feet, mid-level retained height of 15 feet and a minimum retained height of 7.5 feet, with a length of 11 or 13 feet. The steps will be founded at elevations 12, 19.5 and 26 feet.

- Found the wingwall footing steps on a minimum of 1 foot of Compacted Granular Fill. Found the third step on a minimum of 2 feet of Compacted Granular Fill.
- Backfill the anticipated annular space between the open excavation and the wingwall footings due to the large steps with Compacted Granular Fill.
- Use the attached Geotechnical Wall Design Parameters sheet to design the walls.

- Anticipated immediate settlement is ½ inch. Post-construction settlement is anticipated to be negligible.

Wingwall 2B is a cast-in-place retaining wall with a maximum retained height of 29 feet with a length of 23 feet, and will be founded at the proposed bottom of footing elevation of 5.5 utilizing permanent ground anchors.

- Found the wingwall footing on prepared bedrock or a lean concrete leveling pad.
- The bond length, vertical anchor diameter, grout strength and the strength/size of the stressing tendon will be determined by the Contractor.
- The plans should call for a minimum bond length of 10 feet for strand or bar tendons in bedrock.
- The minimum horizontal spacing between anchors is 5 feet or 3 times the diameter of the bond zone.
- Preliminary anchor loads provided by the structural engineer is 120 kips. Anchors with a pullout resistance of 180 kips and higher are feasible if gravity grouted anchors with a 10-foot bond zone in competent bedrock are used.
- As specified in AASHTO Table 11.5.7-1, the resistance factor is 1.0 for anchors when proof testing is conducted on all anchors.
- The plans should provide the anchor Factored Design Loads (FDL) for the controlling Strength and Service Limit States.
- Include a 2-inch thick geofoam inclusion behind the stem of this wall to ensure the assumed active earth pressures are achieved. Use the Geofoam Special Provision included in the appendices of this report.
- Use the attached Geotechnical Wall Design Parameters sheet to design the walls.
- Anticipated immediate settlement is less than ¼ inch. Post-construction settlement is anticipated to be negligible.

Retaining Wall 101 is a cast-in-place retaining wall with a maximum retained height of approximately 16 feet and a length of 19 feet, founded at a proposed bottom of footing elevation of 20 feet.

- Found the wingwall footing on 1 foot of Granular Fill.
- Use the attached Geotechnical Wall Design Parameters sheet to design the wall.
- Anticipated immediate settlement is ½ inch. Post-construction settlement is anticipated to be negligible.

Retaining Wall 102 is a cast-in-place retaining wall with a maximum retained height of 18.3 feet and a length of 29.5 feet, founded at a bottom of footing elevation of 18 feet.

- Found the wingwall on a minimum of 2 feet of Granular Fill.
- Use the attached Geotechnical Wall Design Parameters sheet to design the walls.
- Anticipated immediate settlement is ½ inch. Post-construction settlement is anticipated to be negligible.

Retaining Wall 103 (and steps) are cast-in-place retaining walls with a maximum retained height of 21 feet with a length of 30 feet, founded at the proposed bottom of footing elevation of 15 feet.

- Found the wingwall footing on 1 foot of Granular Fill.
- Use the attached Geotechnical Wall Design Parameters sheet to design the walls.
- Anticipated immediate settlement is ½ inch. Post-construction settlement is anticipated to be negligible.

Stillman Pond Channel Wall is a cast-in-place semi-gravity wall with a maximum retained height of 8.5 feet and an approximate minimum length of 135 feet, founded at proposed bottom of footing elevation 5 feet. It's anticipated that bedrock excavation will be required for portions of this wall. In areas where bedrock is encountered, the bedrock should be over-excavated so as to allow for placement of 1foot of Granular Fill below the footing, for the purpose of providing a uniform bearing surface.

- Found the gravity wall on 1 ft of Granular Fill.
- Use the attached Geotechnical Wall Design Parameters sheet to design the wall.
- Anticipated settlement for the section of the gravity wall on granular fill is ½ inch. Post-construction settlement is anticipated to be negligible.

Construction Considerations:

- Groundwater is anticipated to be encountered for installation of the spread footings for the wingwalls and foundation elements that are founded below elevation 11 feet (The arch pedestal abutments, Stillman Pond channel wall and Wingwall 2B). Dewatering efforts will require the use of fully enclosed cofferdams. Due to a highly variable, sometimes shallow bedrock surface, cantilevered steel sheet piling cofferdams may not be feasible for the entire length of cofferdam required. Dewatering could be accomplished using a braced, steel sheet piling cofferdams with groundwater pumping from a low point within the excavation. Use pay items 'Structure Excavation – Earth (Excluding Cofferdam and Dewatering)' and 'Cofferdam and Dewatering.'
- Structure excavation for retaining walls located above the groundwater table will likely need TERS to support the adjacent Route 1 (Wingwall 1A, Wingwall 2A, Wingwall 1B, Retaining Wall 101 and Retaining Wall 102). The maximum cut slope rates to determine the need for Temporary Earth Retention Systems (TERS) shall be 1 ½ (H) to 1 (V). Cantilevered sheet piling may be feasible for some portions of the excavation support, but some portions will require a braced system or block wall. Use pay item Structure Excavation – Earth (Complete).
- Where TERS and Cofferdam pay items coincide, the pay item shall be 'Cofferdam and Dewatering' utilizing the attached special provision.
- Weathered rock and bedrock may be encountered at the bottom of footing elevation for the arch pedestal foundations and the Stillman Pond Channel Wall with deeper foundation elevations. Most of the rock should be rippable if removal is necessary to construct the required footing thickness. Assume a rock excavation quantity of 130 CY.

Attachments:

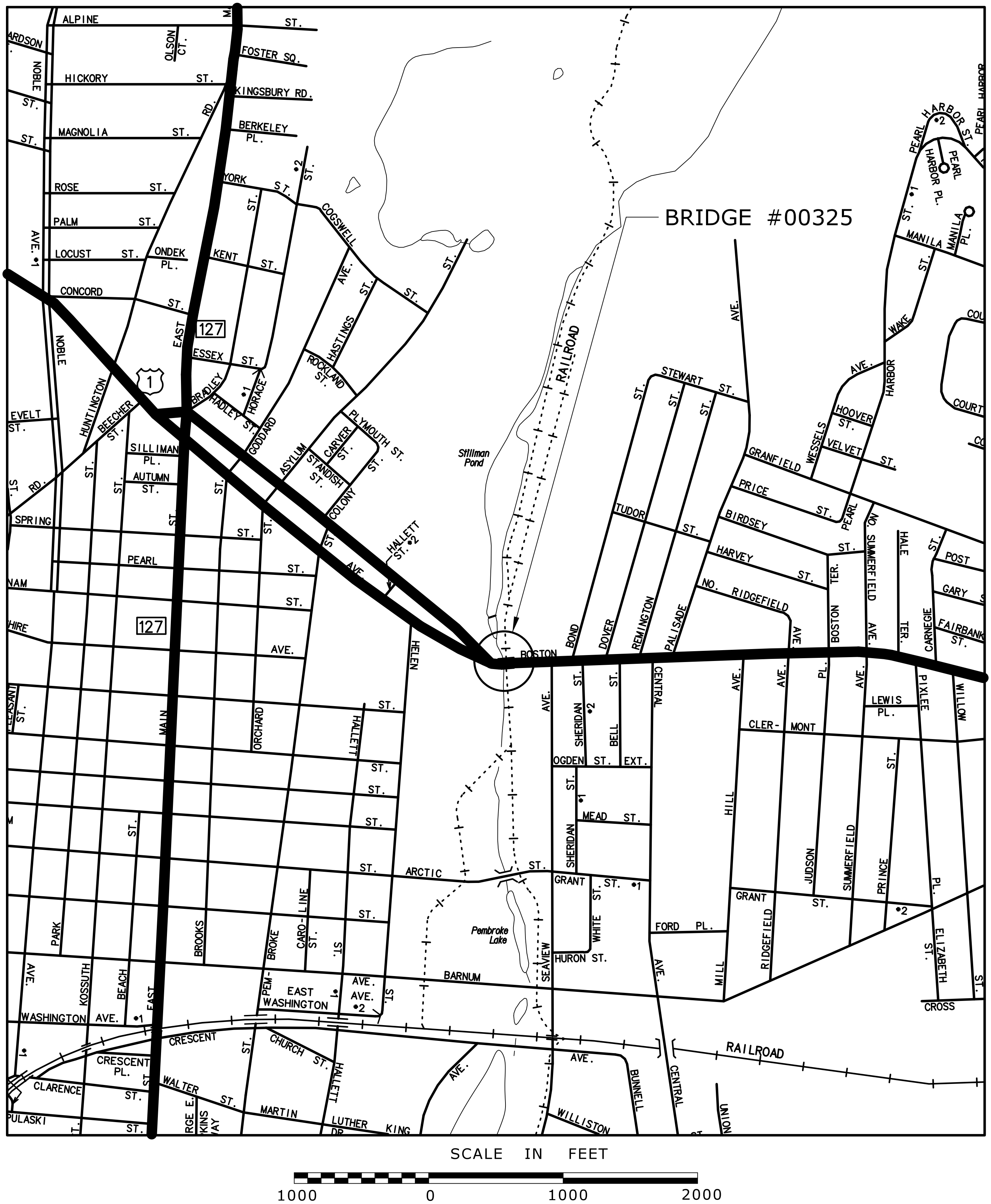
Figures:
- Project Location Plan
- USGS Surficial Soils Map
- USGS Bedrock Geology Map
- Boring and Geophysical Line Location Plan
- Geologic Profile – Bridge Inlet and Retaining Walls
- Geologic Profile – Bridge Outlet and Retaining Walls
- Geophysical Line Profiles

Subsurface Data:
- Boring Logs
- Historical Boring Logs
- Rock Core Data Sheets
- Test Pit Summary

Analysis:
- Laboratory Testing Results

Design:
- Geotechnical Wall Design Sheets (All)
- Design-Build Micropile Special Provision
- Geofoam Special Provision (GeoInclusion)
- Cofferdam and Dewatering Special Provision (TERS coincides)

Other:
- Geophysical (Seismic Refraction) Report



STATE PROJECT NO.:

15-248

CITY/TOWN:

BRIDGEPORT

STATE OF CONNECTICUT
DEPARTMENT OF TRANSPORTATION

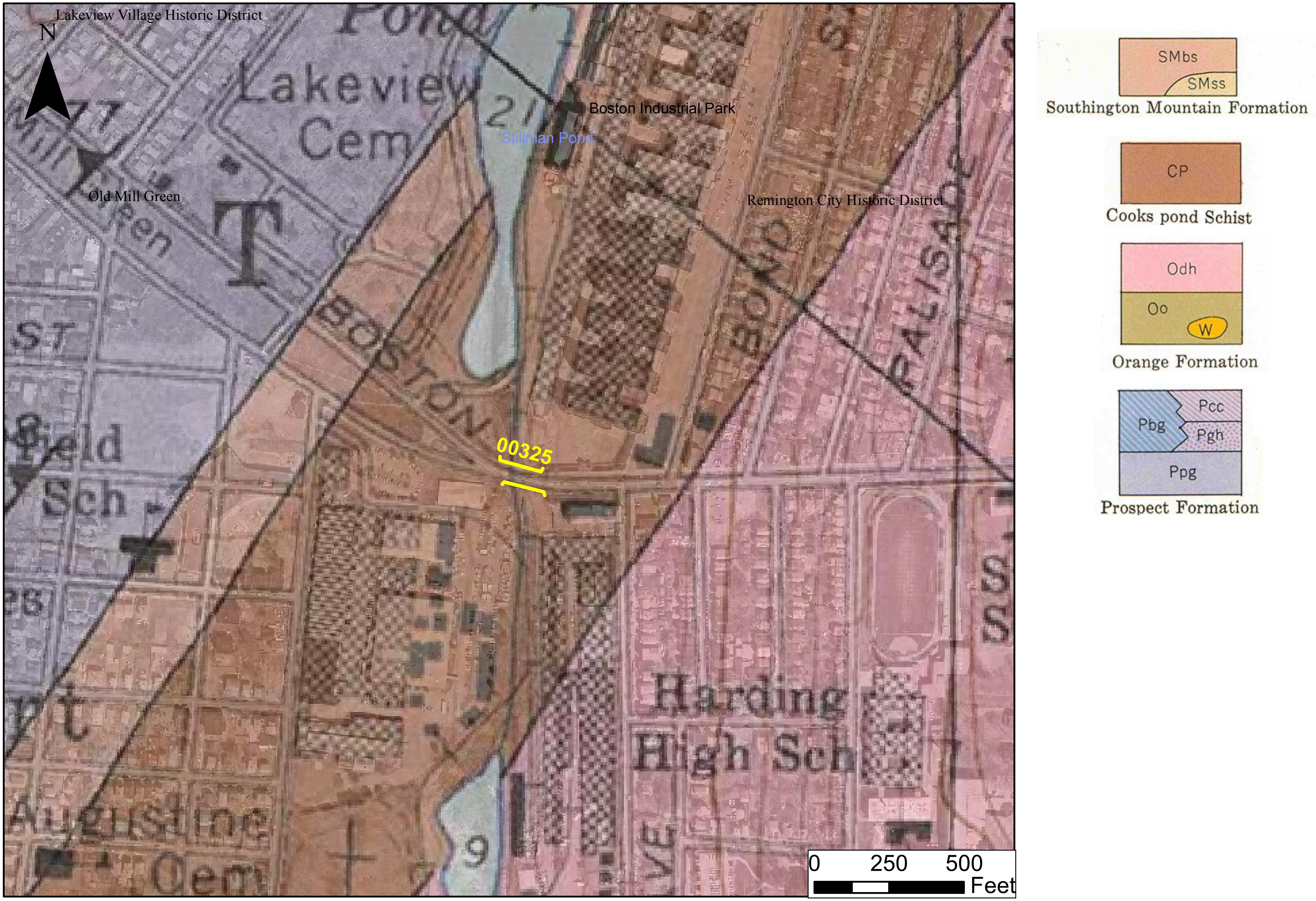
BRIDGE #00325 OVER
STILLMANS POND

OFFICE OF
ENGINEERING

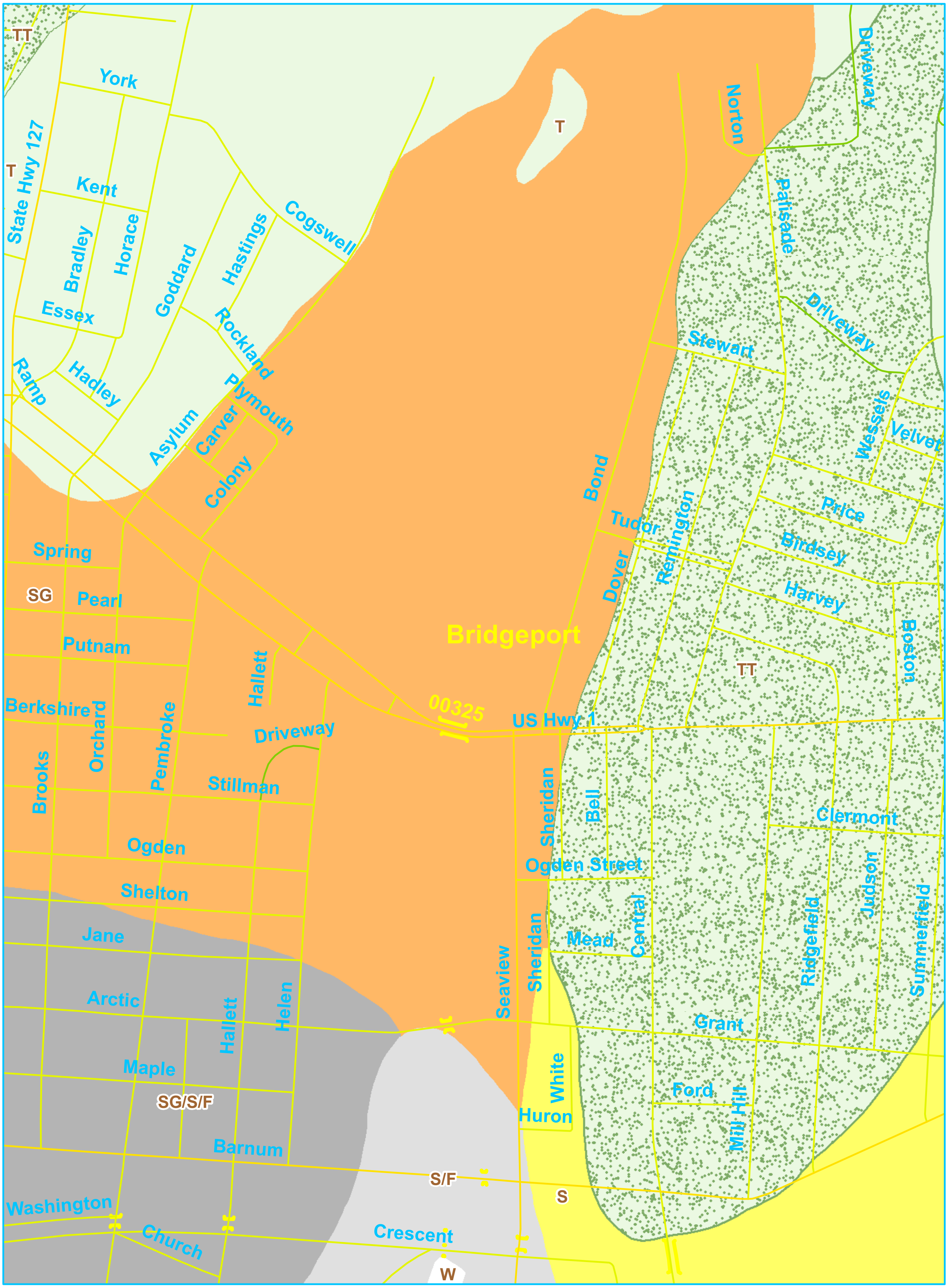
DATE:

01/07/14

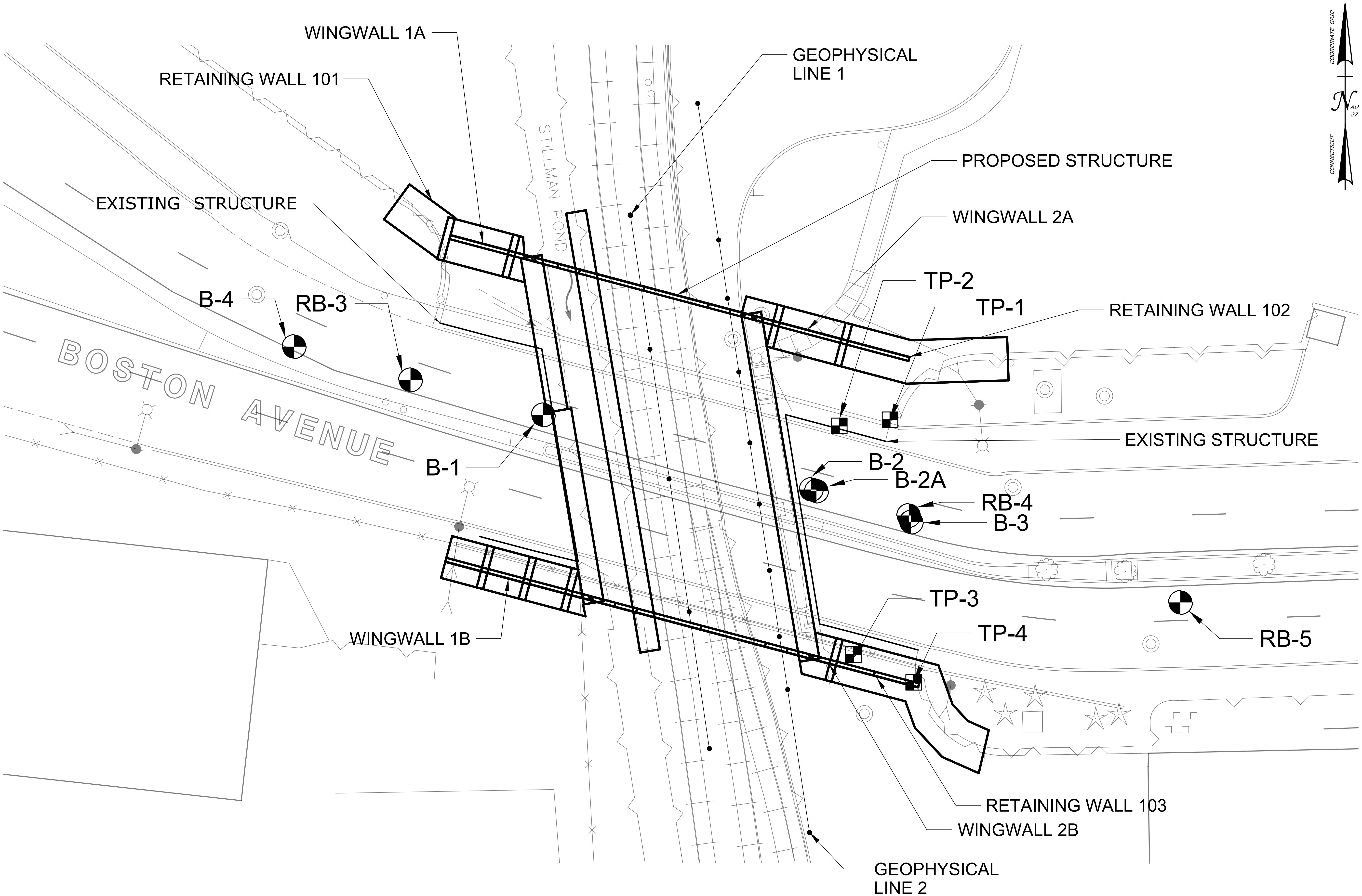
Bedrock Geology



Surficial Materials



Project No. 0015-0248, Bridge No. 00325
Bridgeport, CT



REV.	DATE	REVISION DESCRIPTION	SHEET NO.

THE INFORMATION, INCLUDING ESTIMATED QUANTITIES OF WORK, SHOWN ON THESE SHEETS IS BASED ON LIMITED INVESTIGATIONS BY THE STATE AND IS IN NO WAY WARRANTED TO INDICATE THE CONDITIONS OF ACTUAL QUANTITIES OF WORK WHICH WILL BE REQUIRED.

Plotted Date: 11/13/2018

DESIGNER/DRAFTER: AEH

CHECKED BY: -

STATE OF CONNECTICUT
DEPARTMENT OF TRANSPORTATION

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OFFICE OF ENGINEERING

APPROVED BY:

PROJECT TITLE:

-
REPLACEMENT OF BRIDGE NO. 00325

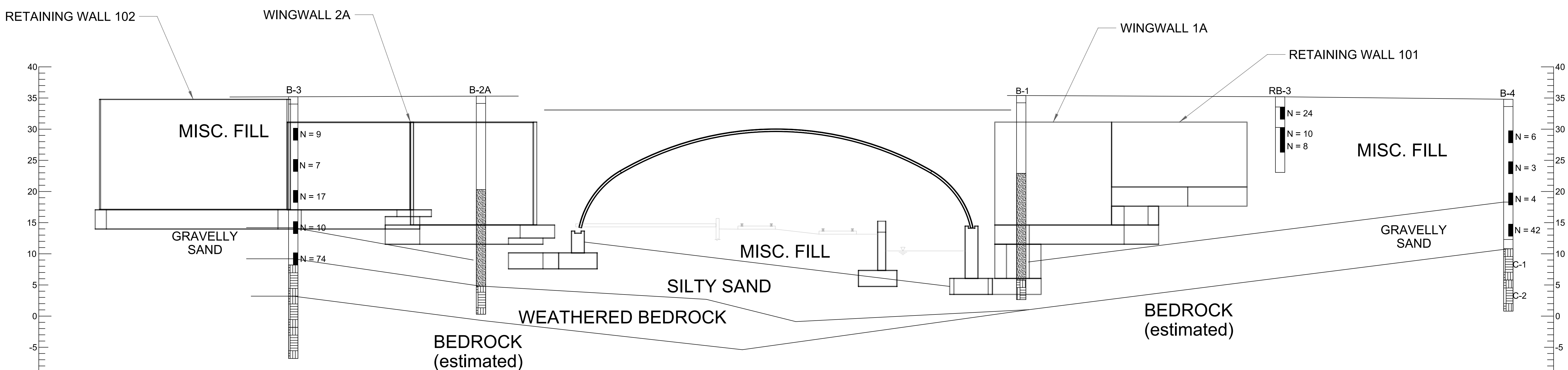
TOWN: **BRIDGEPORT**
-

DRAWING TITLE:
BORING LOCATION PLAN GEOTECHNICAL REPORT

PROJECT NO. 15-248

DRAWING NO. -

SHEET NO.



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Plotted Date: 11/13/2018

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CHECKED BY: -

STATE OF CONNECTICUT
DEPARTMENT OF TRANSPORTATION

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OFFICE OF ENGINEERING

APPROVED BY:

PROJECT TITLE:

REPLACEMENT OF BRIDGE NO. 00325
-

TOWN:

CITY OF BRIDGEPORT

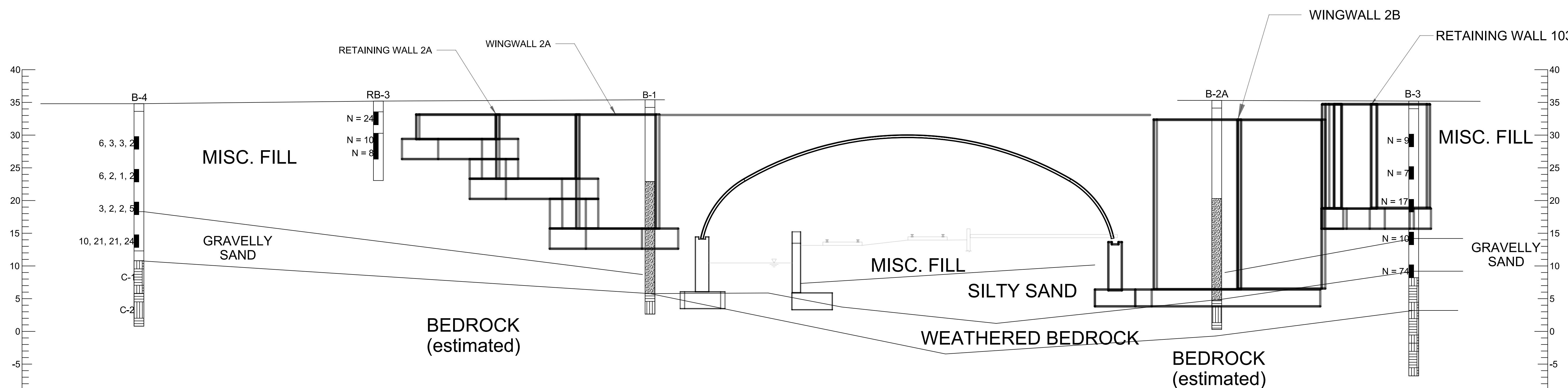
DRAWING TITLE:

INLET PROFILE
GEOTECHNICAL REPORT

PROJECT NO. 15-248

DRAWING NO. -

SHEET NO.



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Plotted Date: 11/13/2018

DESIGNER/DRAFTER: AEH

CHECKED BY: -

STATE OF CONNECTICUT
DEPARTMENT OF TRANSPORTATION

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OFFICE OF ENGINEERING

APPROVED BY:

PROJECT TITLE:

REPLACEMENT OF BRIDGE NO. 00325
-

TOWN:

CITY OF BRIDGEPORT

DRAWING TITLE:

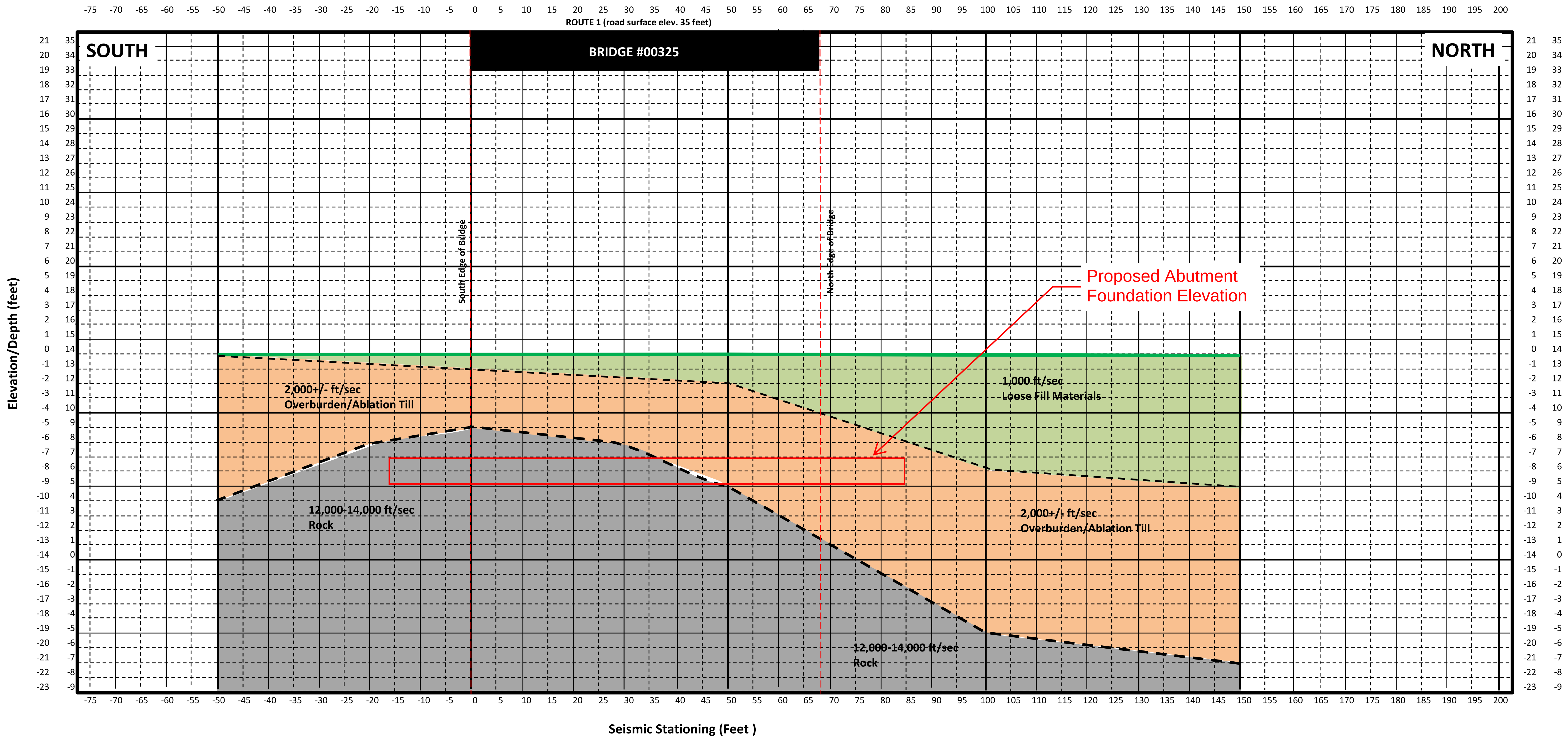
OUTLET PROFILE GEOTECHNICAL REPORT

PROJECT NO. 15-248

DRAWING NO. -

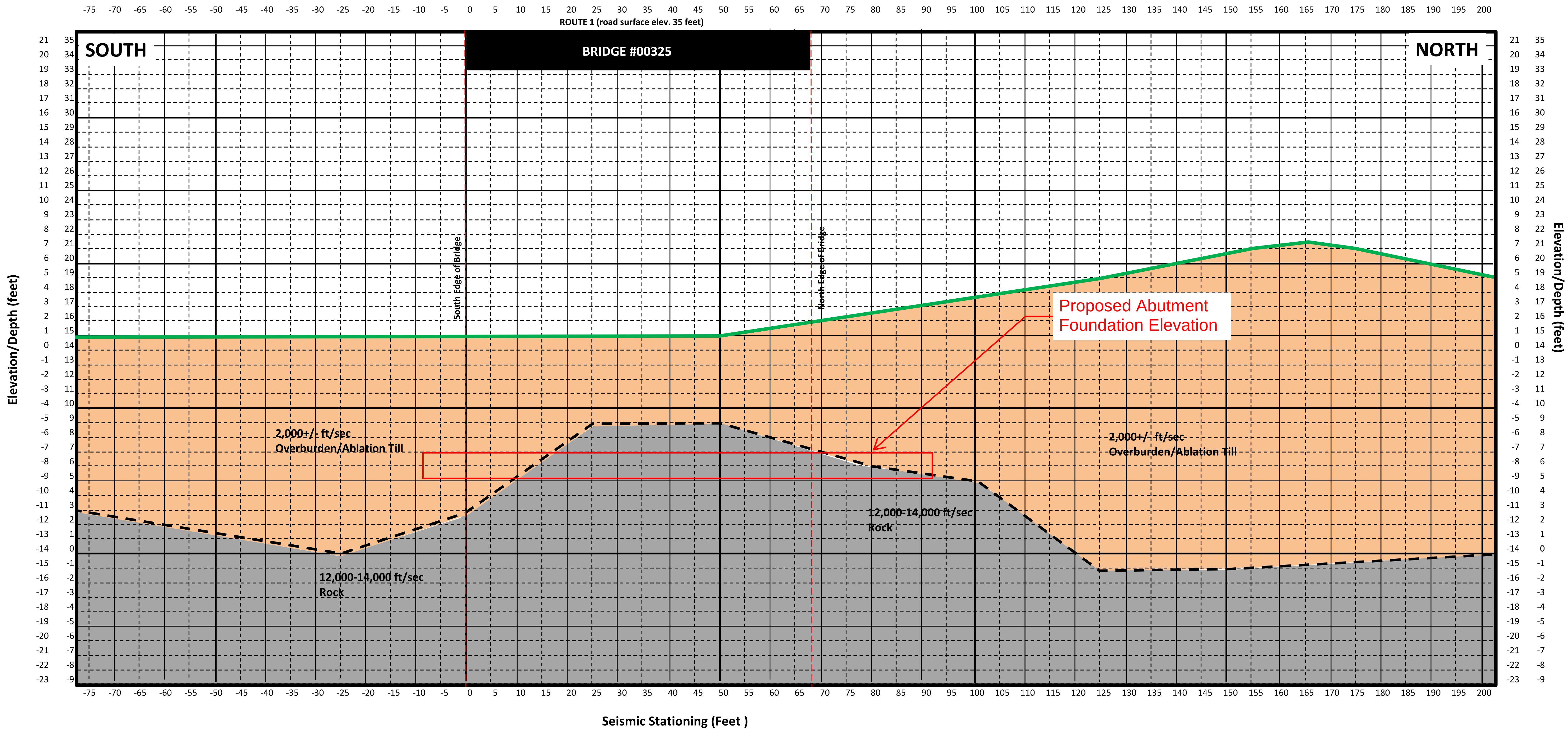
SHEET NO.

Seismic Line 1 - Center between Tracks



Geophysical Survey Depth to Bedrock Bridge # 00325 Route 1 over Stillmans Pond Bridgeport, Connecticut for Connecticut DOT by NDT Corporation	Seismic Results Line 1
	Dec. 2015 — Figure 3

Seismic Line 2 and 3 - Near East Abutment



Geophysical Survey Depth to Bedrock Bridge # 00325 Route 1 over Stillmans Pond Bridgeport, Connecticut for Connecticut DOT by NDT Corporation	Seismic Results Line 2 and 3	
	Dec. 2015	Figure 4

Connecticut DOT Boring Report

Driller:	J. Dorau			Hole No.:	B-1
Inspector:	Glenn L. Arzt	Town:	Bridgeport	Stat./Offset:	
Engineer:	B. McKiernan	Project No.:	15-248	Northing:	131454.3
Start Date:	8-28-15	Route No.:	1	Easting:	483820
Finish Date:	8-28-15	Bridge No.:	00325	Surface Elevation:	35.4

Project Description: Rehabilitation of Bridge 00325

Casing Size/Type: 4in HW	Sampler Type/Size: 2in SS	Core Barrel Type: NQ2
Hammer Wt.: 140lb Fall: 30in.	Hammer Wt.: 140lbs Fall: 30in.	

Groundwater Observations (ft):

Depth (ft)	Sample Type/No.	Blows on Sampler per 6 inches	Pen. (in.)	Rec. (in.)	RQD %	Generalized Strata Description	Material Description and Notes	Elevation (ft)
0						PAVEMENT STRUCTURE	14" PAVEMENT STRUCTURE (Asphalt over Concrete)	35
						MISC. FILL		
5								30
10								25
	S-1	40	3	0.5		CONCRETE	Gray CONCRETE CHIPS	
15	C-1		60	59			CONCRETE. Core times(min); 10.5, 9, 8.5, 8.5, 7.5	20
20	C-2		60	61			CONCRETE. Core times(min); 8, 9, 8, 8, 8.5	15
25	C-3		60	61			CONCRETE. Core times(min); 6.5, 7.5, 7.5, 7, 8	10
30	C-4		60	63	100	BEDROCK	Top 22.5": CONCRETE Bottom 40.5": Gray fine-grained thickly bedded moderately fractured slighlty weathered and strong SCHIST. Core times(min); 7.5, 9, 11.5, 11, 9.5	5
							END OF BORING 32.75ft	
35								0
40								

Sample Type: S = Split Spoon C = Core UP = Undisturbed Piston V = Vane Shear Test

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

Total Penetration in Earth: 29.6ft Rock: 3.15ft	NOTES: Top of footing encountered at 12.5', advanced to 12.7' with roller bit and seated casing. 1" steel bar encountered 5" into C-1.	Sheet 1 of 1
No. of Soil Samples: 1 No. of Core Runs: 4		SM-001-M REV. 1/02

Connecticut DOT Boring Report

Driller: J. Dorau		Hole No.: B-2
Inspector: Glenn L. Arzt	Town: Bridgeport	Stat./Offset:
Engineer: B. McKiernan	Project No.: 15-248	Northing: 131431.8
Start Date: 8-24-15	Route No.: 1	Easting: 483900.9
Finish Date: 8-25-15	Bridge No.: 00325	Surface Elevation: 35.3

Project Description: Rehabilitation of Bridge 00325

Casing Size/Type: 4in HW	Sampler Type/Size: 2in SS	Core Barrel Type: NQ2
Hammer Wt.: 140lb Fall: 30in.	Hammer Wt.: 140lbs Fall: 30in.	

Groundwater Observations (ft):

Depth (ft)	SAMPLES: Sample Type/No.	Blows on Sampler per 6 inches	Pen. (in.)	Rec. (in.)	RQD %	Generalized Strata Description	Material Description and Notes	Elevation (ft)
0						PAVEMENT STRUCTURE	14" PAVEMENT STRUCTURE (Asphalt over Concrete)	35
						MISC. FILL		
5								30
10								25
15	S-1	32	1	1		CONCRETE	Brown C-F SAND, some c-f Gravel, tr. Silt w/concrete chips	20
	C-1		59	59			CONCRETE. Core times(min); 5.5, 5, 8, 8, 8	
20								15
	C-2		60	60			CONCRETE. Core times(min); 6.5, 7, 7.5, 7, 7.5	
25	C-3		36	36			CONCRETE. Core times(min); 6.5, 7, 7	10
30							END OF BORING 28ft	5
35								0
40								

Sample Type: S = Split Spoon C = Core UP = Undisturbed Piston V = Vane Shear Test

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

Total Penetration in Earth: 28ft Rock: 0ft	NOTES: Core barrel clogged 3' into C-3 resulting in water supply hose failure. The core barrel was reinserted to 28ft the next day and coring was resumed but was unable to advance past 28.' Upon extraction, the driller noted all of the teeth had broken off the drill bit and the bit was no longer usable. The hole was abandoned.	Sheet 1 of 1
No. of Soil Samples: 1 No. of Core Runs: 3		SM-001-M REV. 1/02

Connecticut DOT Boring Report

Driller:	J. Dorau			Hole No.:	B-2A
Inspector:	Glenn L. Arzt	Town:	Bridgeport	Stat./Offset:	
Engineer:	B. McKiernan	Project No.:	15-248	Northing:	131431.3
Start Date:	8-31-15	Route No.:	1	Easting:	483902.5
Finish Date:	8-31-15	Bridge No.:	00325	Surface Elevation:	35.3

Project Description: Rehabilitation of Bridge 00325

Casing Size/Type: 4in HW	Sampler Type/Size: 2in SS	Core Barrel Type: NQ2
Hammer Wt.: 140lb Fall: 30in.	Hammer Wt.: 140lbs Fall: 30in.	

Groundwater Observations (ft):

Depth (ft)	SAMPLES: Sample Type/No.	Blows on Sampler per 6 inches	Pen. (in.)	Rec. (in.)	RQD %	Generalized Strata Description	Material Description and Notes	Elevation (ft)
0						PAVEMENT STRUCTURE	14" PAVEMENT STRUCTURE (Asphalt over Concrete)	35
						MISC. FILL		
5								30
10								25
15						CONCRETE		20
	C-1		60	60.5			CONCRETE. Core times(min); 6.5, 7.5, 8, 6.5, 7	
20								15
	C-2		60	59.5			CONCRETE. Core times(min); 6, 6.5, 6, 6.5, 7	
25								10
	C-3		60	61			CONCRETE. Core times(min); 7, 7, 6.5, 7, 7	
30						WEATHERED BEDROCK	Top 4" CONCRETE Bottom 42" Gray fine-grained medium bedded intensely fractured moderately to highly weathered medium strong GNEISS. Core times(min); 2.5, 11.5, 8, 8.5, 7.5	5
	C-4		60	48	0			
35							END OF BORING 35ft	0
40								

Sample Type: S = Split Spoon C = Core UP = Undisturbed Piston V = Vane Shear Test

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

Total Penetration in Earth: 30.5ft Rock: 4.5ft	NOTES: C-4 encountered possible void or completely weathered seam of bedrock at 30'2" to 30'6." Steel chips noted in wash return at same time.	Sheet 1 of 1
No. of Soil Samples: 0 No. of Core Runs: 4		SM-001-M REV. 1/02

Connecticut DOT Boring Report

Driller: J. Dorau		Hole No.: B-3
Inspector: B. McKiernan	Town: Bridgeport	Stat./Offset:
Engineer: B. McKiernan	Project No.: 15-248	Northing: 131421.9
Start Date: 8-19-15	Route No.: 1	Easting: 483931.2
Finish Date: 8-21-15	Bridge No.: 00325	Surface Elevation: 35.2

Project Description: Rehabilitation of Bridge 00325

Casing Size/Type: 4in HW	Sampler Type/Size: 2in SS	Core Barrel Type: NQ2
Hammer Wt.: 140lb Fall: 30in.	Hammer Wt.: 140lbs Fall: 30in.	

Groundwater Observations (ft): @14.5 after 0 hours

Depth (ft)	Sample Type/No.	Blows on Sampler per 6 inches				Pen. (in.)	Rec. (in.)	RQD %	Generalized Strata Description	Material Description and Notes	Elevation (ft)
0									PAVEMENT STRUCTURE / MISC. FILL	14" PAVEMENT STRUCTURE (Asphalt over Concrete)	35
5	S-1	5	5	4	6	24	16			Brown F-C SAND, little Silt, little c-f Gravel	30
10	S-2	10	5	2	4	24	8			Brown F-C SAND, little(+) Silt, little f-c Gravel w/slag	25
15	S-3	7	9	8	9	24	5			Black C-F SAND and F-C GRAVEL, tr. Silt, tr. Brick w/slag	20
20	S-4	5	5	5	5	24	6		GRAVELLY SAND	Black/Brown F-C SAND and F-C GRAVEL, little Silt, tr. Glass, tr. Brick w/slag	15
25	S-5	16	32	42	39	24	14		WEATHERED BEDROCK	Top 6" Brown F-C SAND, some f-c Gravel, tr. Silt Bottom 8" Gray completely WEATHERED BEDROCK	10
30	C-1					60	14	0		Gray fine-grained medium bedded intensely fractured highly weathered and medium strong interlayered SCHIST and GNEISS. Core times(min); 2.5, 2.5, 3, 9.5, 3	5
35	C-2					60	37	17.5	BEDROCK	Gray fine-grained thickly bedded highly fractured moderately weathered and strong interlayered SCHIST and GNEISS. Core times(min); 5, 4, 8, 8.5, 8	0
40	C-3					60	53	85		Gray fine-grained thickly bedded moderately fractured slightly weathered and strong GNEISS. Core times(min); 5, 6, 7, 7.5, 9.5	-5
45										END OF BORING 42ft	

Sample Type: S = Split Spoon C = Core UP = Undisturbed Piston V = Vane Shear Test

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

Total Penetration in Earth: 26ft Rock: 16ft	NOTES: Advanced to 5ft with 3.25in HSA then cased hole after S-1. No wash return during C-1 and first 3ft of C-2.	Sheet 1 of 1
No. of Soil Samples: 5 No. of Core Runs: 3		SM-001-M REV. 1/02

Connecticut DOT Boring Report

Driller:	J. Dorau			Hole No.:	B-4
Inspector:	Glenn L. Arzt	Town:	Bridgeport	Stat./Offset:	
Engineer:	B. McKiernan	Project No.:	15-248	Northing:	131475
Start Date:	8-25-15	Route No.:	1	Easting:	483744.7
Finish Date:	8-27-15	Bridge No.:	00325	Surface Elevation:	34.8

Project Description: Rehabilitation of Bridge 00325

Casing Size/Type: 4in HW	Sampler Type/Size: 2in SS	Core Barrel Type: NQ2
Hammer Wt.: 140lb Fall: 30in.	Hammer Wt.: 140lbs Fall: 30in.	

Groundwater Observations (ft): @24.5 after 0 hours

Depth (ft)	Sample Type/No.	Blows on Sampler per 6 inches				Pen. (in.)	Rec. (in.)	RQD %	Generalized Strata Description	Material Description and Notes	Elevation (ft)
0									PAVEMENT STRUCTURE	14" PAVEMENT STRUCTURE (Asphalt over Concrete)	
									MISC. FILL		
5	S-1	6	3	3	2	24	8			Black C-F SAND and F-C GRAVEL, tr. Silt, tr. Brick w/slag	30
10	S-2	6	2	1	2	24	2			Black C-F SAND and F-C GRAVEL, tr. Silt, tr. Brick w/slag	25
15	S-3	3	2	2	5	24	8		GRAVELLY SAND	Top 2" Black C-F SAND and F-C GRAVEL, tr. Silt, tr. Brick w/slag Bottom 6" Brown F-C SAND, little m-f Gravel, tr. Silt	20
20	S-4	10	21	21	24	24	10			Brown C-F SAND, some c-f Gravel, little Silt	15
									WEATHERED BEDROCK		
25	C-1					60	59	98.3	BEDROCK	Gray fine-grained thickly bedded slightly fractured slightly weathered and strong SCHIST. Core times(min); 13, 11, 10, 10.5, 10	10
30	C-2					60	51	85		Gray fine-grained thickly bedded slightly fractured slightly weathered and strong SCHIST. Core times(min); 10, 9, 10, 9, 2.5	5
35										END OF BORING 34ft	0
40											-5

Sample Type: S = Split Spoon C = Core UP = Undisturbed Piston V = Vane Shear Test

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

Total Penetration in Earth: 22.5ft Rock: 11.5ft	NOTES: Black wash return from 10' to 17'. Difficult advancing past 22.5.' 4" drop during coring from 33' to 34', lost wash return.	Sheet 1 of 1
No. of Soil Samples: 4 No. of Core Runs: 2		SM-001-M REV. 1/02

STATE OF CONNECTICUT
DEPARTMENT OF TRANSPORTATION

MEMORANDUM

Subject: **Project 15-248**
Replacement of Bridge No. 325
U.S. 1 / Stillman's Pond
City of Bridgeport

Date: **June 12, 1997**

to: Mr. Carl F. Bard
Trans. Principal Engineer (Design)
Bureau of Engineering and
Highway Operations

from: Theodore M. Batko
Trans. Principal Engineer
Bureau of Engineering and
Highway Operations

This memorandum presents the Roadway Soils Report for the replacement of Bridge No. 325 carrying U.S. Route 1 over Stillman's Pond Brook. The report is based upon a preliminary alignment dated June 10, 1997. Structure borings have also been taken. Bridge Design will be requesting a Structure Soils and Foundation Report based upon this alignment.

Site

The project involves the replacement of a stone arch bridge carrying Route 1 over Stillman's Pond Brook and an abandoned railroad spur. Approximately 80 meters of roadway either side of the bridge will be reconstructed.

Geotechnical Investigation

Five borings were taken to determine the composition of the pavement structure, bearing stratum and ground water elevation. Borings terminated 3.7 meters below the surface in a fill with the exception of RB-2 which advanced to 2.75 meters. Boring RB-1 was taken to the west of the project and has differing conditions than the portion of roadway scheduled for reconstruction.

The subsurface conditions based upon borings RB-2, RB-3, RB-4, and RB-5, can be generalized as 100 to 200 mm of bituminous concrete over 150 to 220 mm of concrete on fill. The fill is moderately dense and is described as either a black gravely sand mixed with ash and cinders or a brown sandy gravel. Groundwater was not encountered in any of these roadway borings. Boring RB-2 terminated at 2.75 meters on a large boulder or rock.

Recommendations

The soils should provide adequate subgrade bearing capacity for a roadway. The absence of groundwater and depth to rock will preclude the need for underdrains.

Incorporate the boring logs into the contract plans.

If you have any questions please contact Mr. Bruce Olmstead, at extension 3186.

Attachments

Bruce A. Olmstead
cc: Joseph J. Obara
Colleen A. Kissane
Soils and Foundations

DWM
6/12/97

E.G. & R.P.	**SM - 001- M REV. 1/94** **BORING REPORT**	Hole No. RB-1
DRILLER	STATE OF CONNECTICUT	Line & Station
Keyes Associates {A.G.}	DEPARTMENT OF TRANSPORTATION	Offset
SUPERVISOR	TOWN: Bridgeport	N. Coordinate 40093 131538.71
Bruce Olmstead	PROJECT NAME: US 1 over Stillman's Brook	E. Coordinate 147374.5 4834[illegible]7.31
SOILS ENGINEER	PROJECT NUMBER: 15 - 248	State of Connecticut
	BORING CONTRACTOR: General Borings Inc.	DESIGN ENGINEER

		Casing				Auger		Mud	Sampler		Core Barrel			
Surface Elevation: 10.54														
Date Started: 2/1/96	Utilized						X		X					X
Date Finished: 2/1/96	Type	BW	NW	HW	Pipe	Solid	Hollow		SS	UP	B (s)	B (d)	NX (s)	NQ (d)
Groundwater Observations	Size I.D. (mm)	60	76	100	64		82.5		35		35	35	55	55
@ - m after 0 hours	Hammer (kg)	136	136	136	136			Bit	63.5		Type	X	Diamond	
@ m after hours	Fall (m)	0.6	0.6	0.6	0.6				0.76		of Bit		Carbide	

DEPTH	Casing blows per half meter	SAMPLE DEPTH IN METERS FROM - TO	NO.	PEN. m	REC. m	Type	BLOWS PER 0.15 METERS ON SAMPLER 0 - 0.15	0.15 - 0.30	0.30 - 0.45	0.45 - 0.60	STRATA CHANGE: DEPTH, ELEV.	FIELD IDENTIFICATION OF SOIL, REMARKS (INCL. COLOR, LOSS OF WASH WATER, ETC.)
											0.10	Bituminous Concrete
		0.30 - 0.90	1	0.60	0.15	D	10	24	22	26		
												Brown C-F GRAVEL AND SAND, trace silt
		1.50 - 2.10	2	0.60	0.30	D	10	12	13	18		
		3.10 - 3.70	3	0.60	0.40	D	5	9	14	18	3.70	
											6.84	
5												<u>End of Exploration</u>
10												
15												

Casing			Depth		NOTES:
Size	From	To	Earth	Rock	
			3.70 m	0.00 m	
			No. of Samples		
			3		

SAMPLE TYPE CODING: D=Driven C=Core A=Auger UP=Undisturbed Piston V=Vane Test	Hole No. RB-1
PROPORTIONS USED: Trace = 1 - 10%, Little = 10 - 20%, Some = 20 - 35%, And = 35 - 50%	Sheet 1 of 1

E.G. & R.P.	**SM - 001- M REV. 1/94** **BORING REPORT**	Hole No. RB-2
DRILLER	STATE OF CONNECTICUT	Line & Station 51+417.4
Keyes Associates {A.G.}	DEPARTMENT OF TRANSPORTATION	Offset 15.5m LT
SUPERVISOR	TOWN: Bridgeport	N. Coordinate 40097 131551.837
Bruce Olmstead	PROJECT NAME: US 1 over Stillman's Brook	E. Coordinate 147412 4836 28.6
SOILS ENGINEER	PROJECT NUMBER: 15 - 248	State of Connecticut
	BORING CONTRACTOR: General Borings Inc.	DESIGN ENGINEER

Surface Elevation: 10.57		Casing				Auger		Mud	Sampler		Core Barrel			
Date Started: 2/1/96	Utilized						X		X					X
Date Finished: 2/1/96	Type	BW	NW	HW	Pipe	Solid	Hollow		SS	UP	B (s)	B (d)	NX (s)	NQ (d)
Groundwater Observations	Size I.D. (mm)	60	76	100	64		82.5		35		35	35	55	55
@ - m after 0 hours	Hammer (kg)	136	136	136	136			Bit	63.5		Type	X	Diamond	
@ m after hours	Fall (m)	0.6	0.6	0.6	0.6				0.76		of Bit		Carbide	

DEPTH	Casing blows per half meter	SAMPLE DEPTH IN METERS FROM - TO	NO.	PEN. m	REC. m	Type	BLOWS PER 0.15 METERS ON SAMPLER 0 - 0.15	0.15 - 0.30	0.30 - 0.45	0.45 - 0.60	STRATA CHANGE: DEPTH, ELEV.	FIELD IDENTIFICATION OF SOIL, REMARKS (INCL. COLOR, LOSS OF WASH WATER, ETC.)
											0.40	0.25 m Bituminous on 0.15 m of Concrete
		0.50 - 1.10	1	0.60	0.30	D	4	4	4	4	10.17	Black F-C SAND, some f gravel, with ash, [Fill]
		1.50 - 2.10	2	0.60	0.30	D	4	7	9	11	1.70	
											8.87	Brown C-F GRAVEL AND SAND, trace silt
											2.75	
											7.82	<u>End of Exploration</u>
5												
10												
15												

Casing			Depth		NOTES:
Size	From	To	Earth	Rock	Hollow Auger Refusal at 2.75 m
			2.75 m	0.00 m	
			No. of Samples		
			2		

SAMPLE TYPE CODING: D=Driven C=Core A=Auger UP=Undisturbed Piston V=Vane Test

PROPORTIONS USED: Trace = 1 - 10%, Little = 10 - 20%, Some = 20 - 35%, And = 35 - 50%

Hole No.	RB-2
Sheet	1 of 1

E.G. & R.P.	**SM - 001- M REV. 1/94**	**BORING REPORT**	Hole No. RB-3
DRILLER	STATE OF CONNECTICUT		Line & Station 51 + 466.5
Keyes Associates {A.G.}	DEPARTMENT OF TRANSPORTATION		Offset 1.5m LT
SUPERVISOR	TOWN:	Bridgeport	N. Coordinate 40058 13142 3.88
Bruce Olmstead	PROJECT NAME:	US 1 over Stillman's Brook	E. Coordinate 147502 4859 23.85
SOILS ENGINEER	PROJECT NUMBER:	15 - 248	State of Connecticut
	BORING CONTRACTOR:	General Borings Inc.	DESIGN ENGINEER

		Casing				Auger		Mud	Sampler		Core Barrel			
Surface Elevation: 10.73														
Date Started: 2/1/96	Utilized						X		X					X
Date Finished: 2/1/96	Type	BW	NW	HW	Pipe	Solid	Hollow		SS	UP	B (s)	B (d)	NX (s)	NQ (d)
Groundwater Observations	Size I.D. (mm)	60	76	100	64		82.5		35		35	35	55	55
@ - m after 0 hours	Hammer (kg)	136	136	136	136			Bit	63.5		Type	X	Diamond	
@ m after hours	Fall (m)	0.6	0.6	0.6	0.6				0.76		of Bit		Carbide	

DEPTH	Casing blows per half meter	SAMPLE DEPTH IN METERS FROM - TO	NO.	PEN. m	REC. m	Type	BLOWS PER 0.15 METERS ON SAMPLER 0 - 0.15	0.15 - 0.30	0.30 - 0.45	0.45 - 0.60	STRATA CHANGE: DEPTH, ELEV.	FIELD IDENTIFICATION OF SOIL, REMARKS (INCL. COLOR, LOSS OF WASH WATER, ETC.)
											0.40	0.20 m Bituminous on 0.20 m of Concrete
		0.50 - 1.10	1	0.60	0.36	D	6	12	12	9	1.10	Brown F-C SAND, little f-m gravel, little silt
											9.63	
		1.50 - 2.10	2	0.60	0.30	D	6	5	5	6		Black F-C SAND, some f gravel, trace silt with ash, [Fill]
		3.10 - 2.70	3	0.60	0.45	D	5	4	4	5		
											3.70	
											7.03	
5												End of Exploration
10												
15												

Casing Size	From	To	Depth Earth	Rock	NOTES:
			3.70 m	0.00 m	Hollow Auger Refusal at 2.75 m
			No. of Samples		
			3		

SAMPLE TYPE CODING: D=Driven C=Core A=Auger UP=Undisturbed Piston V=Vane Test | Hole No. RB-3

PROPORTIONS USED: Trace = 1 - 10%, Little = 10 - 20%, Some = 20 - 35%, And = 35 - 50% | Sheet 1 of 1

E.G. & R.P.	**SM - 001- M REV. 1/94** **BORING REPORT**	Hole No. RB-4
DRILLER	STATE OF CONNECTICUT	Line & Station 51 + 514
Keyes Associates {A.G.}	DEPARTMENT OF TRANSPORTATION	Offset 0.2m LT
SUPERVISOR	TOWN: Bridgeport	N. Coordinate 40050 131397.638
Bruce Olmstead	PROJECT NAME: US 1 over Stillman's Brook	E. Coordinate 147527 454022.31
SOILS ENGINEER	PROJECT NUMBER: 15 - 248	State of Connecticut
	BORING CONTRACTOR: General Borings Inc.	DESIGN ENGINEER

		Casing				Auger		Mud	Sampler		Core Barrel			
Surface Elevation: 10.75	Utilized						X		X					X
Date Started: 2/1/96	Type	BW	NW	HW	Pipe	Solid	Hollow		SS	UP	B (s)	B (d)	NX (s)	NQ (d)
Date Finished: 2/1/96	Size I.D. (mm)	60	76	100	64		82.5		35		35	35	55	55
Groundwater Observations	Hammer (kg)	136	136	136	136			Bit	63.5		Type	X	Diamond	
@ - m after 0 hours	Fall (m)	0.6	0.6	0.6	0.6				0.76		of Bit		Carbide	
@ m after hours														

DEPTH	Casing blows per half meter	SAMPLE DEPTH IN METERS FROM - TO	NO.	PEN. m	REC. m	Type	BLOWS PER 0.15 METERS ON SAMPLER 0 - 0.15	0.15 - 0.30	0.30 - 0.45	0.45 - 0.60	STRATA CHANGE: DEPTH, ELEV.	FIELD IDENTIFICATION OF SOIL, REMARKS (INCL. COLOR, LOSS OF WASH WATER, ETC.)
											0.40	0.18 m Bituminous on 0.22 m concrete
		0.30 - 0.90	1	0.60	0.30	D	32	30	34	38		Brown C-F SAND, some c-f gravel, trace silt
		1.50 - 2.10	2	0.60	0.30	D	30	10	8	8		
		3.10 - 3.70	3	0.60	0.38	D	10	4	4	8	3.70	
											7.05	
5												<u>End of Exploration</u>
10												
15												

Casing			Depth		NOTES:
Size	From	To	Earth	Rock	
			3.70 m	0.00 m	
			No. of Samples		
			3		

SAMPLE TYPE CODING: D=Driven C=Core A=Auger UP=Undisturbed Piston V=Vane Test

PROPORTIONS USED: Trace = 1 - 10%, Little = 10 - 20%, Some = 20 - 35%, And = 35 - 50%

Hole No. RB-4
Sheet 1 of 1

E.G. & R.P.	**SM - 001- M REV. 1/94** **BORING REPORT**	Hole No. RB-5
DRILLER	STATE OF CONNECTICUT	Line & Station 51 + 539.2
Keyes Associates {A.G.}	DEPARTMENT OF TRANSPORTATION	Offset 5m RT
SUPERVISOR	TOWN: Bridgeport	N. Coordinate 147527
Bruce Olmstead	PROJECT NAME: US 1 over Stillman's Brook	E. Coordinate 40050
SOILS ENGINEER	PROJECT NUMBER: 15 - 248	State of Connecticut
	BORING CONTRACTOR: General Borings Inc.	DESIGN ENGINEER

		Casing				Auger		Mud	Sampler		Core Barrel			
Surface Elevation: 10.6														
Date Started: 2/1/96	Utilized						X		X					X
Date Finished: 2/1/96	Type	BW	NW	HW	Pipe	Solid	Hollow		SS	UP	B (s)	B (d)	NX (s)	NQ (d)
Groundwater Observations	Size I.D. (mm)	60	76	100	64		82.5		35		35	35	55	55
@ - m after 0 hours	Hammer (kg)	136	136	136	136			Bit	63.5		Type	X	Diamond	
@ m after hours	Fall (m)	0.6	0.6	0.6	0.6				0.76		of Bit		Carbide	

DEPTH	Casing blows per half meter	SAMPLE DEPTH IN METERS FROM - TO	NO.	PEN. m	REC. m	Type	BLOWS PER 0.15 METERS ON SAMPLER 0 - 0.15	0.15 - 0.30	0.30 - 0.45	0.45 - 0.60	STRATA CHANGE: DEPTH, ELEV.	FIELD IDENTIFICATION OF SOIL, REMARKS (INCL. COLOR, LOSS OF WASH WATER, ETC.)
											0.40	0.20 m Bituminous on 0.20 m concrete
		0.30 - 0.90	1	0.60	0.30	D	8	8	9	9		Black F-C SAND, little f-m gravel, little silt
											1.50	trace silt with ash, cinders, slag [Fill]
		1.50 - 2.10	2	0.60	0.15	D	21	6	3	4	9.10	
												Brown F-C SAND, little f gravel,
												little silt
		3.10 - 3.70	3	0.60	0.45	D	8	12	16	18	3.70	
											6.90	
5												End of Exploration
10												
15												

Casing Size	From	To	Depth Earth	Rock	NOTES:
			3.70 m	0.00 m	
			No. of Samples		
			3		

SAMPLE TYPE CODING: D=Driven C=Core A=Auger UP=Undisturbed Piston V=Vane Test

PROPORTIONS USED: Trace = 1 - 10%, Little = 10 - 20%, Some = 20 - 35%, And = 35 - 50%

Hole No. RB-5

Sheet 1 of 1

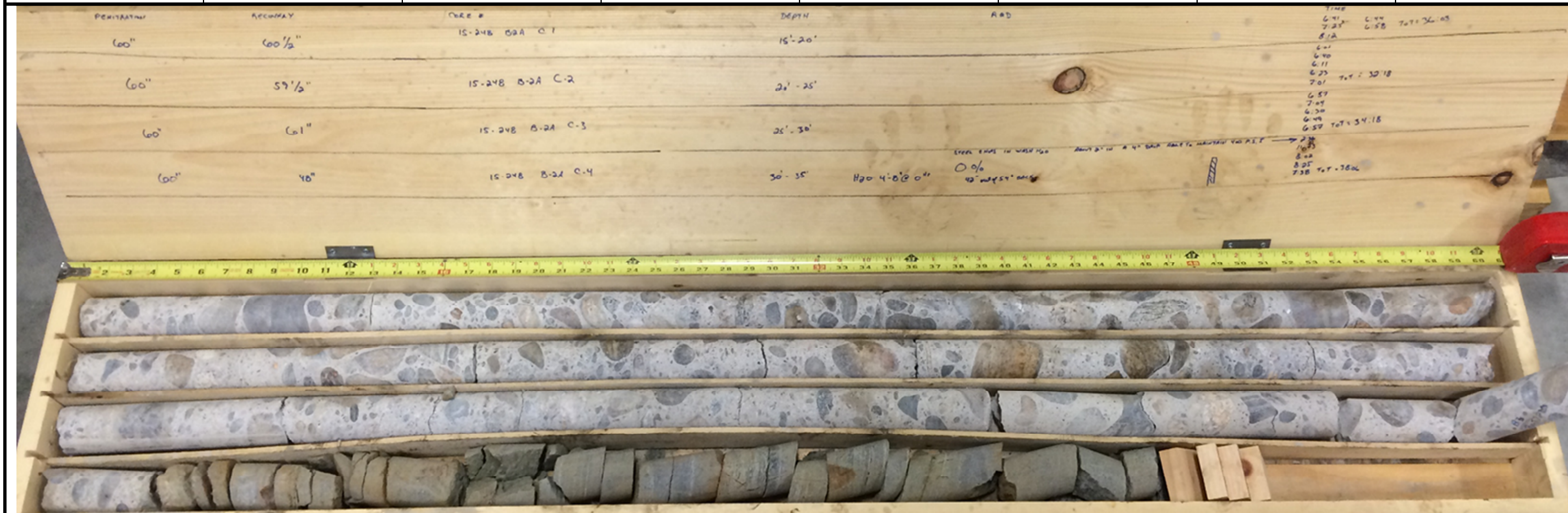
Rock Core Data Sheet

Project No.	Route	Description	Town	Driller	Inspector	Engineer	Date
15-248	1	Br. No. 00325	Bridgeport	J. Dorau	G. Arzt	B. McKiernan	8/28/2015

Boring No.	Sample No.	Sample Depth (ft)	Rock Type	Color	Grain Size	Bedding	Fracturing	Weathering	Strength	Drill Rates (min/ft)	Rec. (%)	RQD (%)
B-1	C-1	12'9"-17'9"	Concrete							10.5;9;8.5 8.5;7.5	98.3	
B-1	C-2	17'9"-22'9"	Concrete							8;9;8;8; 8.5	101.7	
B-1	C-3	22'9"-27'9"	Concrete							6.5;7.5;7.5 7;8	101.7	
B-1	C-4	27'9"-32'9"	Concrete over Schist	Gray	Fine	Thickly Bedded	Moderately	Slightly	Strong	7.5;9;11.5 11;9.5	105	100

Rock Core Data Sheet

Project No.	Route	Description	Town	Driller	Inspector	Engineer	Date
15-248	1	Br. No. 00325	Bridgeport	J. Dorau	G. Arzt	B. McKiernan	8/31/2015



Boring No.	Sample No.	Sample Depth (ft)	Rock Type	Color	Grain Size	Bedding	Fracturing	Weathering	Strength	Drill Rates (min/ft)	Rec. (%)	RQD (%)
B-2A	C-1	15-20	Concrete							6.5;7.5;8; 6.5;7	100.8	
B-2A	C-2	20-25	Concrete							6;6.5;6; 6.5;7	99.2	
B-2A	C-3	25-30	Concrete							7;7;6.5;7; 7	101.7	
B-2A	C-4	30-35	Concrete over Gneiss	Gray	Fine	Medium Bedded	Intensely	Moderately	Medium Strong	2.5;11.5;8 8.5;7.5	80	0

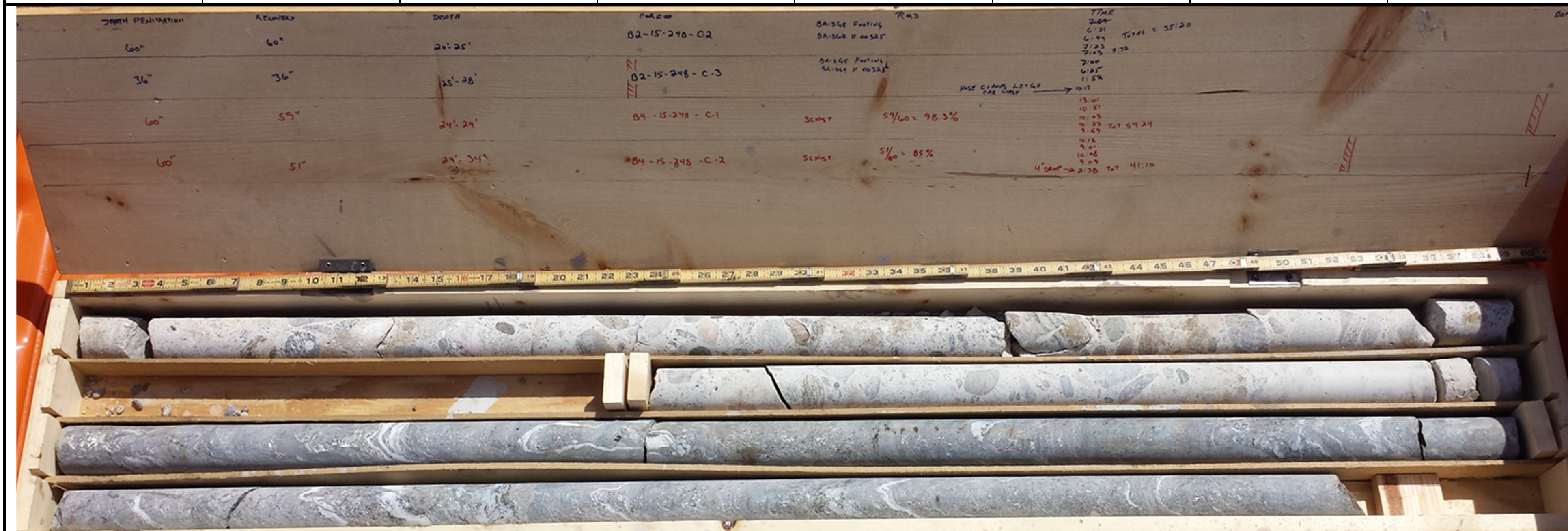
Rock Core Data Sheet

Project No.	Route	Description	Town	Driller	Inspector	Engineer	Date
15-248	1	Br. No. 00325	Bridgeport	J. Dorau	B.McKiernan	B. McKiernan	8/24/2015

Boring No.	Sample No.	Sample Depth (ft)	Rock Type	Color	Grain Size	Bedding	Fracturing	Weathering	Strength	Drill Rates (min/ft)	Rec. (%)	RQD (%)
B-3	C-1	27-32	Schist/Gneiss	Gray	Fine	Medium Bedded	Intensely	Highly	Medium Strong	2.5;2.5;3; 9.5;3	23.3	0
B-3	C-2	32-37	Schist/Gneiss	Gray	Fine	Thickly Bedded	Highly	Moderately	Strong	5;4;8;8.5; 8	61.7	17.5
B-3	C-3	37-42	Gneiss	Gray	Fine	Thickly Bedded	Moderately	Slightly	Strong	5;6;7;7.5; 9.5	88.3	85
B-2	C-1	15-20	Concrete							5.5;5;8;8; 8	100	

Rock Core Data Sheet

Project No.	Route	Description	Town	Driller	Inspector	Engineer	Date
15-248	1	Br. No. 00325	Bridgeport	J. Dorau	G. Arzt	B. McKiernan	8/27/2015



Boring No.	Sample No.	Sample Depth (ft)	Rock Type	Color	Grain Size	Bedding	Fracturing	Weathering	Strength	Drill Rates (min/ft)	Rec. (%)	RQD (%)
B-2	C-2	20-25	Concrete							6.5; 7; 7.5; 7; 7.5	100	
B-2	C-3	25-28	Concrete							6.5;7; 7	100	
B-4	C-1	24-29	Schist	Gray	Fine	Thickly Bedded	Slightly	Slightly	Strong	13;11;10; 10.5;10	98.3	98.3
B-4	C-2	29-34	Schist	Gray	Fine	Thickly Bedded	Slightly	Slightly	Strong	10;9;10;9; 2.5	85	85

Route 1 over Stillman Pond, Bridgeport
DOT Bridge Design Request
Test Pits, 4-17-2018
Onsite Personnel: A. Hare, L. Arno (Soils), Survey Crew, Maintenance Crew (D-3)
Contact: D-3 Stratford, (Joe Hunihan) and D-3 Surveys (Frank Hamm)

*Test pits from WW1A, RW1A, RW1B and WW1B were not completed due to steep slopes and access issues.

1) TP-1
   a. Retaining Wall 2A
   b. Approximate size of test pit: 4.5' x 9', 8' deep
   c. Footing approx.. 5.5 feet thick, approx. 12" from face of RW.
   d. Surveys: 2 shots.

2) TP-2
   a. WingWall 2A
   b. Approximate size of test pit: 10’ x 12’, 14’ deep
   c. No defined footing encountered, no bottom of wall encountered.
   d. Surveys: No data collected.

3) TP-3
   a. Wing Wall 2B
   b. Approximate size of test pit: 4’ x 15’, 13’ deep
   c. No defined footing encountered, no bottom of wall encountered.
   d. Surveys: No data collected.

4) TP-4
   a. Retaining Wall 2B
   b. Approximate size of test pit: 4'x 15', 10' deep
   c. Footing approx.. 4 feet thick, offset approx. 12" from wall.
   d. Surveys: 2 shots.

GRADATION ANALYSIS

PROJECT NO:	**15-248**	TARE NO:	
TOWN:	**Bridgeport**	TARE WT:	
BORING NO:	**B-1**	WET WT. W/TARE:	
DEPTH:	**5' -7'**	DRY WT. W/TARE:	**100.0**
DATE TESTED:	**NA**	DRY WT. SOIL:	100.0
TEST BY:	**A.Hare**	WATER WT:	-100.0
		WATER CONTENT %:	-100.0

SIEVE SIZE	mm	CUMUL. WEIGHT (g)	RETAINED WEIGHT (g)	PERCENT RETAINED	PERCENT FINER
1"	25.4	**0.0**	0.0	0.0%	100.0%
1/2"	12.7	**0.0**	0.0	0.0%	100.0%
1/4"	6.35	**0.0**	0.0	0.0%	100.0%
#10	2.00	**10.0**	10.0	10.0%	90.0%
#20	0.84	**10.0**	0.0	0.0%	90.0%
#40	0.42	**10.0**	0.0	0.0%	90.0%
#60	0.25	**75.0**	65.0	65.0%	25.0%
#100	0.149	**75.0**	0.0	0.0%	25.0%
#140	0.105	**75.0**	0.0	0.0%	25.0%
#200	0.074	**75.0**	0.0	0.0%	25.0%
PAN		**100.0**	25.0	25.0%	0.0%
		TOTAL WT:	100.0		
		Δ SAMPLE:	0.0	0.0%	

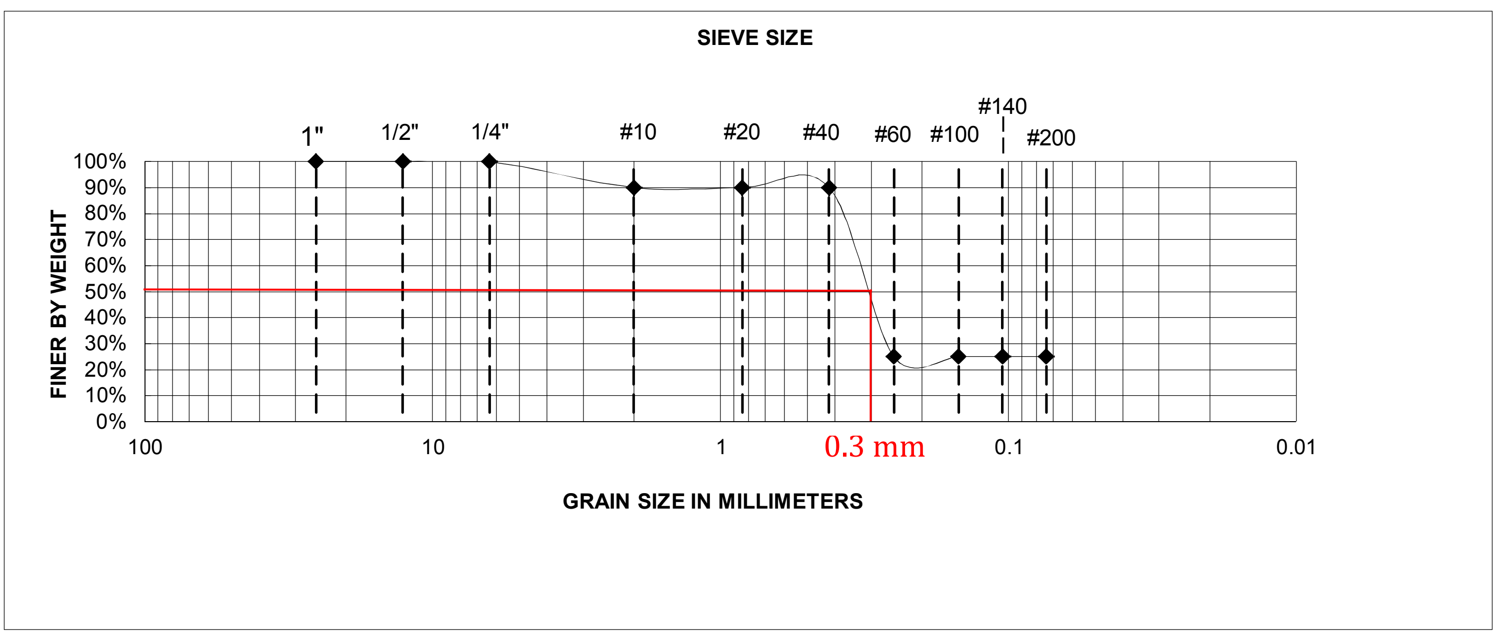
C&MG	0.0%
FG	10.0%
TOT G	10.0%
CS	0.0%
FS	65.0%
TOT S	65.0%
S&C	25.0%
TOT S&C	25.0%

DESCRIPTION

COLOR: **Brown**

trace	F-C	Gravel
	F-C	SAND
some		Silt

	AND	some	little	trace
>50%	35-50%	20-35%	10-20%	1-10%



AASHTO	GRAVEL			SAND		SILT / CLAY
	COARSE	MEDIUM	FINE	COARSE	FINE	

SAMPLE DESCRIPTION	LL	PL	PI
Brown F-C SAND, some Silt, trace F-C Gravel			

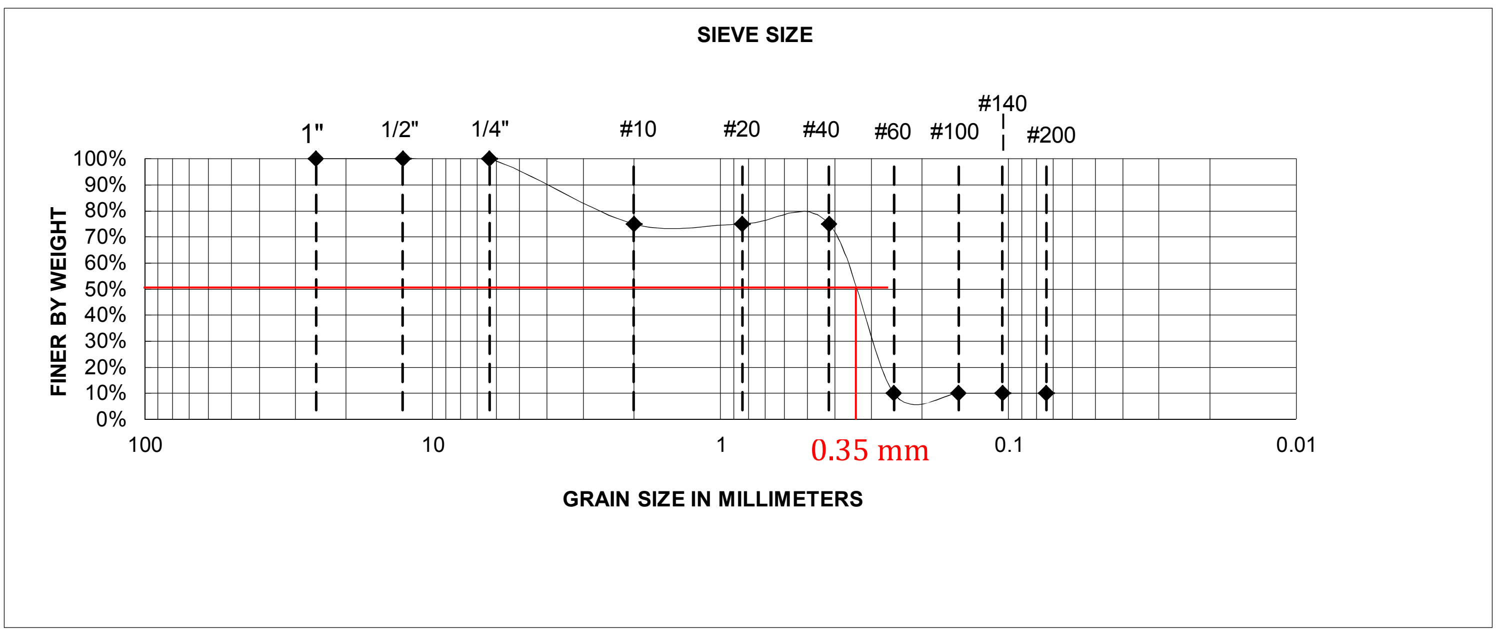
GRADATION ANALYSIS

PROJECT NO:	**15-248**	TARE NO:	
TOWN:	**Bridgeport**	TARE WT:	
BORING NO:	**B-1**	WET WT. W/TARE:	
DEPTH:	**15' - 17'**	DRY WT. W/TARE:	**100.0**
DATE TESTED:	**NA**	DRY WT. SOIL:	100.0
TEST BY:	**A.Hare**	WATER WT:	-100.0
		WATER CONTENT %:	-100.0

SIEVE SIZE	mm	CUMUL. WEIGHT (g)	RETAINED WEIGHT (g)	PERCENT RETAINED	PERCENT FINER		
1"	25.4	**0.0**	0.0	0.0%	100.0%	C&MG	0.0%
1/2"	12.7	**0.0**	0.0	0.0%	100.0%	FG	25.0%
1/4"	6.35	**0.0**	0.0	0.0%	100.0%	TOT G	25.0%
#10	2.00	**25.0**	25.0	25.0%	75.0%	CS	0.0%
#20	0.84	**25.0**	0.0	0.0%	75.0%	FS	65.0%
#40	0.42	**25.0**	0.0	0.0%	75.0%	TOT S	65.0%
#60	0.25	**90.0**	65.0	65.0%	10.0%	S&C	10.0%
#100	0.149	**90.0**	0.0	0.0%	10.0%	TOT S&C	10.0%
#140	0.105	**90.0**	0.0	0.0%	10.0%		
#200	0.074	**90.0**	0.0	0.0%	10.0%		
PAN		**100.0**	10.0	10.0%	0.0%		
		TOTAL WT:	100.0				
		Δ SAMPLE:	0.0	0.0%			

DESCRIPTION		
COLOR:	**Brown**	
some	F-C	Gravel
	F-C	SAND
trace		Silt

	AND	some	little	trace
>50%	35-50%	20-35%	10-20%	1-10%



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<tr><td rowspan="2">AASHTO</td><td colspan="3">GRAVEL</td><td colspan="2">SAND</td><td rowspan="2">SILT / CLAY</td></tr>
<tr><td>COARSE</td><td>MEDIUM</td><td>FINE</td><td>COARSE</td><td>FINE</td></tr>
</table>

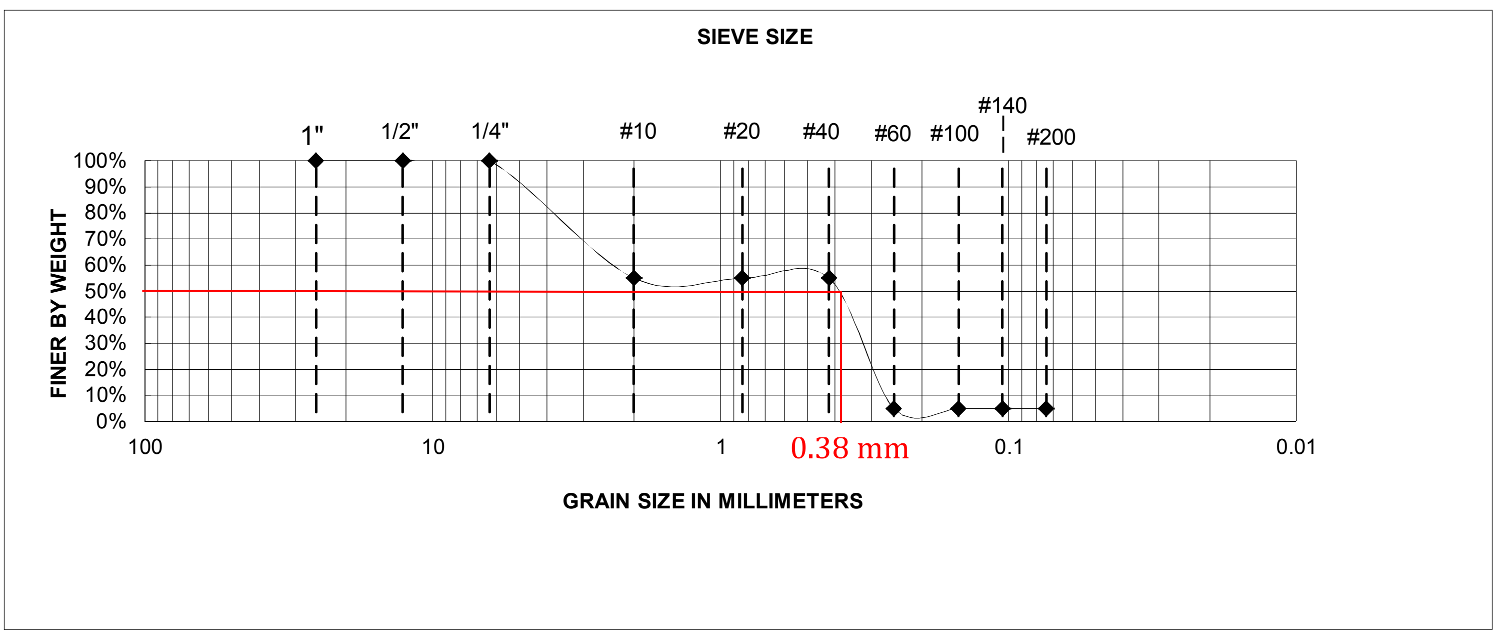
SAMPLE DESCRIPTION	LL	PL	PI
Brown F-C SAND, some F-C Gravel, trace Silt			

GRADATION ANALYSIS

PROJECT NO:	15-248	TARE NO:	
TOWN:	Bridgeport	TARE WT:	
BORING NO:	B-2	WET WT. W/TARE:	
DEPTH:	5' -7'	DRY WT. W/TARE:	100.0
DATE TESTED:	NA	DRY WT. SOIL:	100.0
TEST BY:	A.Hare	WATER WT:	-100.0
		WATER CONTENT %:	-100.0

SIEVE SIZE	mm	CUMUL. WEIGHT (g)	RETAINED WEIGHT (g)	PERCENT RETAINED	PERCENT FINER
1"	25.4	0.0	0.0	0.0%	100.0%
1/2"	12.7	0.0	0.0	0.0%	100.0%
1/4"	6.35	0.0	0.0	0.0%	100.0%
#10	2.00	45.0	45.0	45.0%	55.0%
#20	0.84	45.0	0.0	0.0%	55.0%
#40	0.42	45.0	0.0	0.0%	55.0%
#60	0.25	95.0	50.0	50.0%	5.0%
#100	0.149	95.0	0.0	0.0%	5.0%
#140	0.105	95.0	0.0	0.0%	5.0%
#200	0.074	95.0	0.0	0.0%	5.0%
PAN		100.0	5.0	5.0%	0.0%
		TOTAL WT:	100.0		
		Δ SAMPLE:	0.0	0.0%	

		DESCRIPTION		
		COLOR:	Brown	
C&MG	0.0%			
FG	45.0%			
TOT G	45.0%	AND	F-C	GRAVEL
CS	0.0%			
FS	50.0%			
TOT S	50.0%	AND	F-C	SAND
S&C	5.0%			
TOT S&C	5.0%	trace		Silt
	AND	some	little	trace
>50%	35-50%	20-35%	10-20%	1-10%



AASHTO	GRAVEL			SAND		SILT / CLAY
	COARSE	MEDIUM	FINE	COARSE	FINE	

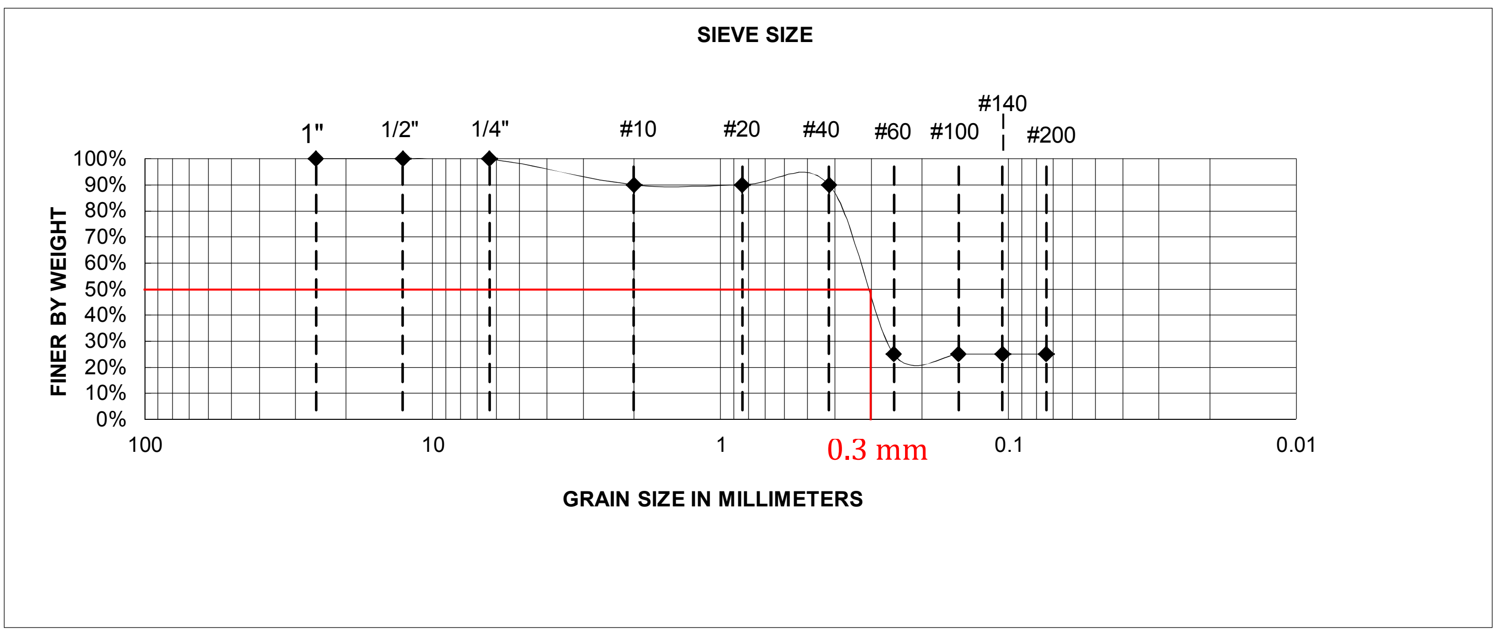
SAMPLE DESCRIPTION	LL	PL	PI
Brown F-C SAND, and F-C GRAVEL, trace Silt			

GRADATION ANALYSIS

PROJECT NO:	**15-248**	TARE NO:	
TOWN:	**Bridgeport**	TARE WT:	
BORING NO:	**RB-4**	WET WT. W/TARE:	
DEPTH:	**15' - 18'**	DRY WT. W/TARE:	**100.0**
DATE TESTED:	**NA**	DRY WT. SOIL:	100.0
TEST BY:	**A.Hare**	WATER WT:	-100.0
		WATER CONTENT %:	-100.0

SIEVE SIZE	mm	CUMUL. WEIGHT (g)	RETAINED WEIGHT (g)	PERCENT RETAINED	PERCENT FINER
1"	25.4	**0.0**	0.0	0.0%	100.0%
1/2"	12.7	**0.0**	0.0	0.0%	100.0%
1/4"	6.35	**0.0**	0.0	0.0%	100.0%
#10	2.00	**10.0**	10.0	10.0%	90.0%
#20	0.84	**10.0**	0.0	0.0%	90.0%
#40	0.42	**10.0**	0.0	0.0%	90.0%
#60	0.25	**75.0**	65.0	65.0%	25.0%
#100	0.149	**75.0**	0.0	0.0%	25.0%
#140	0.105	**75.0**	0.0	0.0%	25.0%
#200	0.074	**75.0**	0.0	0.0%	25.0%
PAN		**100.0**	25.0	25.0%	0.0%
		TOTAL WT:	100.0		
		Δ SAMPLE:	0.0	0.0%	

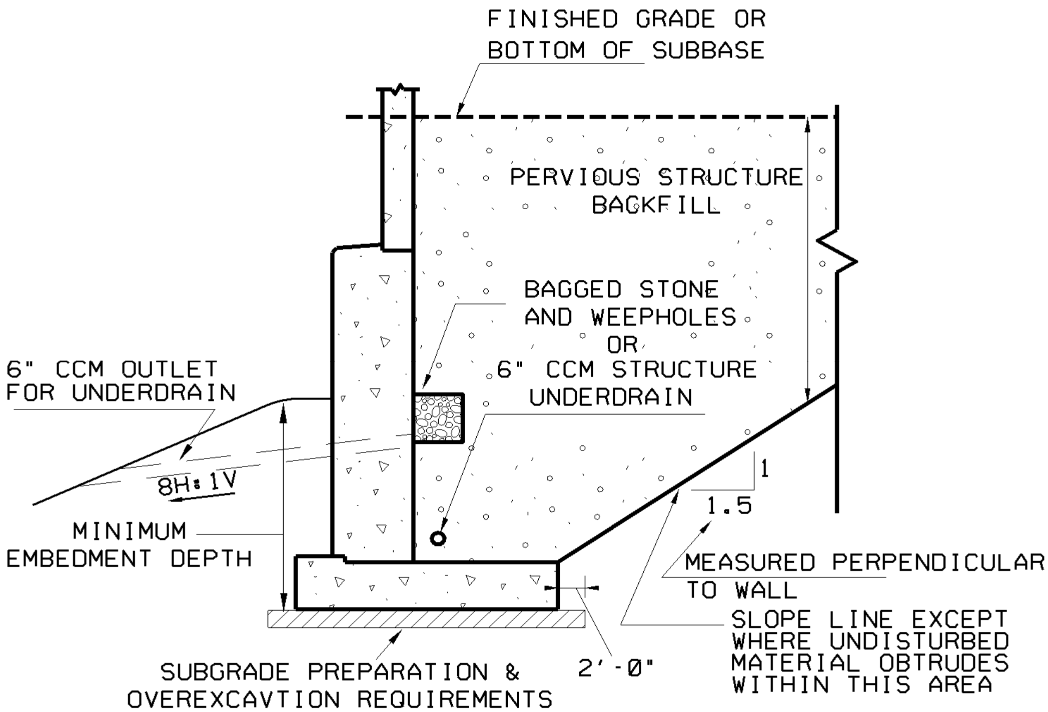
		DESCRIPTION		
		COLOR:	**Brown**	
C&MG	0.0%			
FG	10.0%			
TOT G	10.0%	trace	F-C	Gravel
CS	0.0%			
FS	65.0%			
TOT S	65.0%		F-C	SAND
S&C	25.0%			
TOT S&C	25.0%	some		Silt
	AND	some	little	trace
>50%	35-50%	20-35%	10-20%	1-10%



AASHTO	GRAVEL			SAND		SILT / CLAY
	COARSE	MEDIUM	FINE	COARSE	FINE	

SAMPLE DESCRIPTION	LL	PL	PI
Brown F-C SAND, some Silt, trace F-C Gravel			

Geotechnical Wall Design Parameters
Project No. 15-248
Retaining Wall No. 101



Factored Resistances

	Strength Limit	Service Limit
Bearing	6.2 tsf	2.63 tsf
Sliding	0.6V	0.6V

V=total vertical force

Lateral Earth Loads

Soil Unit Weight, γ :	125 pcf
Lateral Earth Pressure(static):	33 psf
Live Load Surcharge-Uniform Earth Pressure*:	$0.27\gamma\ h_{eq}$

*h_{eq} based on AASHTO-LRFD Table 3.11.6.4-2

Foundation Design Details

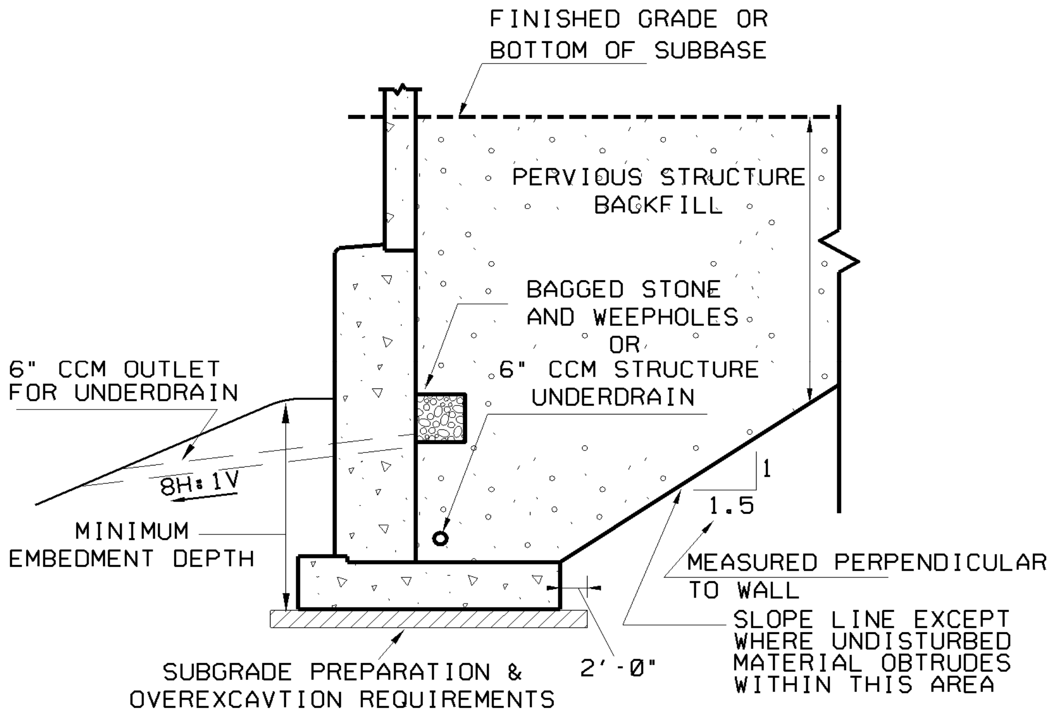
Minimum Embedment Depth:	4 ft.
Backwall Drainage:	Weepholes and Bagged Stone
Subgrade Preparation:	1ft of Granular Fill
Maximum Temporary Cut Slope:	1(V):1.5(H)

Additional Comments

Design recommendations based on 2007 *AASHTO LRFD* and *ConnDOT Bridge Design Manual*.

- Preliminary plans provided show the bottom of footing to be elevation 20.0±.
- Include the following logs on the wall plan sheets: B-1, RB-3, B-4

Geotechnical Wall Design Parameters
Project No. 15-248
WingWall No. 1A



Factored Resistances

	Strength Limit	Service Limit
Bearing	10 tsf	6.25 tsf
Sliding	0.6V	0.6V

V=total vertical force

Lateral Earth Loads

Soil Unit Weight, γ :	125 pcf
Lateral Earth Pressure(static):	33 psf
Live Load Surcharge-Uniform Earth Pressure*:	$0.27\gamma\ h_{eq}$

*h_{eq} based on AASHTO-LRFD Table 3.11.6.4-2

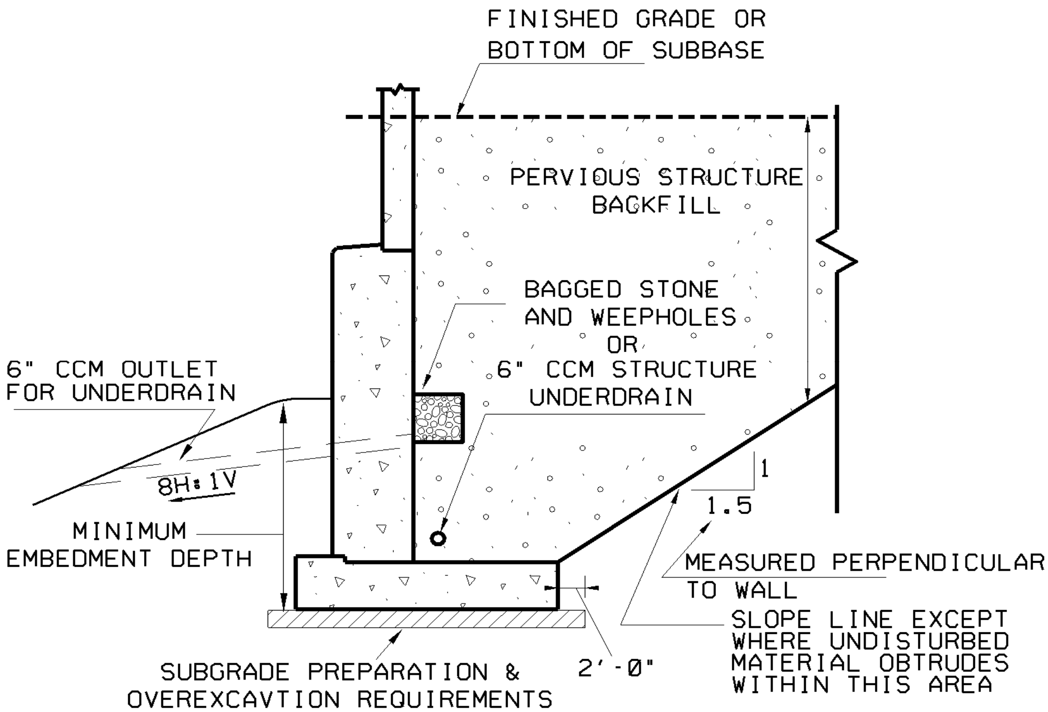
Foundation Design Details

Minimum Embedment Depth:	4 ft.
Backwall Drainage:	Bagged Stone and Weepholes
Subgrade Preparation:	1ft of Granular Fill
Maximum Temporary Cut Slope:	1(V):1.5(H)

Additional Comments

- Design recommendations based on 2007 *AASHTO LRFD* and *ConnDOT Bridge Design Manual*.
- Preliminary plans provided show the bottom of footing to be at elevation 12.5±.
- Include the following logs on the wall plan sheets: RB-3, B-1, B-4

Geotechnical Wall Design Parameters
Project No. 15-248
WingWall No. 1B (Step 1)



Factored Resistances

	Strength Limit	Service Limit
Bearing	7.4 tsf	7.75 tsf
Sliding	0.6V	0.6V

V=total vertical force

Lateral Earth Loads

Soil Unit Weight, γ :	125 pcf
Lateral Earth Pressure(static):	33 psf
Live Load Surcharge-Uniform Earth Pressure*:	$0.27\gamma\ h_{eq}$

*h_{eq} based on AASHTO-LRFD Table 3.11.6.4-2

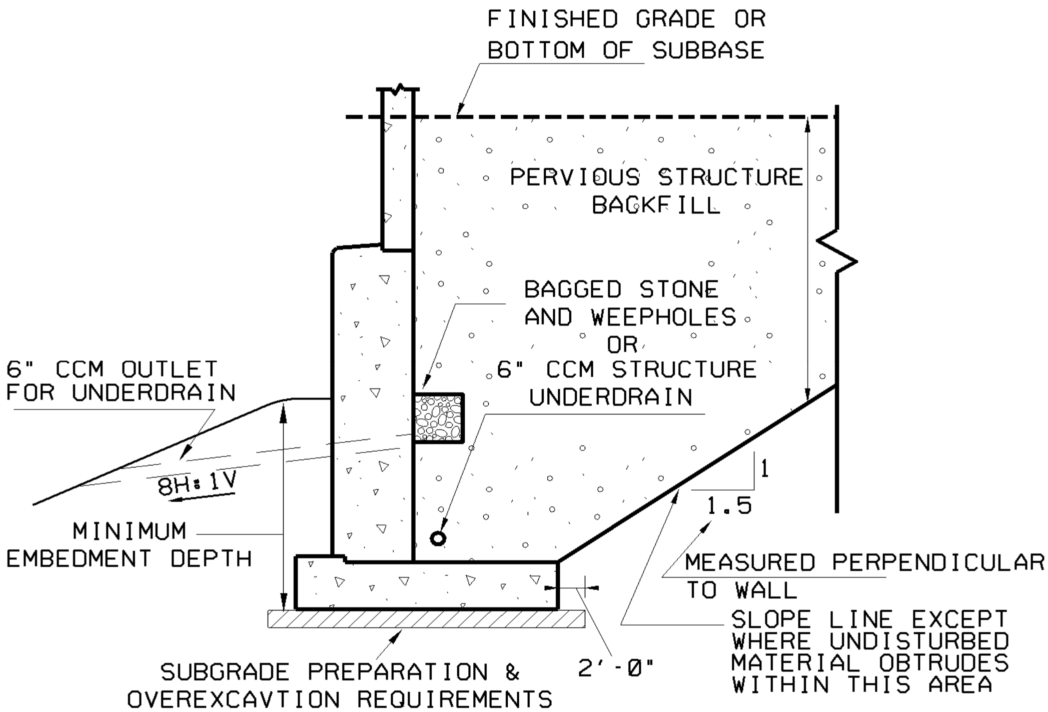
Foundation Design Details

Minimum Embedment Depth:	4 ft.
Backwall Drainage:	Bagged Stone and Weepholes
Subgrade Preparation:	1ft of Granular Fill
Maximum Temporary Cut Slope:	1(V):1.5(H)

Additional Comments

- Design recommendations based on 2007 *AASHTO LRFD* and *ConnDOT Bridge Design Manual*.
- Preliminary plans provided show the bottom of footing to be at elevation 12.0±.
- Include the following logs on the wall plan sheets: RB-3, B-1, B-4

Geotechnical Wall Design Parameters
Project No. 15-248
WingWall No. 1B (Step 2)



Factored Resistances

	Strength Limit	Service Limit
Bearing	5.3 tsf	5 tsf
Sliding	0.6V	0.6V

V=total vertical force

Lateral Earth Loads

Soil Unit Weight, γ :	125 pcf
Lateral Earth Pressure(static):	33 psf
Live Load Surcharge-Uniform Earth Pressure*:	$0.27 \gamma h_{eq}$

*h_{eq} based on AASHTO-LRFD Table 3.11.6.4-2

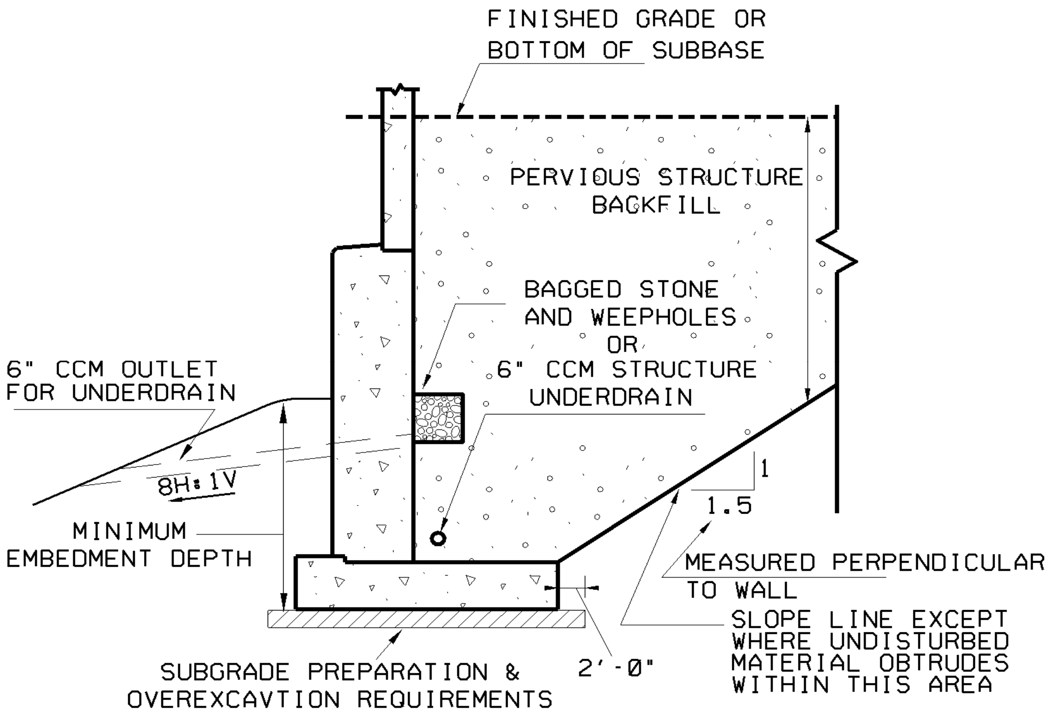
Foundation Design Details

Minimum Embedment Depth:	4 ft.
Backwall Drainage:	Bagged Stone and Weepholes
Subgrade Preparation:	1ft of Granular Fill
Maximum Temporary Cut Slope:	1(V):1.5(H)

Additional Comments

- Design recommendations based on 2007 *AASHTO LRFD* and *ConnDOT Bridge Design Manual*.
- Preliminary plans provided show the bottom of footing to be at elevation 19.5±.
- Include the following logs on the wall plan sheets: RB-3, B-1, B-4

Geotechnical Wall Design Parameters
Project No. 15-248
WingWall No. 1B (Step 3)



Factored Resistances

	Strength Limit	Service Limit
Bearing	4.5 tsf	2 tsf
Sliding	0.6V	0.6V

V=total vertical force

Lateral Earth Loads

Soil Unit Weight, γ :	125 pcf
Lateral Earth Pressure(static):	33 psf
Live Load Surcharge-Uniform Earth Pressure*:	$0.27\,\gamma\,h_{eq}$

*h_{eq} based on AASHTO-LRFD Table 3.11.6.4-2

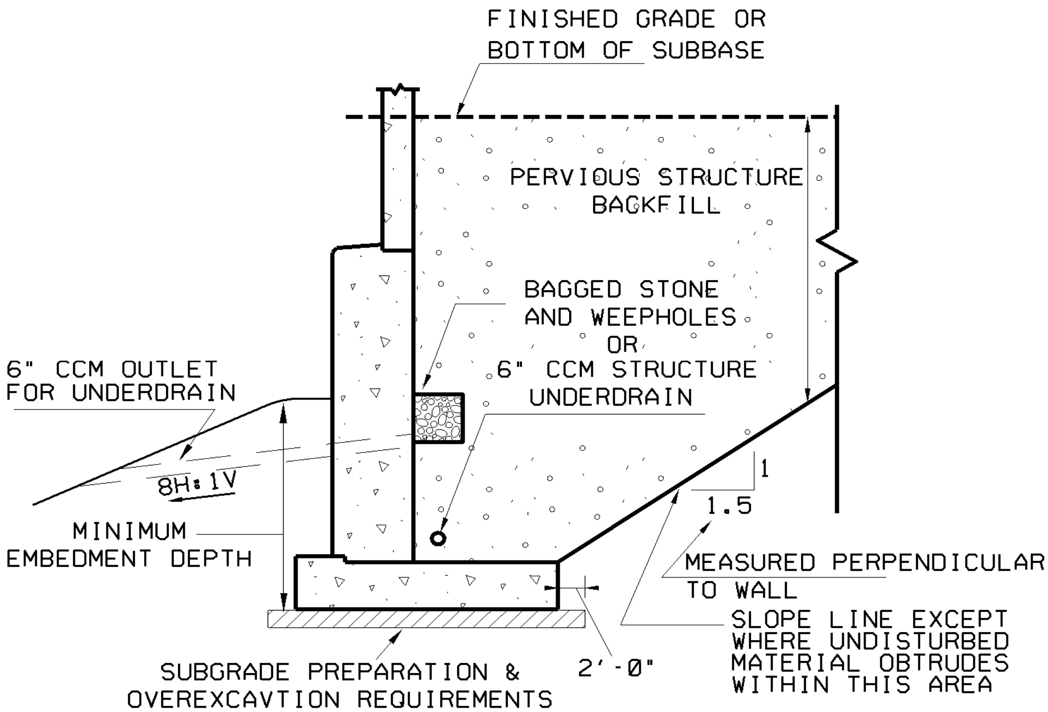
Foundation Design Details

Minimum Embedment Depth:	4 ft.
Backwall Drainage:	Bagged Stone and Weepholes
Subgrade Preparation:	2 ft of Granular Fill
Maximum Temporary Cut Slope:	1(V):1.5(H)

Additional Comments

- Design recommendations based on 2007 *AASHTO LRFD* and *ConnDOT Bridge Design Manual*.
- Preliminary plans provided show the bottom of footing to be at elevation 26.8$\pm$.
- Include the following logs on the wall plan sheets: RB-3, B-1, B-4

Geotechnical Wall Design Parameters
Project No. 15-248
Retaining Wall No. 102



Factored Resistances

	Strength Limit	Service Limit
Bearing	7.6 tsf	2.5 tsf
Sliding	0.6V	0.6V

V=total vertical force

Lateral Earth Loads

Soil Unit Weight, γ :	125 pcf
Lateral Earth Pressure(static):	33 psf
Live Load Surcharge-Uniform Earth Pressure*:	$0.27\gamma\ h_{eq}$

*h_{eq} based on AASHTO-LRFD Table 3.11.6.4-2

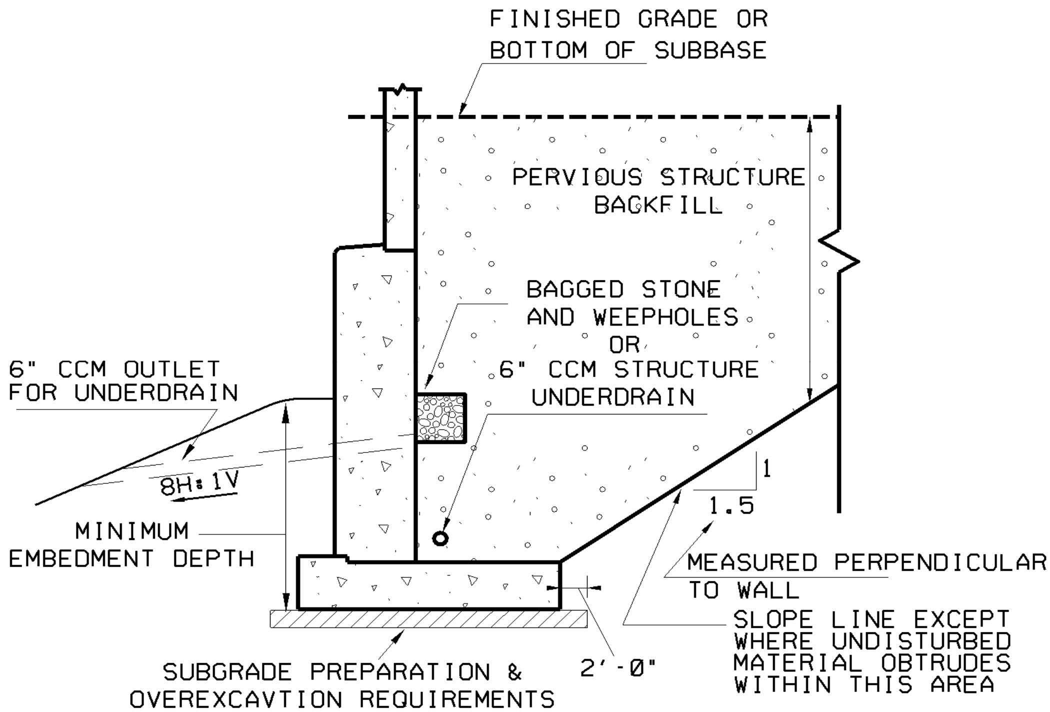
Foundation Design Details

Minimum Embedment Depth:	4 ft.
Backwall Drainage:	Bagged Stone and Weepholes
Subgrade Preparation:	2ft of Granular Fill
Maximum Temporary Cut Slope:	1(V):1.5(H)

Additional Comments

- Design recommendations based on 2007 *AASHTO LRFD* and *ConnDOT Bridge Design Manual*.
- Preliminary plans provided show the bottom of footing to be at elevation 18.0±.
- Include the following logs on the wall plan sheets: B-2, B-3, RB-4

Geotechnical Wall Design Parameters
Project No. 15-248
Wingwall No. 2A (STEP 1)



Factored Resistances

	Strength Limit	Service Limit
Bearing	7.6 tsf	4.0 tsf
Sliding	0.6V	0.6V

V=total vertical force

Lateral Earth Loads

Soil Unit Weight, γ :	125 pcf
Lateral Earth Pressure(static):	33 psf
Live Load Surcharge-Uniform Earth Pressure*:	$0.3\ \gamma\ h_{eq}$

*h_{eq} based on AASHTO-LRFD Table 3.11.6.4-2

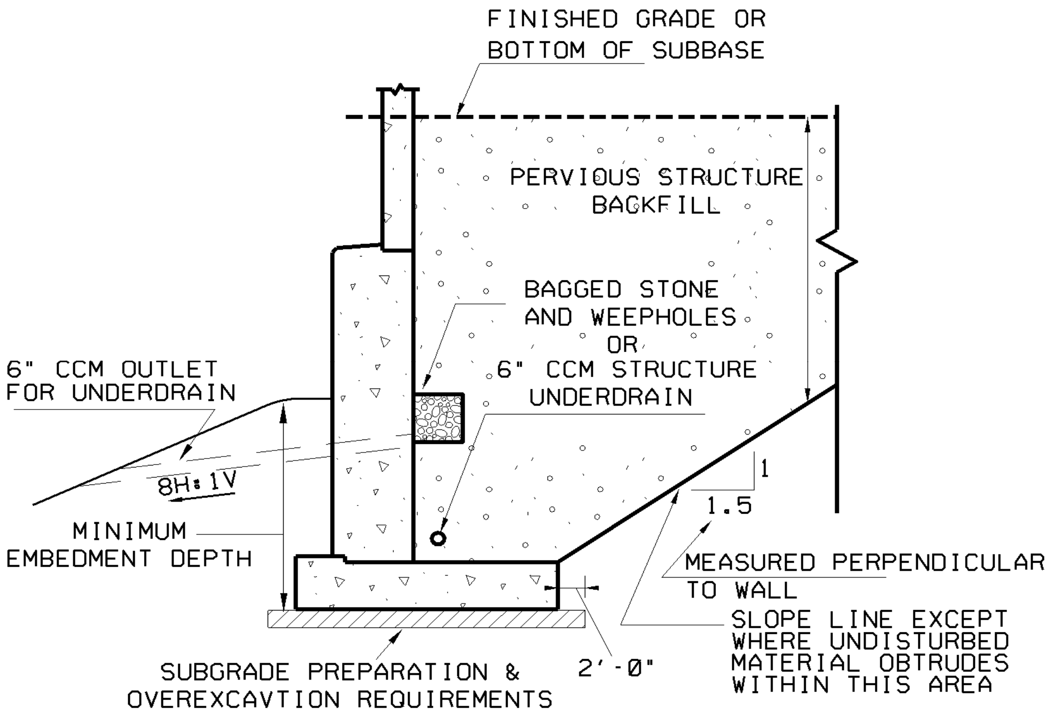
Foundation Design Details

Minimum Embedment Depth:	4 ft.
Backwall Drainage:	Bagged Stone and Weepholes
Subgrade Preparation:	1ft of Granular Fill
Maximum Temporary Cut Slope:	1(V):1.5(H)

Additional Comments

- Design recommendations based on 2007 *AASHTO LRFD* and *ConnDOT Bridge Design Manual*.
- Preliminary plans provided show the bottom of footing to be at elevation 14$\pm$.
- Include the following logs on the wall plan sheets: RB-4, B-2, B-3

Geotechnical Wall Design Parameters
Project No. 15-248
Wingwall No. 2A (STEP 2)



Factored Resistances

	Strength Limit	Service Limit
Bearing	7.5 tsf	2.6 tsf
Sliding	0.6V	0.6V

V=total vertical force

Lateral Earth Loads

Soil Unit Weight, γ :	125 pcf
Lateral Earth Pressure(static):	33 psf
Live Load Surcharge-Uniform Earth Pressure*:	$0.3\ \gamma\ h_{eq}$

*h_{eq} based on AASHTO-LRFD Table 3.11.6.4-2

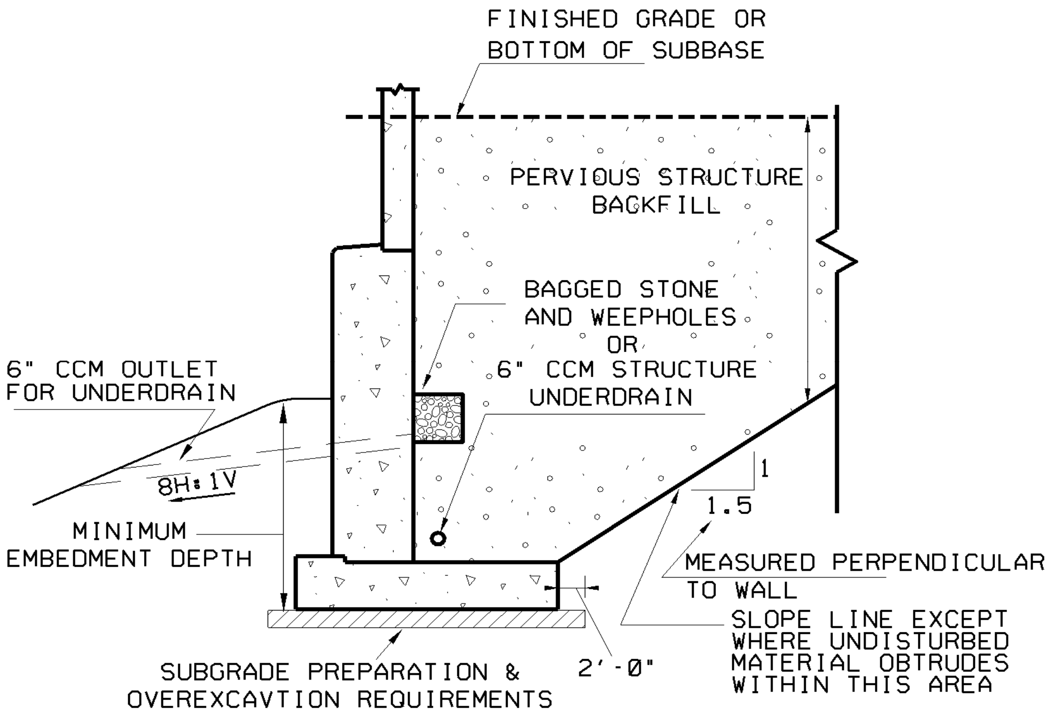
Foundation Design Details

Minimum Embedment Depth:	4 ft.
Backwall Drainage:	Bagged Stone and Weepholes
Subgrade Preparation:	2ft of Granular Fill
Maximum Temporary Cut Slope:	1(V):1.5(H)

Additional Comments

- Design recommendations based on 2007 *AASHTO LRFD* and *ConnDOT Bridge Design Manual*.
- Preliminary plans provided show the bottom of footing to be at elevation 18±.
- Include the following logs on the wall plan sheets: RB-4, B-2, B-3

Geotechnical Wall Design Parameters
Project No. 15-248
Retaining Wall No. 103



Factored Resistances

	Strength Limit	Service Limit
Bearing	9.6 tsf	4 tsf
Sliding	0.6V	0.6V

V=total vertical force

Lateral Earth Loads

Soil Unit Weight, γ :	125 pcf
Lateral Earth Pressure(static):	33 psf
Live Load Surcharge-Uniform Earth Pressure*:	$0.27\ \gamma\ h_{eq}$

*h_{eq} based on AASHTO-LRFD Table 3.11.6.4-2

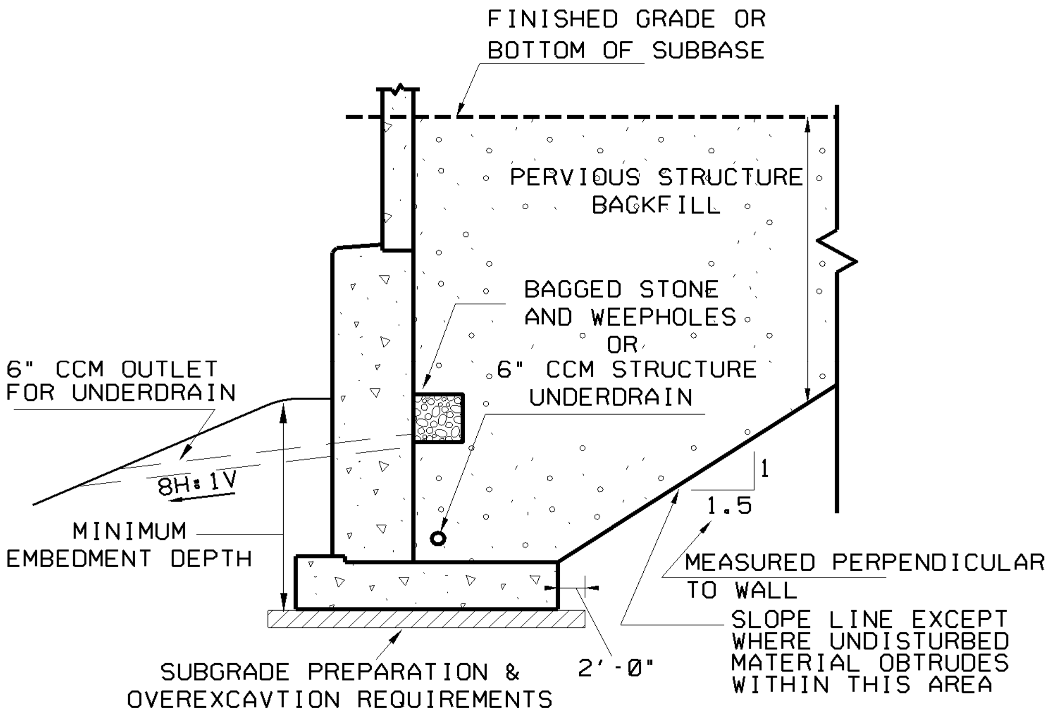
Foundation Design Details

Minimum Embedment Depth:	4 ft.
Backwall Drainage:	Bagged Stone and Weepholes
Subgrade Preparation:	1ft of Granular Fill
Maximum Temporary Cut Slope:	1(V):1.5(H)

Additional Comments

- Design recommendations based on 2007 *AASHTO LRFD* and *ConnDOT Bridge Design Manual*.
- Preliminary plans provided show the bottom of footing to be at elevation 15$\pm$.
- Include the following logs on the wall plan sheets: B-2, RB-5

Geotechnical Wall Design Parameters
Project No. 15-248
Wingwall No. 2B



Factored Resistances

	Strength Limit	Service Limit
Bearing	20 tsf	10 tsf
Sliding	0.6V	0.6V

V=total vertical force

Lateral Earth Loads

Soil Unit Weight, γ :	125 pcf
Lateral Earth Pressure(static):	33 psf
Live Load Surcharge-Uniform Earth Pressure*:	$0.27\,\gamma\,h_{eq}$

*h_{eq} based on AASHTO-LRFD Table 3.11.6.4-2

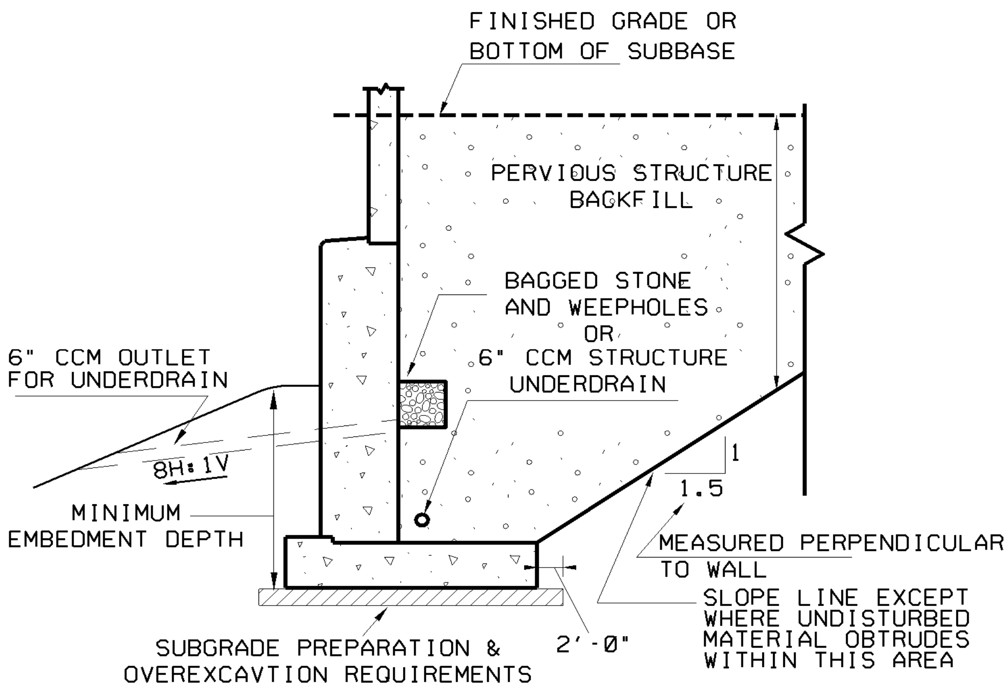
Foundation Design Details

Minimum Embedment Depth:	4 ft.
Backwall Drainage:	Bagged Stone and Weepholes
Subgrade Preparation:	Clean rock or concrete leveling pad
Maximum Temporary Cut Slope:	1(V):1.5(H)

Additional Comments

- Design recommendations based on 2007 *AASHTO LRFD* and *ConnDOT Bridge Design Manual*.
- Analysis assumes active earth pressures due to anticipated movements at the top of the wall of over 0.33 inches.
- Preliminary plans provided show the bottom of footing to be at elevation 5.5±.
- Include the following logs on the wall plan sheets: B-2, RB-5

Geotechnical Wall Design Parameters
Project No. 15-248
Stillman Pond Channel Wall (Gravity)



Factored Resistances

	Strength Limit	Service Limit
Bearing	7.4 tsf	7 tsf
Sliding	0.6V	0.6V

V=total vertical force

Lateral Earth Loads

Soil Unit Weight, γ :	125 pcf
Lateral Earth Pressure(static):	35 psf
Live Load Surcharge-Uniform Earth Pressure*:	$0.27\gamma\ h_{eq}$

*h_{eq} based on AASHTO-LRFD Table 3.11.6.4-2

Foundation Design Details

Minimum Embedment Depth:	4 ft.
Backwall Drainage:	Bagged Stone and Weepholes
Subgrade Preparation:	1 ft Granular Fill.
Maximum Temporary Cut Slope:	1(V):1.5(H)

Additional Comments

- Design recommendations based on 2007 *AASHTO LRFD* and *ConnDOT Bridge Design Manual*.
- Preliminary plans/cross sections provided show the bottom of footing to be at elevation 5±.
- Include the following logs on the wall plan sheets: B-4, RB-3

ITEM #702026A - MICROPILES

Description: This work shall consist of constructing micropiles as shown on the approved working drawings and as specified herein. The Contractor is responsible for furnishing all design, materials, products, accessories, tools, equipment, services, transportation, labor and supervision, and manufacturing techniques required for the design, installation and testing of micropiles and micropile top attachments for this project.

The Contractor shall select the micropile type, size, pile top attachment, installation means and methods, estimate the ground-grout bond value and determine the required grout bond length and final micropile diameter. The Contractor's designer shall design micropiles that will develop the load capacities required for the arch pedestal structure design, as well as the required resistance for the lateral demand of the arch pedestals. The micropile load capacities shall be confirmed by verification and proof load testing as required and must meet the test acceptance criteria specified herein.

Materials: Furnish materials new and without defects. Materials for micropiles shall consist of the following:

Admixtures for Grout: Admixtures shall conform to Article M.03.01 of the Form 817. Accelerators are not permitted. Expansive admixtures and admixtures containing chlorides are not permitted.

Cement: All cement shall be Portland cement conforming to ASTM C 150/AASHTO M85, Types II, III or V.

Centralizers and Spacers: Centralizers and spacers shall be fabricated from schedule 40 PVC pipe or tube, steel, or material non-detrimental to the reinforcing steel. Wood shall not be used. Centralizers and spacers shall be securely attached to the reinforcement; sized to position the reinforcement within ½ inch of plan location from center of pile; sized to allow grout tremie pipe insertion to the bottom of the drillhole; and sized to allow grout to freely flow up the drillhole and casing and between adjacent reinforcing bars.

Grout: Neat cement or fine aggregate/cement mixture. The designer is responsible for indicating the 3 day and 28 day compressive strengths. The grout shall conform to the specification AASHTO T106/ASTM C109 and to any minimum and/or maximum properties shown on the plans. The grout shall conform to Article M.03.01 of the Form 817.

Permanent Casing Pipe: Permanent steel casing/pipe shall conform to required minimum and/or maximum properties required by the Contractor's arch designer. The permanent steel casing/pipe shall be designed to withstand the design service loadings on the approved working drawings and the proof/verification tests loading described in this specification. The steel casing/pipe shall conform to one or more of the following specifications ASTM 252, ASTM 106, or API (N-80). The designer will be responsible for indicating the applicable material specification(s) and any welding or fabrication conditions that apply.

Reinforcing Bars: Reinforcing steel shall be deformed bars in accordance with ASTM A615/AASHTO M31. The grade, thickness and number of bars shall be indicated by the designer and shall conform to any minimum and/or maximum properties shown on the approved working drawings. Continuous spiral deformations (i.e. continuous threadbars) shall be used. Bar tendon couplers, if required, shall develop the ultimate tensile strength of the bars without evidence of any failure.

Construction Methods:

1 - Contractor's Experience Requirements:

The micropile Contractor shall be experienced in the construction and load testing of micropiles and have successfully constructed at least 5 projects in the last 5 years involving construction totaling at least 100 micropiles of similar capacity to those required in these plans and specifications.

The Contractor shall have previous micropile drilling and grouting experience in soil/rock similar to project conditions. The Contractor shall submit construction details, structural details and load test results for at least three previous successful micropile load tests from different projects of similar scope to this project.

The Contractor shall assign a professional engineer, licensed in the State of Connecticut, to supervise the work. This engineer shall have experience on at least 10 projects of similar scope to this project completed over the past 5 years. The Contractor shall not use manufacturers' representatives to satisfy the supervising engineer requirements of this section. The contractor may use a single independent consultant for this purpose, provided the consultant has specific experience as listed above, and operates their business specifically for the purpose of transferring technology and skills in micropiling to contractors. The on-site foremen and drill rig operators shall also have experience on at least 10 projects over the past 5 years installing micropiles of equal or greater capacity than required in these plans and specifications.

The Contractor shall assign a professional engineer, licensed in the State of Connecticut, to design the micropiles. This engineer shall have experience in the design of at least 3 successfully completed micropile projects over the past 5 years, with micropiles of similar capacity to those required in these plans and specifications. This engineer shall also be responsible for design, supervision and reporting of the verification and proof test(s).

At least 45 calendar days before the planned start of micropile construction, the Contractor shall submit 5 copies of the completed project reference list and a personnel list. The project reference list shall include a brief project description with the owner's name and current phone number and load test reports. The personnel list shall identify the supervising project engineer, drill rig operators, and on-site foremen to be assigned to the project. The personnel list shall contain a summary of each individual's experience and be complete enough for the Engineer to determine whether each individual satisfies the required qualifications.

Work shall not start, nor materials ordered, until the Engineer's written approval of the Contractor's experience qualification is given. The Engineer may suspend work if the Contractor uses non-approved personnel. If work is suspended, the Contractor shall be fully liable for all resulting cost and no adjustment in contract time will result from the suspension.

2-Micropile Design Requirements and Submittals

The micropiles shall be designed to meet the specific loading conditions anticipated from the Contractor's arch structure designer, and as shown on the approved working drawings. The micropile design shall conform to all required minimum and/or maximum properties shown on the approved working drawings. Design the micropiles and pile top to footing connections using the Service Load Design (SLD) procedures contained in the FHWA "Micropile Design and Construction Guidelines Manual", Report No. FHWA-SA-97-070 and the "Connecticut Department of Transportation Bridge Design Manual".

The required geotechnical safety factors/strengths factors (for SLD Design) shall be 2.0, unless specified otherwise. Estimated applied foundation loading will be as provided by the arch pedestal designer. Other information including but not limited to easements, rights-of-way and other applicable design criteria will be as shown on the plans or specified herein. Structural design of any individual micropile structure elements not covered in the FHWA manual shall be by the SLD method in conformance with appropriate articles of the most current Edition of the AASHTO Standard Specification for Highway Bridges, including current interim specifications.

Steel pipe used for micropile permanent casing shall incorporate an additional 1/16" thickness for sacrificial steel corrosion protection.

Where required as shown on the plans, corrosion protection of the internal steel reinforcing bars, consisting of either encapsulation, epoxy coating, or grout, shall be provided in accordance with the Material portion of this specification. Where the permanent casing is used for a portion of the micropile, encapsulation shall extend at least 5ft into the casing.

The Contractor shall prepare and submit to the Engineer working drawings. Working drawings shall be submitted in accordance with Article 1.05 of the Form 817. The working drawings shall include all information required for the design, plans, construction and quality control of the micropile foundations and arch pedestals. This information should include the following, but not necessarily be limited to;

1. Design Computations

    a. A written summary report which describes the overall micropile design.

    b. Applicable code requirements and design references.

    c. Micropile structure critical design cross-section(s) geometry including soil/rock strata and piezometric levels and location, magnitude and direction of applied loadings, including slope or external surcharge loads.

    d. Design criteria including, soil/rock shear strengths (friction angle and cohesion), unit weights, and grout-to-ground bond values and micropile drillhole diameter assumptions for each soil/rock strata.

e. Safety factors/strength factors used in the design of the ground-grout bond values, surcharges, soil/rock and material unit weights, steel, grout and concrete materials.

f. Design calculation sheets with the project number, micropile structure location, designation, date of preparation, initials of designer and checker, and page number at the top of each page. Provide an index page with the design calculations.

g. Design notes including an explanation of any symbols and computer program used in the design.

h. Pile to footing connection calculations.

2. Plans

   a. A plan view of the micropile structures identifying;

      i. A reference baseline and elevation datum.

      ii. The offset from the construction centerline or baseline to the face of the micropile structure at all changes in horizontal alignment.

      iii. Beginning and end of micropile structure stations.

      iv. Right-of-way and permanent or temporary construction easement limits, location of all known active and abandoned utilities, adjacent structures or other potential interference. The centerline of any drainage structure or drainage pipe behind, passing through or passing under the micropile structure.

      v. Subsurface exploration locations shown on the plan view of the proposed micropile structure alignment with appropriate reference baselines to fix the locations of the exploration relative to the micropile structure.

      vi. Dimensioned and detailed micropile and pile caps layout plan.

   b. An elevation view of the micropile structure(s) identifying;

      i. Elevation view showing micropile locations and elevations; vertical and horizontal spacing; batter and alignment and the location of drainage elements (if applicable).

      ii. Existing and finish grade profiles both behind and in front of the micropile structure.

   c. Design parameters and applicable codes.

   d. General notes for constructing the micropile structure including construction sequencing or other special construction requirements.

   e. Horizontal and vertical curve data affecting the micropile structure and micropile structure control points. Match lines or other details to relate micropile structure stationing to centerline stationing.

   f. A listing of the summary of quantities on the elevation drawing of each micropile structure showing pay item estimated quantities.

g. Micropile typical sections including micropile spacing and inclination; minimum drillhole diameter; pipe casing and reinforcing bar size and details; splice type and locations; centralizers and spacers; grout bond zone and casing plunge length (if used); corrosion protection details; and connection details to the substructure footing, anchorage, plates, etc.

h. A typical detail of verification and production proof test micropiles defining the micropile length, minimum drillhole diameter, inclination, and load test bonded and unbonded test lengths.

i. Details, dimensions and schedules for all micropiles, casing and reinforcing steel, including reinforcing bar bending details.

j. Details for constructing micropile structures around drainage facilities (if applicable).

3. Construction Procedures

   a. Detailed step-by-step description of the proposed micropile construction procedure, including personnel, testing and equipment to assure quality control. This step-by-step procedure shall be shown in sufficient detail to allow the Engineer to monitor the construction and quality of the micropiles.

   b. Proposed start date and time schedule and micropile installation schedule providing the following:

      i. Micropile number.

      ii. Micropile design load.

      iii. Type and size of rebar.

      iv. Minimum total bond length.

      v. Total micropile length.

      vi. Micropile top footing attachment.

   c. If welding of casing is proposed, submit the welding procedure. All welding shall be done in accordance with the current AWS Structural Welding Code.

   d. Information on space requirements for installation equipment that verify the proposed equipment can perform at the site.

   e. Plan describing how surface water, drill flush, and excess waste grout will be controlled and disposed.

   f. Certified mill test reports for the reinforcing steel and for permanent casing. The ultimate strength, yield strength, elongation, and material properties composition shall be included. For API N-80 pipe casing, coupon test results may be submitted in lieu of mill certification.

   g. Proposed Grouting Plan. The grouting plan shall include complete descriptions, and details for the following:

      i. Grout mix design and type of materials to be used in the grout including certified test data and trial batch reports. The Contractor shall also provide specific gravity of the wet mix design.

ii. Methods and equipment for accurately monitoring and recording the grout depth and grout volume as the grout is being placed.

iii. Estimated curing time for grout to achieve specified strength. Previous test results for the proposed grout mix completed within one year of the start of grouting may be submitted for initial verification and acceptance and start of production work. During production, grout shall be tested in accordance with Article M.03.01 of the Form 816.

iv. Procedure and equipment for Contractor monitoring of grout quality. At a minimum, the Contractor shall be required to use a Baroid Mud Balance (per API RP-13B-1) to check the specific gravity of the mixed grout prior to placement of the grout into each drilled micropile.

4. Detailed plans for the proposed micropile load testing method. This shall include all drawings, details, and structural design calculations necessary to clearly describe the proposed test method, reaction load system capacity and equipment setup, types and accuracy of apparatus to be used for applying and measuring the test loads and pile top movements in accordance with this specification.

5. Calibration reports and data for each test jack, pressure gauge and master pressure gauge and electronic load cell to be used. The calibration tests shall have been performed by an independent testing laboratory, and tests shall have been performed within 90 calendar days of the date submitted. Testing shall not commence until the Engineer has reviewed and accepted the jack, pressure gauge, master pressure gauge and electronic load cell calibration data.

Work shall not begin until the construction submittals have been received, reviewed, and accepted in writing by the Engineer. Any submittals that are found to be unacceptable by the engineer shall be revised, resubmitted and accepted prior to commencing work.

3 - Pre-Construction Meeting.

A pre-construction meeting will be scheduled by the Engineer and held prior to the start of micropile construction. The design Engineer, supervising Engineer, prime Contractor, arch pedestal designer, and micropile specialty Contractor, shall attend the meeting. Attendance is mandatory. The pre-construction meeting will be conducted to clarify the construction requirements for the work, to coordinate the construction schedule and activities, and to identify contractual relationships and delineation of responsibilities amongst the prime Contractor and the various Subcontractors – specifically those pertaining to excavation for micropile structures, installation of temporary sheeting, anticipated subsurface conditions, micropile installation and testing, micropile structure survey control and site drainage control.

4 - Site Drainage Control.

The Contractor shall control and properly dispose of drill flush and construction related waste, including excess grout, in accordance with Section 1.10 of the Form 816, any related specifications within the contract documents and all applicable codes and regulations. Drill flush shall be conveyed by pipe, hose or conduit a minimum 20ft away from the location where the micropile is being drilled and away from any adjacent structure or facility. Provide positive control and discharge of all surface water that will affect construction of the micropile installation. Maintain all pipes or conduits used to control surface water during construction. Repair damage caused by surface water at no additional cost. Upon substantial completion of the work, remove surface water control pipes or conduits from the site. Alternatively, with the approval of the Engineer, pipes or conduits that are left in place, may be fully grouted and abandoned or left in a way that protects the structure and all adjacent facilities from migration of fines through the pipe or conduit and potential ground loss.

Immediately contact the Engineer if unanticipated existing subsurface drainage structures or other utilities are discovered during excavation or drilling. Suspend work in these areas until remedial measures meeting the Engineer's approval are implemented.

5 - Excavation

Coordinate the work and the excavation so the micropile structures are safely constructed. Perform the micropile construction and related excavation in accordance with the Plans and approved submittals.

6 - Micropile Allowable Construction Tolerances

1. Centerline of piling shall not be more than 3 inches from indicated plan location.
2. Pile shall be plumb or battered within 2 percent of total-length plan alignment.
3. Top elevation of pile shall be plus 1-inch or minus 1-inch maximum from vertical elevation indicated.
4. Centerline of reinforcing steel shall not be more than 0.5-inches from indicated location.

7 - Micropile Installation

The micropile Contractor shall select the drilling method, the grouting procedure and the grouting pressure used for installation of the micropiles. The micropile Contractor shall also determine the micropile casing size, final drillhole diameter and bond length, and central tendon reinforcement steel sizing necessary to develop the specified load capacities and load testing requirements. The micropile Contractor is also responsible for estimating the grout take. There will be no extra payment for grout overruns.

Should the approved working drawings require uncased drilling of the micropile into bedrock, the permanent and/or temporary casing shall be drilled a minimum 6 inches into ledge or to a depth within the ledge so as to prevent subsidence of over burden into the uncased and/or bond zone portion of the drill hole (i.e. the rock socket). The approved working drawings will show estimated permanent casing lengths for each substructure unit. There will be no payment for differences in required length of temporary casing.

The drilling equipment and methods shall be suitable for drilling through the conditions to be encountered, without causing damage to the overburden above rock head, any overlying or adjacent structures, buried structures or utilities, or services. If called for in the drilling method description, or by the nature of the stratum to be drilled through, the micropile Contractor shall furnish an overburden casing of the type and thickness, which can be installed without distortion. Casings that fail, fracture, or otherwise distort during drilling or after drilling shall, unless otherwise directed, be withdrawn or replaced at the micropile Contractor's expense. The drillhole must be open along it's full length to at least the design minimum drillhole diameter prior to placing grout and reinforcement.

Temporary casing or other approved method of pile drillhole support will be required in caving or unstable ground to permit the pile shaft to be formed the minimum design drillhole diameter. The Contractor's proposed method(s) to provide drillhole support and to prevent detrimental ground movements shall be reviewed by the Engineer. Detrimental ground movement is defined as movement which requires remedial repair measures, in order to maintain site conditions as determined by the Engineer. Use of drilling fluid containing bentonite or any other non-reverting drilling fluid is not allowed.

During construction, the Contractor shall observe the ground conditions in the vicinity of the micropile construction site on a daily basis for signs of ground heave or subsidence. Immediately notify the Engineer if signs of movements are observed. The micropile Contractor shall immediately suspend or modify drilling or grouting operations if ground heave or subsidence is observed, if the micropile structure is adversely affected, or if adjacent structures are damaged from the drilling or grouting. If the Engineer determines that the movements require corrective action, the micropile Contractor shall take corrective actions necessary to stop the movement or perform repairs.

Reinforcement may be placed prior to grouting the drillhole. Reinforcement surface shall be free of deleterious substances such as soil, mud, grease or oil that might contaminate the grout or coat the reinforcement and impair bond. Pile reinforcement groups, if used, shall be sufficiently strong to withstand the installation and grouting process without damage or disturbance.

The micropile Contractor shall check pile top elevations and adjust all installed micropiles to the planned elevations.

Centralizers and spacers shall be provided at 10ft centers maximum spacing. The upper and lower most centralizer shall be located a maximum of 3ft from the top and bottom of the micropile. Centralizers and spacers shall permit the free flow of grout without misalignment of the reinforcing bar(s) and permanent casing. The reinforcing steel shall be inserted into the drill hole to the desired depth without difficulty. Partially inserted reinforcing bars shall not be driven or forced into the hole. The micropile Contractor shall re-drill and reinsert reinforcing steel when necessary to facilitate insertion.

Lengths of casing and reinforcing bars to be spliced shall be secured in proper alignment and in a manner to avoid eccentricity or angle between the axes of the two lengths to be spliced. Splices and threaded joints shall meet the requirements of the rebar material. Threaded pipe casing joints shall be located at least two casing diameters (OD) from a splice in any reinforcing bar. When multiple bars are used, bar splices shall be staggered at least 1 foot.

Micropiles shall be grouted the same day the load transfer bond length is drilled. The grouting equipment used shall be a colloidal grout plant, and shall produce a grout free of lumps and undispersed cement. Paddle type mixers are not acceptable. The micropile Contractor shall have means and methods of measuring the grout quantity and pumping pressures during the grouting operations. The grout pump shall be equipped with a pressure gauge to monitor grout pressure. A second pressure gauge shall be placed at the point of injection into the pile top. The pressure gauge shall be capable of measuring pressures of at least 145 psi or twice the actual grout pressure used, whichever is greater. The grout shall be kept in agitation prior to mixing. Grout shall be placed within one hour of mixing. The grouting equipment shall be sized to enable each pile to be grouted in one continuous operation. The grout shall be injected from the lowest point of the drill hole and injection shall continue until uncontaminated grout flows from the top of the pile. The grout may be pumped through pumped through grout tubes, casing, hollow stem augers or drill rods. Temporary casing, if used, shall be extracted in stages ensuring that, after each length of casing is removed the grout level is brought back up to the ground level before the next length is removed. The tremie pipe or casing shall always extend below the level of the existing grout in the drillhole. The grout takes shall be controlled to prevent excessive heave or fracturing of rock or soil formations. Upon completion of grouting, the grout tube may remain in the hole, but must be filled with grout.

If the Contractor elects to use a post-grouting system, Working Drawings and details shall be submitted to the Engineer for review in accordance with Article 1.05 of the Form 817.

Grout within the micropile verification and proof test piles shall attain the minimum required 3-day compressive strength prior to load testing. During production, micropile grout shall be tested by the Contractor for compressive strength in accordance with AASHTO T106/ASTM C109 at a frequency of no less than one set of three 2-inch grout cubes, or 3” cylinders, from each grout plant each day of operation or per every 10 piles, whichever occurs more frequently. The compressive strength shall be the average of the 3 cubes tested.

Grout consistency as measured by grout density shall be determined by the micropile Contractor per API RP-13B-1 at a frequency of at least one test per pile, conducted just prior to start of pile grouting. The Baroid Mud Balance used in accordance with API RP-13B-1 is an approved device for determining the grout density of neat cement grout.

Provide grout cube/cylinder compressive strength and grout density test results to the Engineer within 24 hours of testing.

8 - Micropile Installation Records.

The micropile Contractor shall prepare and submit to the Engineer full-length installation records for each micropile installed. The records shall be submitted within one work shift after that pile installation is completed. The data shall be recorded on a micropile installation log. A separate log shall be provided for each micropile.

9 – Verification and Proof Tests

Perform verification and proof testing of piles at the locations specified on the approved working drawings. Perform compression load testing in accord with ASTM D1143 and tension load testing in accord with ASTM D3689, except as modified herein.

Perform pre-production verification pile load test(s) to verify the design of the pile system and the construction methods proposed prior to installing any production piles. Sacrificial verification test pile(s) shall be constructed in conformance with the approved working drawings. Verification test pile(s) shall be installed at the location(s) shown on the approved working drawings or at a location(s) approved by the Engineer.

Verification load test(s) shall be performed to verify that the Contractor installed micropiles will meet the compression and/or tensile load capacities and load test acceptance criteria and to verify the length of the micropile load transfer bond zone is adequate. The micropile verification load test results must verify the Contractor's design and installation methods.

The drilling method, grouting method, casing length, micropile diameter (cased and uncased), reinforcing bar length and length of embedment for the verification test pile shall be identical to those specified for the production piles at the given locations. The verification test micropile structural steel sections shall be sized to safely resist the maximum test load.

The maximum verification and proof test loads applied to the micropile shall not exceed 80 percent of the structural capacity of the micropile structural elements, include steel yield in tension, steel yield or buckling in compression, or grout crushing in compression. Any required increase in strength of the verification and proof test pile elements above the strength required for the production piles shall be provided for in the Contractor's bid price.

Testing equipment shall include dial gauges, dial gauge independent reference frame, jack and pressure gauge, electronic load cell (with readout device), and a reaction frame. The load cell is required only for the creep test portion of the verification test. The contractor shall provide a description of test setup and jack, pressure gauge and load cell calibration curves in accordance with the Submittals Section.

Design the testing reaction frame to be sufficiently rigid and of adequate dimensions such that excessive deformation of the testing equipment does not occur. Align the jack, bearing plates, and stressing anchorage such that unloading and repositioning of the equipment will not be required during the test.

Apply and measure the test load with a hydraulic jack and pressure gauge. The pressure gauge shall be graduated in 100psi increments or less. The jack and pressure gauge shall have a pressure range not exceeding twice the anticipated maximum test pressure. Jack ram travel shall be sufficient to allow the test to be done without resetting the equipment. Monitor the creep test load hold during verification tests with both the pressure gauge and the electronic load cell. Use the load cell to accurately maintain a constant load hold during the creep test load hold increment of the verification test.

Measure the pile top movement with a dial gauge capable of measuring to 0.001 inches. The dial gauge shall have a travel sufficient to allow the test to be done without having to reset the gauge. Visually align the gauge to be parallel with the axis of the micropile and support the gauge independently from the jack, pile or reaction frame. Use a minimum of two dial gauges when the test setup requires reaction against the ground or single reaction piles on each side of the test pile.

Test verification piles to a maximum test load of 2.0 times the maximum allowable compressive load, hereafter termed, "Design Load" shown on the approved working drawings. The verification pile load tests shall be made by incrementally loading the micropile in accordance with the following cyclic load schedule for both compression and tension loading (test the compression prior to tension):

Verification Test Loading Schedule		
AL = Alignment Load DL = Design Load		
	LOAD	HOLD TIME
1	AL (.05 DL)	1 minute
2	0.25 DL	1 minute
3	0.50 DL	1 minute
4	AL	1 minute
5	0.25 DL	1 minute
6	0.50 DL	1 minute
7	0.75 DL	1 minute
8	AL	1 minute
9	0.25 DL	1 minute
10	0.50 DL	1 minute
11	0.75 DL	1 minute
12	1.00 DL	1 minute
13	AL	1 minute
14	0.25 DL	1 minute
15	0.50 DL	1 minute
16	0.75 DL	1 minute
17	1.00 DL	1 minute
18	1.33 DL	60 minutes
19	1.75 DL	1 minute
20	2.00 DL (Maximum Test Load)	10 minutes
23	AL	1 minute

The test load shall be applied in increments of 25 percent of the DL load. Each load increment shall be held for a minimum of 1 minute. Pile top movement shall be measured at each load increment. The load-hold period shall start as soon as each test load increment is applied. The verification test pile shall be monitored for creep at the 1.33 Design Load (DL). Pile movement during the creep test shall be measured and recorded at 1, 2, 3, 4, 5, 6, 10, 20, 30, 50, and 60 minutes. The alignment load shall not exceed 5 percent of the DL load. Dial gauges shall be reset to zero after the initial AL is applied.

The acceptance criteria for micropile verification load test are:

1. The Contractor's arch structure designer shall determine the criteria for tolerable movement during the load test at the top of the micropile.

2. At the end of the 1.33 DL creep test load increment, test piles shall have a creep rate not exceeding 0.05 inch/log cycle time (1 to 10 minutes) or 0.1 inch/log cycle time (6 to 60 minutes or the last log cycle if held longer). The creep rate shall be linear or decreasing throughout the creep load hold period.
3. Failure does not occur at any load increment up to and including the 2.0 D.L. max test load. Failure is defined as load at which attempts to further increase the test load simply result in continued pile movement.

Upon completion of the test, the Contractor shall prepare and submit a professional engineer stamped report of the test results for review and acceptance by the Engineer prior to beginning installation of production micropiles. This report shall include written confirmation of the verification micropile's capacity.

If a verification tested micropile fails to meet the acceptance criteria, the Contractor shall modify the design, the construction procedure, or both. These modifications may include modifying the installation methods, increasing the bond length, or changing the micropile type. Any modification that necessitates changes to the structure shall be submitted as a revision to the working drawings and require the Engineer's review and acceptance. Any modifications of design or construction procedures or cost of additional verification test piles and load testing shall be at the Contractor's expense. At the completion of verification testing, test piles shall be removed down to the elevation specified by the Engineer.

Perform proof load tests at the micropile locations as shown on the plans. Perform proof load tests on the first set of production piles installed at each designated substructure unit prior to the installation of the remaining production piles in that unit. The initial proof test piles shall be installed at the locations shown on the plans. Upon completion of each test, the Contractor shall prepare and submit a professional engineer stamped report of the test results for review and acceptance by the Engineer

Proof test piles to a maximum test load of 1.67 times the micropile Design Load shown on the Plans or Working Drawings. Proof tests shall be made by incrementally loading the micropile in accordance with the following schedule, to be used for both compression and tension loading:

Proof Test Loading Schedule		
AL = Alignment Load DL = Design Load		
	LOAD	HOLD TIME
1	AL	1 minute
2	0.25 DL	1 minute
3	0.50 DL	1 minute
4	0.75 DL	1 minute
5	1.00 DL	1 minute
6	1.33 DL	60 minutes Creep Test
7	1.67 DL (Maximum Test Load)	1 minute
8	AL	1 minute

Depending on performance, either a 10 minute or 60-minute creep test shall be performed at the 1.33 DL Test Load. Where the pile top movement between 1 and 10 minutes exceeds 1 mm, the Maximum Test Load shall be maintained an additional 50 minutes. Movements shall be recorded at 1, 2, 3, 5, 6, 10, 20, 30, 50 and 60 minutes. The alignment load shall not exceed 5 percent of DL. Dial gauges shall be reset to zero after the initial AL is applied.

The acceptance criteria for micropile proof load tests are:

1. The Contractor's arch structure designer shall determine the criteria for tolerable movement during the load test at the top of the micropile.

2. At the end of the 1.33 DL creep test load increment, test piles shall have a creep rate not exceeding 0.05 inch/log cycle time (1 to 10 minutes) or 0.1 inch/log cycle time (6 to 60 minutes). The creep rate shall be linear or decreasing throughout the creep load hold period.

3. Failure does not occur at the 1.67 DL maximum test load. Failure is defined as the load at which attempts to further increase the test load simply result in continued pile movement.

If a proof-tested micropile fails to meet the acceptance criteria, the Contractor shall immediately proof test another micropile within that footing. For failed piles and further construction of other piles, the Contractor shall modify the design, the construction procedure, or both. These modifications may include installing replacement micropiles, incorporating piles at not more than 50% of the maximum load attained, post-grouting the tested pile and re-proof testing the pile, modifying installation methods, increasing the bond length, or changing the micropile type. Any modification that necessitates changes to the structure design shall require the Engineer's prior review and acceptance. Any modifications of design or construction procedures, or cost of additional verification test piles and verification and/or proof load testing, or replacement production micropiles, shall be at the Contractor's expense.

Method of Measurement: This work will be paid for on a lump sum basis and will not be measured for payment.

Basis of Payment: This work will be paid for at the contract lump sum price for “Micropiles”, complete in place, which price shall include all work shown within the pay limits on the Contract Drawings for the Micropiles including but not limited to the following: furnishing and installation of micropiles and all components, performing and monitoring Verification Tests and Proof Tests as described herein, all tools, labor equipment and material incidental hereto. There will be no separate payment for mobilization and demobilization for this item.

ITEM # 0216009A Expanded Polystyrene Fill

Description:

Work under this item shall include furnishing and placing the EPS fill where indicated on the contract drawings and as specified herein.

Materials:

Blocks shall be smooth and flat on all surfaces and have a dimensional tolerance of ±0.5%. Blocks shall be manufactured using a modified resin that contains a fire retardant additive. Blocks shall be seasoned by storing them at the manufacturer's facility in normal ambient room temperature for a minimum of 72 hours after being released from the mold. Blocks shall meet the following physical requirements after seasoning:

Physical Properties

Min. Block Dry Density (lbs/ft^3)	1.0
Min. Test Specimen Dry Density	0.9
Min. Compressive Strength @1% deformation (psi)	5.8
Flexural Strength (psi)	30
Flammability (Oxygen Index, %)	24

The following reference standards shall apply in whole or in part to material supplied under this specification:

Applicable Standards

ASTM D6817 Standard Specification for Rigid Cellular Polystyrene Geofoam
ASTM C390 Criteria for Sampling and Acceptance of Preformed Thermal Insulation Lots

The EPS blocks shall be produced by a manufacturer with an in-place quality control program which is monitored and certified by an accredited, independent third-party testing organization.

Submittals: Submit detailed manufacturing records for the tested blocks which clearly state, in part, the percentage, type (in-plant or post-consumer), and original density of any recycled EPS material (regrind) used in the molding process.

Basis of Acceptance: Each EPS block shall be labeled with the manufacturer's name, product type, lot number, date of manufacture, weight and density (as measured after seasoning and trimming). Unlabeled blocks will be rejected. The Contractor shall supply detailed manufacturing records of individual blocks if requested by the Engineer.
The Engineer will perform on-site density tests by weighing and measuring one block randomly chosen from each truckload or from each 2500± cubic feet of EPS delivered to the project site. The Contractor shall provide a calibrated scale accurate to within 0.1 lbs and with sufficient capacity for this purpose. Blocks shall be kept clean and dry prior to weighing. If any block does not meet the minimum density requirement, the entire sampled truckload or 2500± cubic foot batch will be rejected by the Engineer.

EPS blocks that do not meet tolerances, or have side area surface damage of 20% or more or volume damage of 1% or more will be rejected.

Construction Methods:

General

A. Exercise care to prevent damage to the EPS during delivery, storage and construction. Protect the EPS blocks from (1) Organic solvents such as acetone, benzene, and paint thinner; (2) Petroleum based solvents such as gasoline and diesel fuel; (3) Open flames and (4) Prolonged exposure to sunlight (more than 30 days).
B. Provide a system of temporary weights or tie downs, approved by the Engineer, to anchor the EPS blocks if there is wind gust or flooding potential.
C. Do not drive or operate heavy machinery or place concentrated loads directly on the EPS blocks. EPS blocks damaged due to the Contractor's operations will be removed and replaced at no additional cost to the Project.
D. Trim the EPS blocks in the field where necessary with a portable hot wire device supplied by the manufacturer, or a handsaw, or an alternative cutting method approved by the Engineer.

Block Placement

A. Place the EPS blocks as indicated in the contract documents.
B. There shall be no debris of any kind between adjacent surfaces of EPS blocks or between the EPS blocks and the structure they will abut.
C. There shall be no standing water or accumulated snow or ice where the EPS blocks are to be placed.
D. EPS blocks shall be placed so that all vertical and horizontal joints between blocks are tight. Avoid continuous joints between blocks by laying blocks in a running bond pattern and orienting the long axis of the blocks in each successive layer perpendicular to the long axis of the blocks in the previous layer.
E. Blocks shall be placed such that the resulting exterior surfaces of the EPS Block Fill structures are vertical and planar within a tolerance of 1/8-inch between blocks.

Method of Measurement:

The quantity of Expanded Polystyrene Fill shall be the actual volume, in cubic yards, satisfactorily installed, as field measured in its final position.

Basis of Payment:

This work will be paid for at the contract unit price, per cubic yard, for "Expanded Polystyrene Fill" of the specified minimum density, which price shall include the preparation of subgrade, dewatering, the furnishing, placing and trimming of blocks, all materials, equipment, tools and labor incidental thereto. The unit price shall also include all required material testing.

SECTION 2.04
COFFERDAM AND DEWATERING
COFFERDAM MATERIAL LEFT IN PLACE

2.04.01—Description: Work under this item shall consist of the design, construction, maintenance and removal of a cofferdam, and necessary dewatering within the cofferdam, as shown on the plans.

If designated on the plans, the installed cofferdam material shall be left in place.

For the purposes of this specification, cofferdam shall be understood to mean any type of temporary earth retaining system, the type of which the Contractor elects to build, to fully enclose and confine an area to be pumped dry to enable construction to be carried out, and that satisfies the condition that the existing facilities be properly contained during excavation for the placement of substructures or other facilities.

2.04.03—Materials: Sheet pile material left in place shall meet the requirements of ASTM A328.

2.04.03—Construction Methods: The Contractor shall submit to the Engineer cofferdam working drawings in accordance with 1.05.02. The Contractor's proposed design must meet all requirements established in regulatory permits for the Project, the requirements of 1.10, and any stage construction configurations.

1. Cofferdams: Construction of the cofferdam shall be carried to the height shown on the plans and to an adequate depth. The cofferdam shall be constructed so that the work within can be safely carried to the bottom of the structure excavation.

The interior dimensions of the cofferdam shall be sufficient for the unobstructed and satisfactory completion of all necessary substructure work, including but not limited to pile driving, form building, inspection and pumping.

The Contractor shall be responsible for maintenance of the cofferdam. If the cofferdam becomes tilted or displaced prior to the completion of all work to be done within, the cofferdam shall be righted, reset, or enlarged as may be necessary to provide the clearance for the unobstructed performance of all necessary work.

The cofferdam shall be completely dewatered as required to complete the work entirely in the dry, except as specified below.

> When conditions are encountered that render it impractical to dewater the cofferdam, the Engineer may require the placing of underwater concrete of such dimensions as will be necessary to allow the Contractor to complete the work in the dry. The placement of underwater concrete shall comply with 6.01.03-6.

The cofferdam must be constructed to protect uncured masonry and concrete against damage from a sudden rising of the water and prevent damage to structure foundations by erosion. No part of the cofferdam which extends into the substructure may be left in place without written permission from the Engineer.

At least 30 calendar days prior to the start of constructing or installing a cofferdam, working drawings and design calculations for Cofferdam and Dewatering shall be submitted in accordance with the requirements of Article 1.05.02(2).

2. Dewatering: Pumping from the interior of any cofferdam shall be done in such a manner as to preclude the possibility of water moving through uncured masonry or concrete. During the placement of concrete or masonry, and for at least 24 hours thereafter, any pumping shall be done from a suitable sump located outside the horizontal limits and below the elevation of the work being placed or as directed by the Engineer.

The pumped water must be discharged in accordance with the requirements of 1.10. Pumping to dewater a cofferdam shall not start until any underwater concrete has sufficiently set to withstand the hydrostatic pressure created by pumping.

3. Removal of Cofferdam: Unless designated on the plans or directed by the Engineer, the Contractor shall remove all parts of the cofferdam and associated dewatering components after completion of the required work. This shall be done in such a way as not to disturb or otherwise damage any permanent construction.

4. Cofferdam Material Left in Place: Sheet piling used in constructing the cofferdam may be designated on the plans to be left in place The sheet piling shall be cut off at elevations shown on the plans or approved in advance by the Engineer, and the cut off portions shall be removed by the Contractor from the Site.

2.04.04—Method of Measurement:

1. Cofferdam and dewatering will be measured for payment by the actual quantity installed and accepted, in linear feet along the centerline of the top of the cofferdam.

If the cofferdam becomes tilted or displaced prior to the completion of all work to be done within, the corrections and adjustments of the cofferdam will not be measured for payment.

2. Cofferdam material left in place will be measured for payment by the actual quantity of linear feet of material left in place and accepted by the Engineer.

2.04.05—Basis of Payment:

1. Cofferdam and Dewatering: Payment for this work will be made at the Contract unit price per linear foot for "Cofferdam and Dewatering," measured as described above, which price shall include all costs of design, materials, equipment, labor, work, and any related environmental controls used in dewatering operations, which are required for the construction of the cofferdam shown in the plans; of any repair, correction, adjustment or reconstruction of such cofferdam as required by the plans; removal of obstructions; pumping and dewatering; removal of such cofferdam, and related environmental controls used in dewatering operations.

If the total number of linear feet of the cofferdam as accepted by the Engineer is greater than the quantity as designated on the original Contract plans, the Department will pay the Contractor for the revised number of such linear feet at the Contract unit price, subject to the provisions of 1.04.02 and 1.04.03.

If the Engineer allows the addition or enlargement of a cofferdam for the convenience or other benefit of the Contractor, but does not deem it essential for the performance of the Contract work, no additional payment will be made for the cofferdam or portion of the cofferdam which the Engineer does not deem essential.

2. Cofferdam Material Left in Place: In addition to Cofferdam and Dewatering, that portion of the cofferdam designated on the plans or ordered to be left in place will be paid for at the Contract unit price per linear foot for "Cofferdam Material Left in Place," which price shall include the cost of the sheet piling material left in place, the work to cut the sheet piling and removal of the cut off portions from the Site and all work incidental thereto.

Pay Item	Pay Unit
Cofferdam and Dewatering	l.f.
Cofferdam Material Left in Place	l.f.

SEISMIC REFRACTION
GEOPHYSICAL INVESTIGATION
DEPTH TO BEDROCK

BRIDGE #00325
ROUTE 1 over STILLMANS POND

BRIDGEPORT, CONNECTICUT

Prepared for

CONNECTICUT DEPARTMENT OF TRANSPORTATION

DECEMBER, 2015

NDT CORPORATION

NDT CORPORATION

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Tel: 978-563-1327
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December 17, 2015

Mr. Brett McKiernan
Connecticut Department of Transportation
Soils and Foundations Section
2800 Berlin Turnpike
Newington, Connecticut 06131-7546

Subject: Seismic Refraction Survey for Bridge No. 00325 Route 1 over Stillmans Pond, in Bridgeport, CT.

Dear Mr. McKiernan:

NDT conducted seismic refraction measurements to develop two bedrock profiles; one near the east abutment and one at the center-line beneath Bridge # 00325, Route 1 over Stillmans Pond in Bridgeport, Connecticut. Fieldwork was conducted on December 9th, 2015.

We thank you for the opportunity to perform this work and look forward to being of service to you in the future. If you have any questions or require additional information, call the undersigned at 978-563-1327.

Sincerely,

NDT Corporation

Paul S Fisk

Paul S Fisk

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3.0	RESULTS	page 1

FIGURES

APPENDIX 1 SEISMIC REFRACTION

1.0 OVERVIEW

A seismic refraction survey was conducted by NDT Corporation's Field technicians Keith Holster and Timothy Westerlind along the center line and east abutment beneath Bridge #00325 Route 1 over Stillmans Pond in Bridgeport, Connecticut. The fieldwork was completed on December 9th, 2015. The purpose of the survey was to develop two bedrock profiles beneath the bridge. The general location of the project area is shown in Figure 1 and the seismic lines of coverage are overlain on a site plan for the survey area in Figure2. Seismic refraction data was acquired with a 24 channel system with 5 and 10 foot geophone spacing's and seismic energy generated every 50 feet with a sledge hammer.

2.0 METHOD OF INVESTIGATION:

Seismic Refraction utilize the natural energy transmitting properties of the soils and rocks and is based on the principle that the velocity at which seismic waves travel through the earth is a function of the physical properties (elastic moduli and Poisson's ratio) of the materials. Direct and refracted compressional wave velocity data were used to evaluate material types and thickness and determining the approximate depth to layer interfaces for bedrock profiling. A more complete discussion of the seismic refraction survey method is included in Appendix 1.

Three seismic lines of data were collected and were 200 feet long with 24 geophone sensors and energy generated ("shots") at each end, quarter points, and center for each line. The three (south to north) seismic lines were referenced to the south edge of the bridge/sidewalk; station 0. Line 1 was a 200 foot long seismic line (stations -50 to 150) located along the center line of the arch (approximately 35 feet west of the east abutment) between the two railroad tracks. Line 2 (stations -75 to 125) and Line 3 (stations 0 to 200) were located approximately 14 feet west of the east abutment.

The ground surface elevation at the center-point of the bridge (approximately Line 1 station 35) was determined to be Elevation 14 by on-site CTDOT personnel. This was used as a reference elevation for the seismic ground surface and subsequent depths to rock.

3.0 RESULTS:

The results are presented as a depth to bedrock profiles in Figures 3 and 4. All measurements were made from the south edge of the bridge/sidewalk while elevations and depths are referenced to the ground surface elevation of 14 determined at the center point of the bridge (Line 1 station 35).The results in general are indicative of an overburden soil layer with a velocity of 1,000 ft. /sec, overlying a glacial till with a velocity of 2,000 +/- ft. /sec and a competent rock with a velocity of 12,000 to 14,000+/- ft. /sec. The results are shown as profiles in Figures 3 and 4. Depth to the bedrock ranges between elevation 1ft. and elevation 9 ft. under Bridge #00325. Seismic velocities in general consisted of a top fill layer with a velocity range of

1,000 ft. /sec. This is underlain by a 2,000 +/-ft./sec overburden/ablation till over bedrock with a velocity range of 12,000 to 14,000 +/- ft. /sec.

Overburden/fill with a 1,000 ft. /sec velocity is consistent with loose soils/sands/fill material typical of natural soils, fluvial deposits, and/or construction fill. Overburden with a 2,000±ft/sec velocity value is consistent with unstratified glacial drift or ground moraine. These tills consist of an admixture of clays, sands and gravels with occasional and sometimes frequent boulders associated with an ablation till.

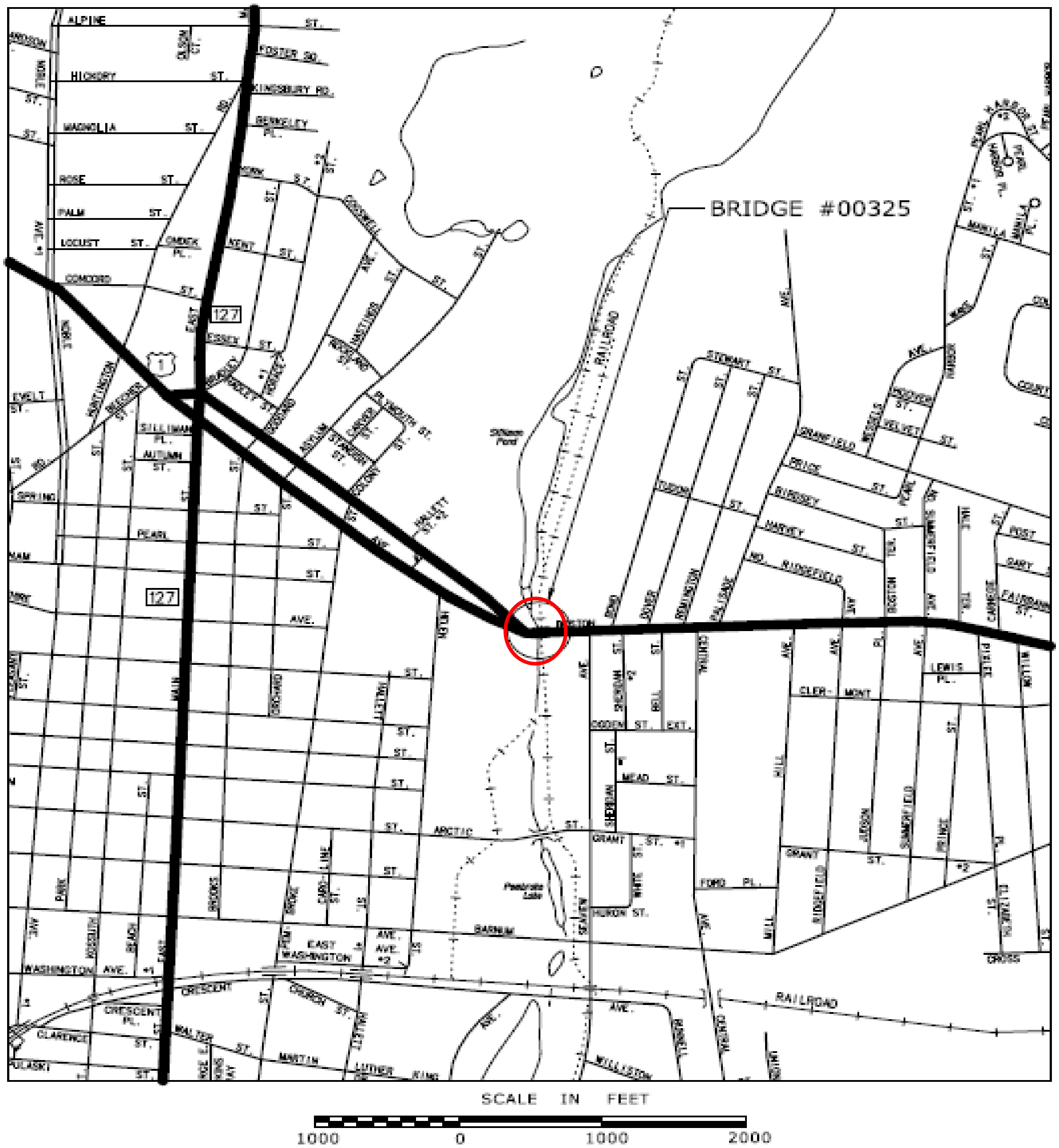
Bedrock velocities of 10,000 to 14,000 ft. /sec are indicative of normally fractured bedrock and typical of sedimentary and metamorphic rocks.

The seismic refraction data were interpreted using the critical distance method. Delayed bedrock wave arrivals were used to more accurately portray the bedrock surface between critical distance depth calculations. The delayed arrivals at individual geophone locations are an indicator of variability in the rock surface.

The rock surface in this survey area is irregular and the average top of rock is shown as an undulating dashed black line on the profile. Top of bedrock surface shown on the profile section is an average rock surface, localized high and low areas exist. Definition of high and low areas is a function of the seismic spread length, number of "shots" taken, geophone spacing, velocity contrast, and the irregularity of the rock surface. Variations of 3+/- feet are not accurately profiled.

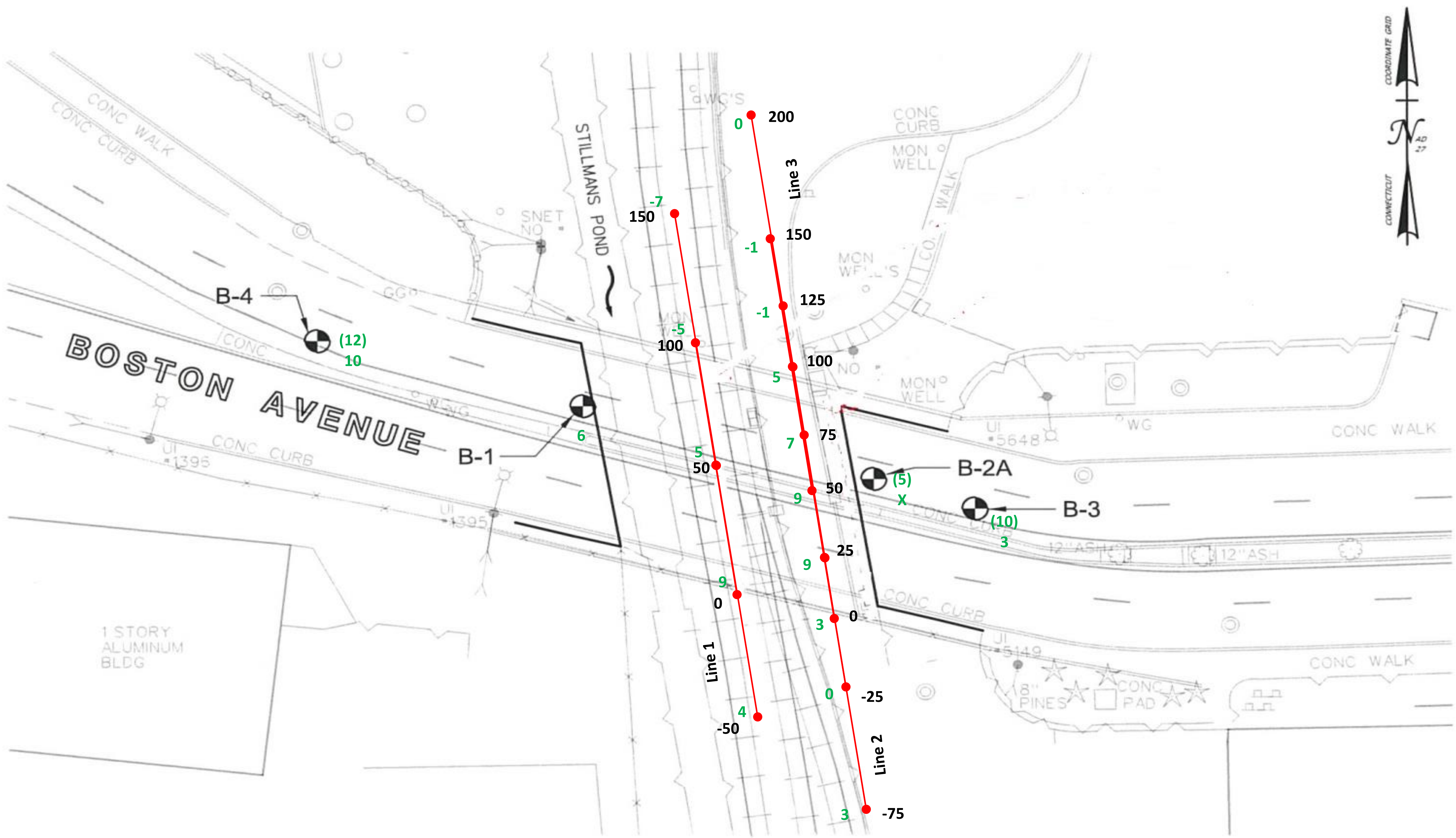
The top of rock ranges between elevation 9 and -7 along seismic Line 1 and between elevation 9 and -1 along seismic Lines 2 and 3. The seismic data indicates a "higher" ridge of rock under the bridge with rock dipping to greater depths north and south of the bridge extents.

FIGURES



STATE PROJECT NO.: 15-248 CITY/TOWN: BRIDGEPORT	STATE OF CONNECTICUT DEPARTMENT OF TRANSPORTATION BRIDGE #00325 OVER STILLMANS POND	OFFICE OF ENGINEERING	DATE: 01/07/14

Geophysical Survey Depth to Bedrock Bridge # 00325 Route 1 over Stillmans Pond Bridgeport, Connecticut for Connecticut DOT by NDT Corporation	Area of Investigation	
	Dec. 2015	Figure 1



(12) Elevation Top of Weathered Rock

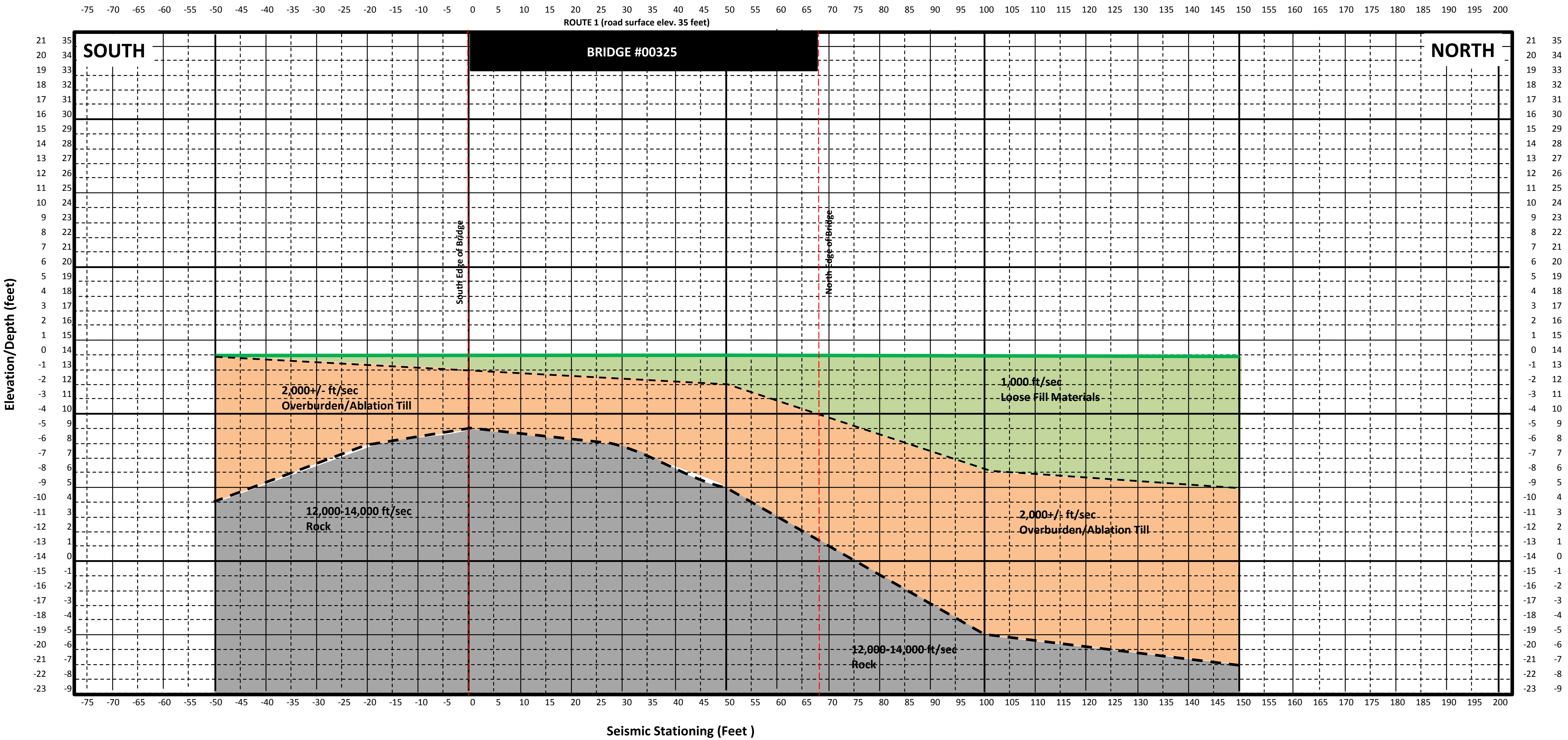
10 Elevation Top of Rock

Seismic Lines of Coverage

Seismic Shot Location

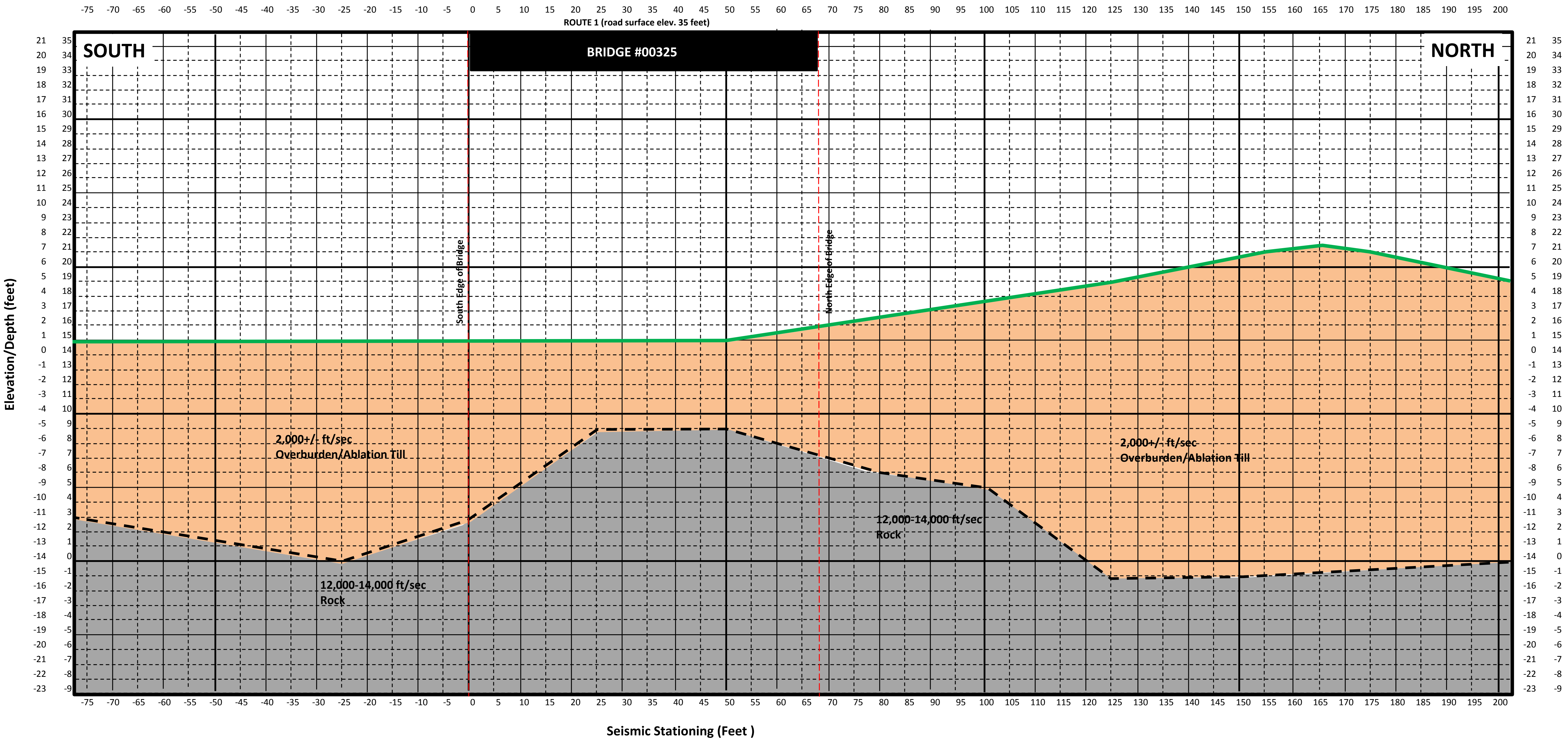
Geophysical Survey Depth to Bedrock Bridge # 00325 Route 1 over Stillmans Pond Bridgeport, Connecticut for Connecticut DOT by NDT Corporation	Lines of Coverage and Results	
	Dec. 2015	Figure 2

Seismic Line 1 - Center between Tracks



Geophysical Survey Depth to Bedrock Bridge # 00325 Route 1 over Stillmans Pond Bridgeport, Connecticut for Connecticut DOT by NDT Corporation	Seismic Results Line 1	
	Dec. 2015	Figure 3

Seismic Line 2 and 3 - Near East Abutment



Geophysical Survey Depth to Bedrock Bridge # 00325 Route 1 over Stillmans Pond Bridgeport, Connecticut for Connecticut DOT by NDT Corporation	Seismic Results Line 2 and 3	
	Dec. 2015	Figure 4

APPENDIX 1

SEISMIC REFRACTION

APPENDIX: SEISMIC REFRACTION

OVERVIEW

Seismic exploration methods utilize the natural energy transmitting properties of the soils and rocks and are based on the principle that the velocity at which seismic waves travel through the earth is a function of the physical properties (elastic moduli and Poisson's ratio) of the materials. Energy is generated at the ends and at the center of the seismic spread. The geophone/hydrophone is in direct contact with the earth/water and converts the earth's motion resulting from the energy generation into electric signals with a voltage proportional to the particle velocity of the ground motion. The field operator can amplify and filter the seismic signals to minimize background noise. Data are recorded on magnetic disk and can be printed in the field. Interpretations are based on the time required for a seismic wave to travel form a source to a series of geophones/hydrophones located at specific intervals along the ground surface. The resultant seismic velocities are used for:

* Material identification.
* Stratigraphic correlation.
* Depth determinations.
* Calculation of elastic moduli values and Poison's ratio.

A variety of seismic wave types, differing in resultant particle motion, are generated by a near surface seismic energy source. The two types of seismic waves for seismic exploration are the compressional (P) wave and the shear (S) wave. Particle motion resulting from a (P-wave) is an oscillation, consisting of alternating compression and dilatation, orientated parallel to the direction of propagation. An S-wave causes particle motion transverse to the direction of propagation. The P-wave travels with a higher velocity of the two waves and is of greater importance for seismic surveying. The following discussions are concerned principally with P-waves.

Possible seismic wave paths include a direct wave path, a reflected wave path or a refracted wave path. These wave paths are illustrated in FIGURE A1. The different paths result in different travel times, so that the recorded seismic waveform will theoretically show three distinct wave arrivals. The direct and refracted wave paths are important to seismic refraction exploration while the reflected wave path is important for seismic reflection studies.

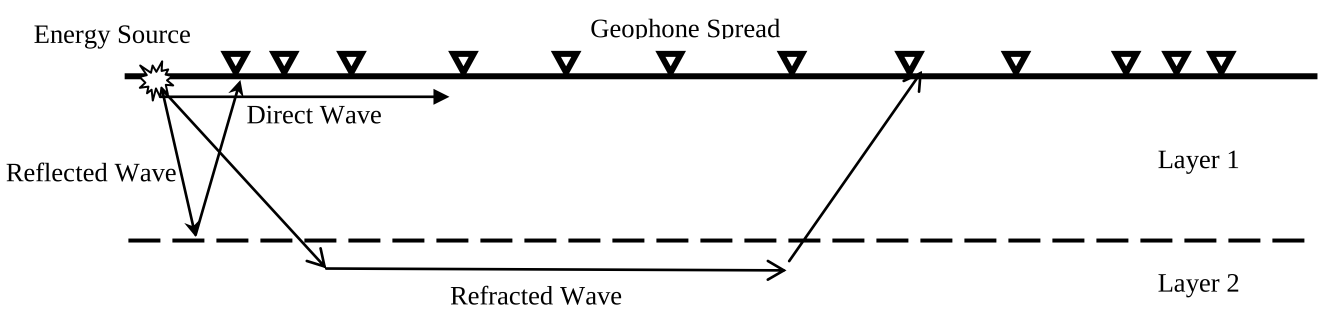


FIGURE A1:
SEISMIC WAVE PATHS FOR DIRECT WAVE, REFLECTED WAVE AND REFRACTED WAVE ILLUSTRATING EFFECTS OF A BOUNDARY BETWEEN MATERIALS WITH DIFFERENT ELASTIC PROPERTIES

Seismic waves incident on the interface between materials of different elastic properties at what is termed the critical angle are refracted and travel along the top of the lower layer. The critical angle is a function of the seismic velocities of the two materials. These same waves are then refracted back to the surface at the same angle. The recorded arrival times of these refracted waves, because they depend on the properties and geometry of the subsurface, can be analyzed to produce a vertical profile of the subsurface. Information such as the number, thickness and depths of stratigraphic layers, as well as clues to the composition of these units can be ascertained.

The first arrivals at the geophones/hydrophones located near the energy source are direct waves that travel through the near surface. At greater distances, the first arrival is a refracted wave. Lower layers typically are higher velocity materials, therefore the refracted wave will overtake both the direct wave and the reflected wave, because of the time gained travelling through the higher velocity material compensates for the longer wave path. Depth computations are based on the ratio of the layer velocities and the distance from the energy source to the point where refracted wave arrivals over take direct arrivals.

Although not the usual case, a constraint on refraction theory is that material velocities ideally should increase with depth. If a velocity inversion exists, i.e. where a higher velocity layer overlies a low velocity layer, depths and seismic velocities can be calculated but the uncertainty in calculations is increased unless borehole data are available.

APPLICATIONS

Seismic refraction technique is an accurate and effective method for determining the thickness of subsurface geologic layers. Applications for engineering design, assessment, and remediation as well as ground water and hydrogeologic studies include:

* Continuous profiling of subsurface layers including the bedrock surface
* Water-table depth determinations
* Mapping and general identification of significant stratigraphic layers
* Detection of sinkholes and cavities
* Detection of bedrock fracture zones
* Detection of filled-in areas
* Elastic moduli and Poisson's ratio values for subsurface layers

Seismic refraction investigations are particularly useful because seismic velocities can be used for material identification. FIGURE A2 presents a guide to material identification based on P-wave seismic velocities. In rocks and compacted overburden material, the seismic waves travel from grain to grain so that the measured seismic velocity value is a direct function of the solid material. In porous or fractured rock and most overburden materials the seismic waves travel partly or wholly though the fluid between the grains.

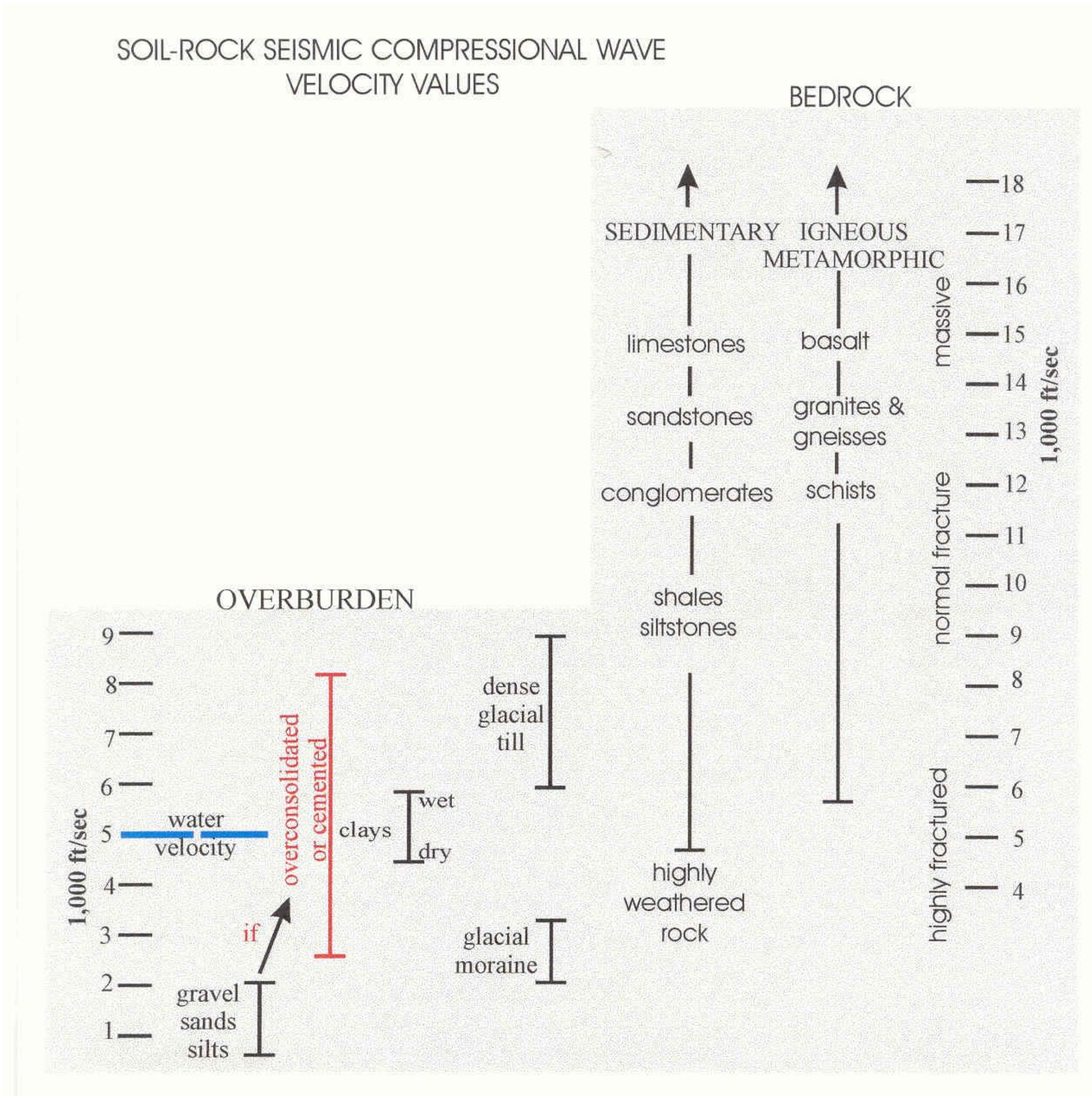


FIGURE A2:
GUIDE TO MATERIAL IDENTIFICATION BY P-WAVE VELOCITY

Seismic compressional wave velocities in unconsolidated deposits are significantly affected by water saturation. The seismic velocity values of unsaturated overburden materials such gravels, sands and silts generally fall in the range of 1,000 to 2,000 ft/sec. When these materials are water saturated, that is when the space between individual grains are 100% filled with water, the seismic velocities range from 4,800 to 5,100 ft/sec, equivalent to the compressional P-wave velocity of sound in water. This is because the seismic wave assumes the velocity of the faster medium, that of water. Even a small decrease in the saturation level will substantially lower the measured P-wave velocity of the material. Because of this velocity contrast between saturated and unsaturated materials, the water table acts as a strong refractor.

Seismic investigations over unconsolidated deposits are used to map stratigraphic discontinuities and to unravel the gross stratigraphy of the subsurface. These can be vertically as in the case of a dense till layer beneath a layer of saturated material or horizontally as in the case of the boundaries of a fill material. Often these boundaries represent significant hydrologic boundaries, such as those between aquifers and aquicludes.

A common use of seismic refraction is the determination of the thickness of a saturated layer in unconsolidated sediments and the depth to relatively impermeable bedrock or dense glacial till. Continuous subsurface profiles and even contour maps of the top of a particular horizon or layer of interest can be developed from a suite of seismic refraction data.

Bedrock velocities FIGURE A2 vary over a broad range depending on variables, which include:

* Rock type
* Density
* Degree of jointing/fracturing
* Degree of weathering

Fracturing and weathering generally reduce seismic velocity values in bedrock. Low velocity zones in seismic data must be evaluated carefully to determine if they are due to overburden conditions or fractured/weathered or perhaps even faulted bedrock.

<u>EQUIPMENT:</u>

The basic equipment necessary to conduct a seismic refraction investigation consists of:

* Energy source
* Seismometers (Geophones/Hydrophones)
* Seismic cables
* Seismograph

Energy sources used for seismic surveys are categorized as either non-explosive or explosive. The energy for a non-explosive seismic signal can be provided by one of the following:

* Sledge Hammer (very shallow penetration)
* Weight Drop
* Seisgun
* Airgun
* Sparker
* Vibrators (for reflection surveys)

Explosive sources can be categorized as:

* Dynamite
* Primers
* Blasting Agents

Choice of energy source is dependent on site conditions, depth of investigation, and seismic technique chosen as well as local restrictions. Explosive sources may be prohibited in urban areas where non-explosive sources can be routinely used. Deeper investigations usually require a larger energy source: therefore, explosives may be required for sufficient penetration.

Geophones/Hydrophones are sensitive vibration detectors, which convert ground motion to an electric voltage for recording the seismic wave arrivals. Seismic cables, which link the geophones/hydrophones and seismograph are generally fabricated with pre-measured locations for the geophones/hydrophones and shot point definitions.

The seismograph can be single channel or multi-channel, although, multi-channel seismographs (12 to 24 channels) are preferred and necessary for all but the simplest of very shallow surveys. The seismograph, amplifies (increases the voltage output of the geophones), conditions/filters the data, and produces analog and digital archives of the data. The analog archive is in the form of a thermal print of the data, which can be printed directly after acquisition in the field. The digital archive is stored on magnetic disk and can be used for subsequent computer processing and enable more extensive and detailed interpretation of seismic data.

ACQUISITION CONSIDERATIONS:

Several concerns arise before data collection, which must be addressed before of any seismic survey:

* Geophone spacing and Spread length
* Energy Source (discussed above)
* On-site utilities and cultural features (buildings, high tension lines, buried utilities, etc.)
* Vibration generating activities
* Geology
* Topography

To acquire seismic refraction data, a specific number of geophones are spaced at regular intervals along a straight line on the ground surface; this line is commonly referred to as a seismic spread. The length of spread determines the depth of penetration; a longer spread is required for a greater depth of penetration. Spread length should be approximately three to five times the required depth of penetration. Required resolution will control the number of geophones in each spread and the distance between each geophone. Closer spacings and more geophones usually result in more detail and greater resolution.

Cultural effects such as vibration generating activities, on-site utilities, and building affect where data can be acquired, and where lines/spreads are located. High volume traffic areas may require nighttime acquisition. If the survey is to be conducted near a building where vibration-sensitive manufacturing is conducted, data acquisition may be constrained to particular time intervals and appropriate energy sources must be used. Over head and buried utilities must be located an avoided, for both safety and induced electrical noise concerns. Since the seismic method measures ground vibration, it is inherently sensitive to noise from a variety of sources such as traffic, wind, rain etc. Signal Enhancement, such as record stacking, accomplished by adding a number of seismic signals from a repeated source, causes the seismic signal to "grow" out of the noise level, permitting operation in noisier environments and at greater source to phone spacings.

Knowledge of site geology can be used to determine the energy source. Some geologic materials, such as loose, unsaturated alluvium, do not transmit seismic energy as well and a powerful energy source may be required. Geologic conditions also dictate whether or not drilled shotholes are required. Site geology can also dictate the positioning of seismic lines/spreads. Where a bedrock depression of a feature is suspected, seismic lines should be orientated perpendicular to the suspected trend of the feature. Seismic cross profiles may be necessary to confirm depths to a particular refracting horizon.

The topography of a site dictates whether or not surveyed elevations are required. If possible, refraction profile lines should be positioned along level topography. For highly variable topography, a continuous elevation profile may be required to ensure sufficiently accurate cross-sections and to permit the use of time corrections in the interpretation of the refraction data.

<u>DATA PRESENTATION AND INTERPETATION:</u>

Interpretation of seismic refraction data involves solving a number of mathematical equations with the refraction data as it is presented on a travel-time versus distance chart. Seismic refraction data FIGURE A3 can be processed by plotting the "First Arrival" travel times at each geophone location. The preferred format of data presentation is a graph (Travel Time Plot) illustrated in FIGURE A4, in which travel time in milliseconds is plotted against source-receiver distance. From such a chart, the velocities of each layer can be obtained directly from the increase slope of each straight-line segment. Using the velocities the critical angle of refraction for each boundary can be calculated using Snell's Law. Then, utilizing these velocities, and angles and the recorded distances to crossover points (where line segments cross); the depths and thickness of each layer can be calculated using simple geometric relationships.

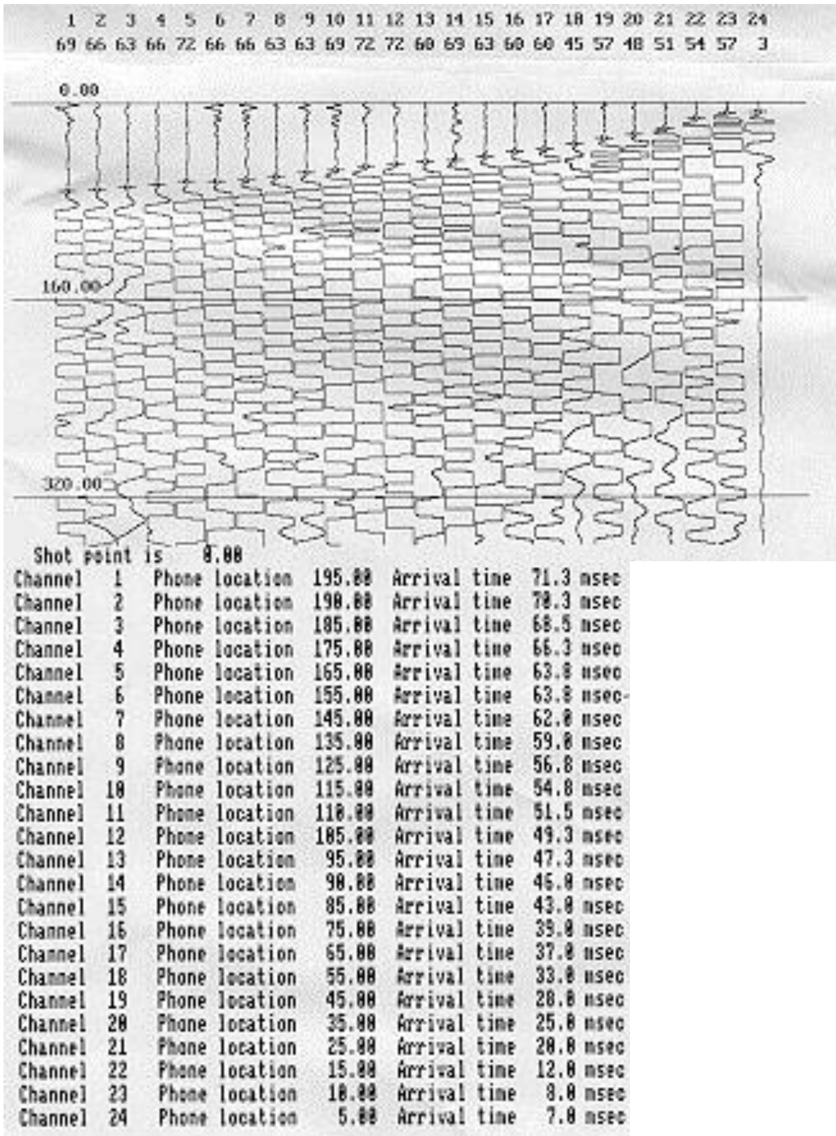


FIGURE A3:
TYPICAL 24 CHANNEL ANALOG SEISMIC REFRACTION RECORD, WITH FIRST ARRIVAL TIMES

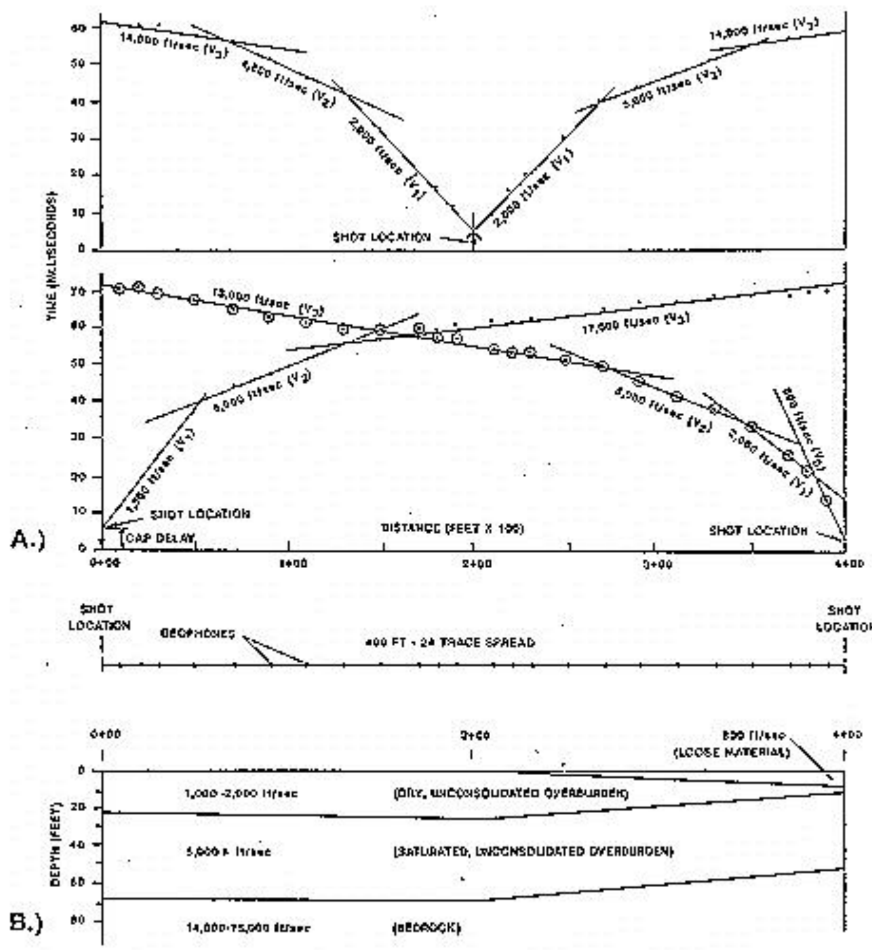


FIGURE A4:
A: TRAVEL-TIME PLOTS; UPPER PLOT IS A CENTER SHOT, LOWER PLOT IS TWO END SHOTS
B: RESULTING PROFILE OF SUBSURFACE MATERIALS SHOWING INTERFACE BETWEEN DIFFERENT SEISMIC VELOCITY LAYERS

The results of any seismic survey, refraction or reflection are usually presented in profile form showing elevations of seismic horizons/layers. Data acquired on a grid basis can be contoured and used to construct isopach maps. Seismic velocities and therefore, generalized material identifications should be presented on refraction profiles along with any test borings used for correlation to establish confidence in the overall subsurface data, both seismic and borings.

Where profiles indicate dipping boundaries, calculation of dips, true depths and true velocities involve more complicated equations. Further more, corrections for differing elevations and varying thicknesses of weathered zones must often be made. Fracturing and weathering generally reduce seismic velocity values in bedrock. Consequently, travel-time plots with late arrivals must be evaluated carefully to determine if the late arrival times (slower velocities) are due to overburden conditions or fractured/weathered bedrock.

Very thin layers or low velocity zones often complicate the travel-time chart as well. Although not the usual case, one constraint on refraction theory is that material velocities ideally should increase with depth. If a velocity inversion exists, i.e. where a higher velocity layer overlies a low velocity layer, depths and seismic velocities can be calculated but the uncertainty in calculations is increased unless borehole velocity data are available.

ADVANTAGES AND LIMITATIONS:

The seismic refraction technique, when properly employed, is the most accurate of the geophysical methods for determining subsurface layering and materials. It is extremely effective in that as much as 2,000 linear feet or more of profiling can be acquired in a field day. The resulting profiles can be used to minimize drilling and place drilling at locations where borehole information will be maximized resulting in cost-effective exploration. A standard drilling program runs the risk of missing key locations due to drillhole spacing. This risk is substantially reduced when refraction is used.

In summary, the advantages and limitations of the seismic techniques are:

Advantages:

* Material identification
* Subsurface data over broader areas at less cost than drilling
* Relatively accurate depth determination
* Correlation between drillholes
* Preliminary results available almost immediately
* Rapid data processing

Limitations:

* As depth of interest and geophone spacing increases, resolution decreases
* Thin layers may be undetected
* Velocity inversions may add uncertainty to calculations
* Susceptible to noise interference in urban areas, which require use of grounded cables and equipment, signal enhancement and alternative energy sources.