Geotechnical Engineering Report

Johnson BF 0248(4)

VT 100C Bridge Improvements
Johnson, Vermont

Prepared for

Vermont Agency of Transportation

1 National Life Drive
Montpelier, Vermont 5633-5001

Revised December 2015
CHA Project No. 28828.1000.32000

III Winners Circle, Albany, NY 12205-0269
www.chacompanies.com

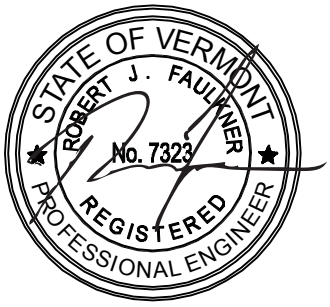
This report has been prepared and reviewed by the following qualified engineers employed by CHA.

Report Prepared By:

Nicholas Deflorio
Geotechnical Engineer

Report Reviewed By:

Charles W. Symmes, PE
Geotechnical Technical Leader



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1.0 INTRODUCTION

1.1 Purpose and Scope of Services

This report summarizes the results of services performed by Clough, Harbour & Associates, LLP (CHA) for the Vermont Agency of Transportation (VTrans) for the exploration of structural features and subsurface conditions and the design of a retaining wall for a bridge rehabilitation project (Johnson BF 0248(4)). The primary objectives of these services were to evaluate subsurface conditions at the proposed retaining wall location and provide geotechnical recommendations for wall design and to obtain existing geometry and subsurface data for the existing substructures.

1.2 Site and Project Description

This bridge rehabilitation project includes the replacement of two bridge superstructures and the construction of a retaining wall. The bridges are VT Route 100C Bridge 1 and VT Route 100C Bridge 2. The proposed retaining wall is to be located east of Bridge 2, adjacent to Bell Brook. Bridge 1 and Bridge 2 are located approximately 150 and 300 feet east, respectively, of the intersection of VT Route 100C and Sinclair Road in Johnson, Vermont. The project site is shown on Figure 1 which is included in Appendix A.

The Left Branch and Right Branch of Gihon River flows from south to north under Bridge 1 and Bridge 2. In addition, Bell Brook flows towards Bridge 2 from the northeast and joins the Right Branch of Gihon River at the northeast wingwall of Bridge 2. Bedrock outcrops were observed in the streambeds in the vicinity of the project area. As observed during the exploration programs, the water in the three channels near the bridges was approximately 6 to 12 inches deep.

The embankment slope from VT Route 100C down to the Bell Brook to the east of the Bridge 2 northeast wingwall is very steep and reportedly has had issues with erosion and slope instability.

Photographs of the site and this slope are included in Appendix B. A retaining wall is proposed to begin at the end of the northeast wingwall and extend approximately 75 feet to the east. This wall will be designed to retain and protect the roadway in the event of soil loss on the embankment due to future erosion and slope failure. A nongravity cantilever wall is proposed to minimize excavation and allow construction of the wall to be accomplished within the existing right of way.

1.3 Project Authorization

CHA performed services in accordance with proposal number X41651-P1 dated May 28, 2014 and X44435-P1 dated April 28, 2015. VTrans authorized the work as part of Contract PS0108 in a letter dated June 19, 2014 and June 3, 2015.

2.0 SUBSURFACE EXPLORATION

2.1 Boring Program

Borings B-101 and B-102 were advanced on August 11 and August 12, 2014. Boring B-103 was advanced on June 16, 2015 in the vicinity of the proposed retaining wall within the westbound travel lane of VT Route 100C. Borings B-201 and B-202 were advanced adjacent to the begin and end abutments of Bridge 1, respectively. Borings B-203 and B-204 were advanced adjacent to the begin and end abutments of Bridge 2, respectively. The bridge abutment borings were advanced between June 16 and June 18, 2015.

The boring locations were obtained by measuring from existing features. Boring locations are shown on Figure 2, Boring Location Plan, included in Appendix A. The elevations at the boring locations were estimated from topographic mapping of the site. The locations and elevations of the borings should be considered accurate only to the degree implied by the method used to determine them.

A CHA geotechnical engineer observed the borings to ensure that proper drilling techniques were used, and prepared boring logs based upon visual descriptions of the soil and rock samples and observation of the drilling action. A copy of the boring logs is included in Appendix C. Additional information regarding the sample descriptions is included on the Legend to Subsurface Logs included in Appendix C.

CHA retained New England Boring Contractors of Londonderry, New Hampshire to advance the 2014 borings. The borings were performed by a truck-mounted drill rig advancing 4-inch-inside-diameter flush joint casing. QCQA Laboratories, Inc. of Schuylerville, New York were retained to advance the 2015 borings. The borings were performed by an ATV rubber tire-mounted drill rig advancing 4-inch inside diameter flush joint casing.

The drillers performed Standard Penetration Tests (SPT) using a split spoon sampler with an

approximately 140-pound safety hammer at the depths noted on the borings logs in general accordance with AASHTO T 206: Standard Method of Test for Penetration Test and Split-Barrel Sampling of Soils. Additional information regarding the soil sampling procedures is included on the Legend to Subsurface Logs included in Appendix C.

An NX size core barrel was used to collect concrete and bedrock samples from the borings. The Rock Quality Designation (RQD) values were then determined in the field for the bedrock core samples. Additional information regarding the rock core sampling procedures and sample descriptions is included on the Legend to Subsurface Logs. Upon completion, the boreholes were backfilled with soil cuttings and patched with bituminous cold patch.

2.2 Structure Coring, Non-Destructive Testing and Geophysical Survey

QCQA Laboratories obtained one horizontal concrete core from the end abutments of both bridges using a 4-inch outside diameter portable core drill. Both cores were advanced approximately 1 to 3 feet below the existing bridge seat from within the stream. The core advanced at Bridge 1 was identified as C-1 and penetrated through the abutment at approximately 24 inches. The core advanced at Bridge 2 was identified as C-2 and penetrated through the abutment at approximately 29 inches.

CHA retained NDT Corporation of Sterling, Massachusetts to perform non-destructive testing and geophysical surveying of the existing bridge abutments, wingwalls, and proposed retaining wall alignment. The testing performed by NDT Corporation included ground penetrating radar, sonic/ultrasonic measurements and seismic refraction testing. The procedures and results of this effort are provided in the NDT Nondestructive and Geophysical Survey Report, found in Appendix E.

2.3 Laboratory Testing

Soil, rock, and concrete samples were submitted to the VTrans soil lab for the tests indicated in Table 1. The tests were performed in general accordance with the following standard test methods:

- AASHTO T-88: Particle Size Analysis of Soils
- ASTM D7012: Standard Test Method for Compressive Strength and Elastic Moduli of Intact Rock Core Specimens under Varying States of Stress and Temperatures
- ASTM C42: Compressive Strength of Concrete Core

A copy of the laboratory test report is included in Appendix D and the results are summarized on the boring logs in Appendix C where applicable.

Table 1: Laboratory Analysis Samples

Boring/Core	Depth (ft)	Strata	AASHTO T-88	ASTM D7012	ASTM C42
B-101	7-9	Silty Sand	X		
B-102	5-7	Silty Sand	X		
B-102	9-11	Silty Sand	X		
B-101	11.1-11.8	Phyllite		X	
B-102	14.1-14.9	Phyllite		X	
B-102	22-23.1	Phyllite		X	
B-103	0.5-2	Fill	X		
B-103	9-10.5	Weathered Rock	X		
B-103	19.1-19.9	Phyllite		X	
B-103	25.7-26.6	Phyllite		X	
B-201	4-6	Fill	X		
B-201	19.5-21	Weathered Rock	X		
B-201	24-24.9	Phyllite		X	
B-202	15-16	Fill	X		
B-202	19-20.5	Weathered Rock	X		
B-203	4-6	Fill	X		
B-203	19-21	Weathered Rock	X		
B-203	31.5-32.2	Phyllite		X	
B-204	4-6	Fill	X		
B-204	18.5-19.1	Phyllite		X	
C-1	Cylinders 1 & 2	Concrete			X
C-2	Cylinders 3 & 4	Concrete			X

3.0 SUBSURFACE CONDITIONS

3.1 Regional Geology

CHA reviewed the following publications to assess the regional geologic conditions:

- Doll, C.G., W. M. Cady, J. B. Thompson, and M. P. Billings (1970) "Surficial Geologic Map of Vermont." Vermont Geological Survey.
- Ratcliffe, N.M., R.S. Stanley, M.H. Gale, P.J. Thompson, and G.J. Walsh, (2011). "Bedrock Geologic Map of Vermont", U.S. Geological Survey Scientific Investigations Map 3184.

According to the *Surficial Geologic Map of Vermont*, the soils at the site consist of alluvium and lake sand. Bedrock outcrops are mapped to the south of the site.

According to the *Bedrock Geologic Map of Vermont*, the bedrock within the project area is likely phyllite or schist.

3.2 Subsurface Stratigraphy

Subsurface conditions encountered in the borings are detailed and described on the boring logs included in Appendix C of this report. Subsurface conditions are generally described below.

<u>Pavement</u> – A 5 to 8-inch thick layer of asphalt pavement was encountered at the ground surface in the borings.

<u>Subbase</u> – A 3 to 4-inch thick layer of subbase was encountered below the asphalt pavement in borings B-101, B-102, and B-202. The subbase consisted of fine to coarse gravel, was gray, and visually observed as moist.

Fill – Granular fill was encountered below the subbase or pavement in all borings to depths of approximately 3 to 5 feet in borings B-101 through B-103 and to depths of approximately 14.5 to 19 feet in borings B-201 through B-204. The fill consisted of varying amounts of fine to coarse sand, fine to coarse gravel, and silt. The fill was brown and was visually classified as moist. The SPT N-values ranged from 1 to 50 blows per foot (bpf) indicating very loose to dense conditions.

Silty Sand – Soil layers generally consisting of silty sand were encountered below the fill in borings B-101, B-102, and B-103 to depths of approximately 7.5 to 11.3 feet. The silty sand generally consisted of fine to coarse sand with little to some silt. The silty sand was brown and visually classified as moist to wet. A mild organic odor was observed in select samples recovered in the sand. The SPT N-values generally ranged from 4 to 31 bpf, indicating very loose to dense conditions. Higher blow counts were recorded at the interface of the silty sand with the fill and bedrock but were attributed to the boundary strata density.

Concrete – Concrete was encountered below the fill material in borings B-201 and B-202 to depths of 19.5 and 19 feet, respectively. The bottom of the concrete was interpreted based on core sample recovery and observed advancement of the core barrel.

Completely Weathered Bedrock – Completely weathered bedrock was encountered below the fill, silty sand or concrete in borings B-103, B-201, B-202, B-203 and B-204 to the top of bedrock at depths ranging from 11 to 28.5 feet. The completely weathered bedrock generally consisted of silt and fine to coarse sand with varying amounts of fine to coarse gravel and broken bedrock fragments. The weathered bedrock was gray and visually classified as moist. The SPT N-values ranged from 20 to greater than 100 bpf, indicating medium dense to very dense conditions.

Phyllite – Phyllite bedrock was encountered beneath silty sand in borings B-101 and B-102, and beneath completely weathered bedrock in borings B-103, B-201, B-202, B-203, and B-204 to termination depths. The phyllite was gray, hard, completely to slightly weathered and had RQD values ranging from 20 to 100 percent indicating very poor to excellent rock quality conditions.

3.3 Groundwater Observations

Groundwater was estimated at depths outlined in Table 2 during drilling operations.

Table 2: Groundwater Observations

Boring	Depth (ft)	Elevation (ft)
B-101	5.0	576
B-102	8.0	574
B-201	4.0	577
B-202	8.0	572
B-203	14.0	566
B-204	4.0	576

The boreholes were generally only open for a short time period during the drilling activities, and water was used as a drilling fluid. Also some of the soils at the site had high fines content and produce water slowly. Therefore, groundwater level observations during drilling operations may not represent static conditions. Seasonal factors such as temperature and precipitation also affect groundwater levels. For this reason, long-term groundwater levels may differ from those described in this report.

4.0 EVALUATION AND RECOMMENDATIONS

The following recommendations have been prepared based upon the guidelines presented in the 2010 VTrans Structures Design Manual which is based upon the AASHTO LRFD Bridge Design Specifications.

4.1 Nongravity Cantilever Wall

A nongravity cantilever wall is suitable for support of the embankment and roadway during the proposed design scenario. The wall system should be designed in accordance with AASHTO Section 11.8, Nongravity Cantilevered Walls. The bedrock present beneath the site will limit the penetration depth of driven sheet piles, therefore a drilled soldier pile and lagging wall is recommended.

The soldier piles, likely steel H-piles, should be installed in drilled boreholes. Sheet piling pairs should be considered as lagging to retain soil in the design scenario as they can be driven between the soldier piles without the need for excavation.

Based on the subsurface exploration, construction boreholes progressed through the existing fill and silty sand soils may be prone to collapsing especially in the presence of groundwater. For this reason boreholes may need to be temporarily cased during construction to the top of bedrock. After drilling into the bedrock, the H-piles should be placed and the annular space should be filled with concrete to the top of bedrock. After the concrete has set, bored holes should be backfilled with a low strength flowable fill or pea gravel to maintain the excavation after the casing has been removed. These types of backfill materials will allow for subsequent sheet pile installation to the top of bedrock.

There are various methods available to make the H-Pile to sheet pile connections including prefabricated connectors such as those manufactured by PilePro. This type of connector is welded onto the H-piles prior to installation.

Corrosion of the steel pile elements should be considered during design of the wall system. Oversizing the piles based on a corrosion rate of 0.003 inches per year over the service life of the structure is suitable for this application. Protective coatings may also be considered for the soldier piles in lieu of heavier sections, however should only be used for the sheet piles if the design carefully considers the potential for damage to the coating during installation.

4.1.1 Lateral Earth Pressures

The retaining wall should be designed to support the imposed lateral earth pressures in the design scenario which may include partial or total removal of soil on the Bell Brook side of the wall due to potential future erosion and slope failure. A maximum retained height of approximately 9 to 11 feet is anticipated in this event depending on final placement of the wall.

We recommend the lateral earth pressure parameters provided in Table 3 for analysis of the wall. A minimum soldier pile embedment of 5 feet into the bedrock is recommended should the analysis result in shorter bedrock embedment.

Table 3: Lateral Earth Pressure Parameters

Strata	Total Unit Weight[1], γ (pcf)	Friction Angle between Soil and Wall[2], δ (degrees)	Effective Angle of Internal Friction, ϕ' (degrees)	Shear Strength of Rock Mass, S_m (ksf)
Existing Fill Material	125	18	34	N/A
Silty Sand	115	17	28	N/A
Completely Weathered Bedrock	130	18	34	N/A
Phyllite Bedrock	N/A	N/A	N/A	3.1

Notes:
[1]Effective unit weight should be applied below the design groundwater level discussed in Section 4.3. Effective unit weight is defined as the total unit weight less the unit weight of water, 62.4 pcf.
[2]Assumes steel sheet piles are used for lagging.

- Lateral pressures resulting from surcharge loading should be evaluated per Section 3.11.6 of AASHTO.

- The wall should be analyzed for stability under hydrostatic equilibrium conditions as well as under flood drawdown conditions where a differential hydrostatic condition may occur unless adequate drainage details are provided.

4.2 Seismic Site Classification

The seismic site class for these structures are B based on the subsurface information available and AASHTO Table 3.10.3.1-1.

4.3 Groundwater and Control of Water

A groundwater level equivalent to the design high water level for the project should be used for analysis of the wall.

The boreholes for H-pile installation should be cased to prevent collapse as discussed in Section 4.1. Groundwater should be pumped from the boreholes prior to concrete placement or the concrete should be placed using appropriate tremie methods if preferred or if dewatering is ineffective.

5.0 CLOSURE

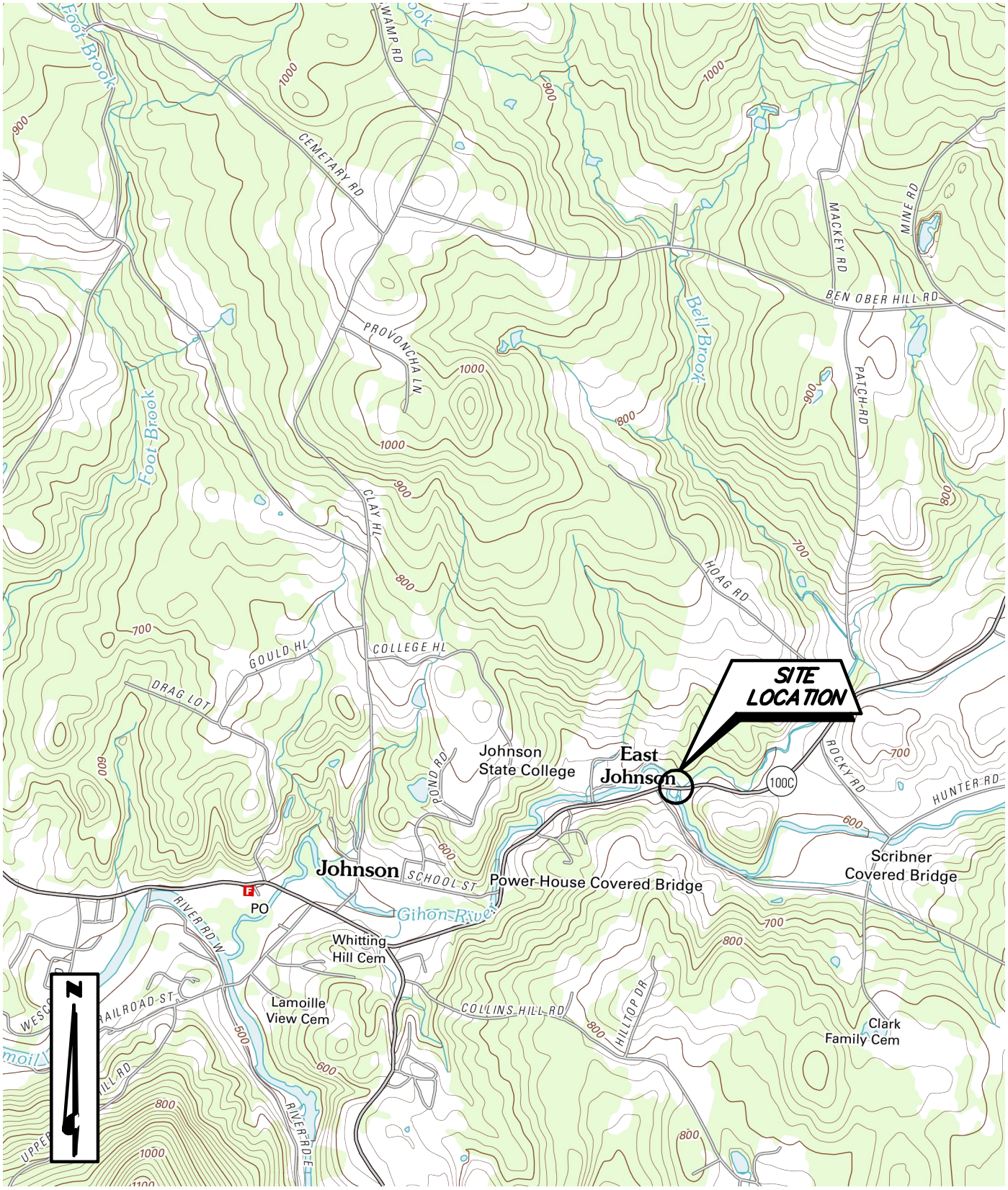
The geotechnical recommendations presented in this report are based, in part, on project and subsurface information available at the time this report was prepared and in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made. Some variation of subsurface conditions may occur between locations explored that may not become evident until construction. Depending on the nature and extent of the variations, it may be necessary to re-evaluate the recommendations presented in this report.

CHA does not accept responsibility for designs based upon our recommendations unless we are engaged to review the final plans and specifications to determine whether any changes in the project affect the validity of our recommendations and whether our recommendations have been properly implemented in the design.

This report has been prepared solely for design purposes and shall not be incorporated by reference or other means in the Contract Documents. If this report is included in the Contract Documents, it shall be for information only. Earthwork specification clauses shall take precedence.

APPENDIX A

FIGURES



SOURCE: U.S.G.S. 7.5' Topographic
QUADRANGLE: JOHNSON, VT

SCALE: 1"=2000'

CHA
III Winners Circle, PO Box 5269
Albany, NY 12205-0269
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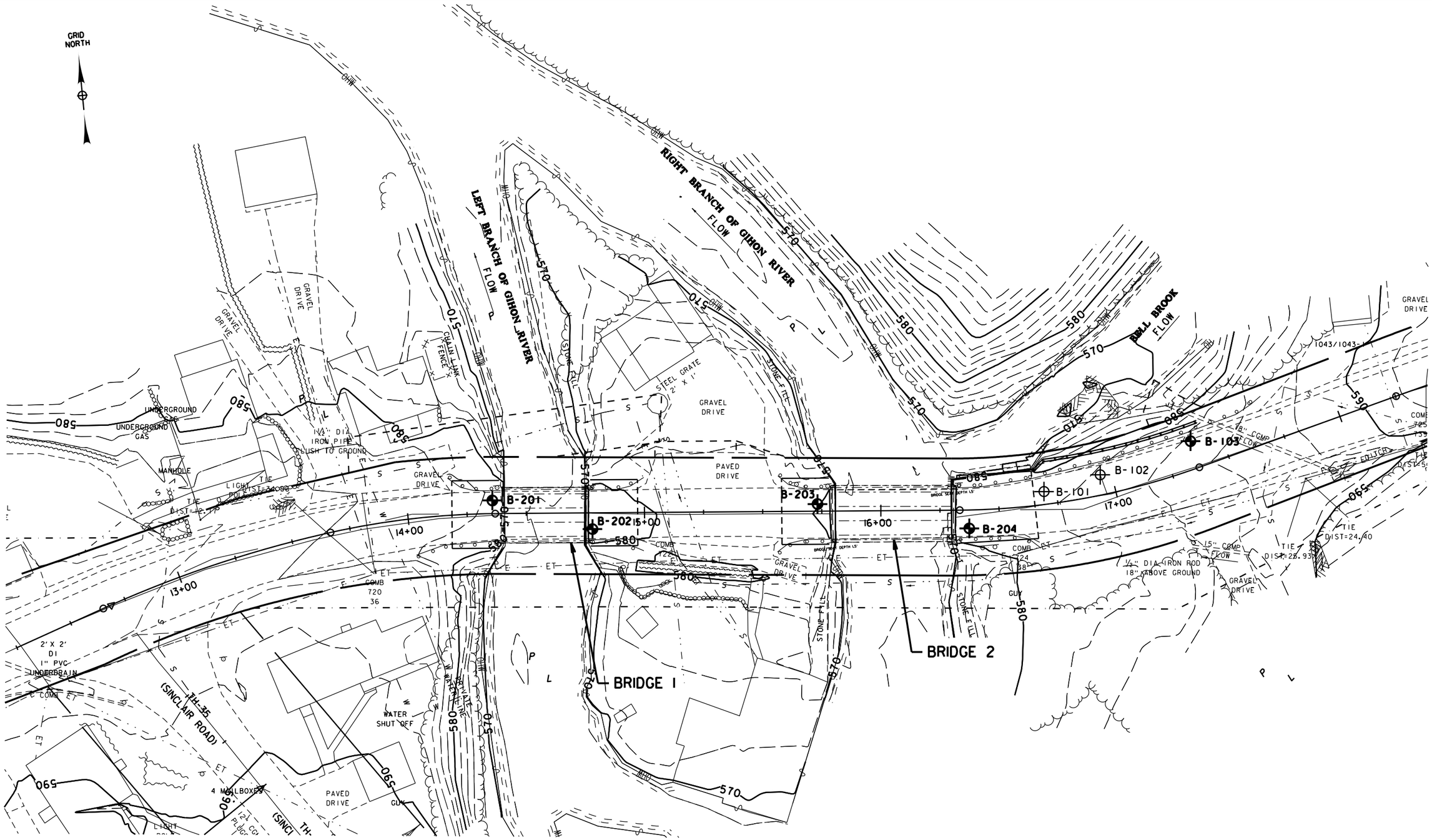
SITE LOCATION MAP

JOHNSON BF 0248(4)
VT 100C BRIDGE IMPROVEMENTS
JOHNSON, VERMONT

PROJECT NO. 28828

DATE: 07/2015

FIGURE 1



40 0 40 FT

SCALE

LEGEND

B-101 APPROXIMATE BORING LOCATION

B-201 PROPOSED BORING LOCATION

FILE NAME = $FILE$
DATE/TIME = $DATE$
USER = $NTUSER$

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BORING LOCATION PLAN

JOHNSON BF 0248(4)
VT 100C BRIDGE IMPROVEMENTS
JOHNSON, VT

FIGURE
2

DATE: 12/2015

APPENDIX B

SITE PHOTOGRAPHS

1

Proposed wall location, looking west
May 15, 2014

2

Proposed wall location, looking east
May 14, 2014

CHA	**VT 100C Bridge Improvements** **Johnson BF 0248(4)** **Johnson, VT**
CHA # 28828.1000.32000	November 2015

3

Drilling boring B-101, looking northeast
August 11, 2014

4

View of Bridge 2, looking west
August 11, 2014

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5

Advancing horizontal core at Bridge 2
June 16, 2015

6

Drilling Boring B-203
June 16, 2015

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APPENDIX C

BORING LOGS

LEGEND TO SUBSURFACE LOGS

SAMP./CORE NUMBER	SAMP. ADV(ft) LEN CORE (ft)	RECOVERY (ft)	Blows per 6" on Split Spoon Sampler	"N" VALUE or RQD%	SAMPLE	DEPTH (Feet)	GRAPHICS	DESCRIPTION AND CLASSIFICATION	ELEVATION (Feet)	Remarks on Character of Drilling, water return, etc	WATER LEVELS AND/OR WELL DATA
S1	2.0	1.8	2-3-4-5	7				f. SAND, Some Silt, trace f. gravel, brown, loose, moist (A-2-4)	100		
R1	2.0	2.0	N/A	88%				Mica SCHIST, gray, soft, slightly weathered, closely fractured, good RQD			
1	2	3	4	5	6	7	8	9 10	11	12	13

Subsurface Logs present material classifications, test data, and observations from subsurface investigations at the subject site as reported by the inspecting geologist or engineer. In some cases, the classifications may be made based on laboratory test data when available. It should be noted that the investigation procedures only recover a small portion of the subsurface materials at the site. Therefore, actual conditions between borings and sampled intervals may differ from those presented on the Subsurface Logs. The information presented on the logs provide a basis for an evaluation of the subsurface conditions and may indicate the need for additional exploration. Any evaluation of the conditions reported on the logs must be performed by Professional Engineers or Geologists.

1. SAMP./CORE NUMBER – Samples are numbered for identification on containers, laboratory reports or in text reports.

2. SAMP.ADV/LEN.CORE – Length of sampler advance or length of coring run measured in feet.

3. RECOVERY – Amount of sample actually recovered after withdrawing sampler or core barrel from bore hole measured in feet.

4. SAMPLE BLOWS/6" – Unless otherwise noted, blow counts represent values obtained by driving a 2.0" (O.D.), 1-3/8" (I.D.) split spoon sampler into the subsurface strata with a 140 pound weight falling 30" as per AASHTO T 206. After an initial penetration of 6" to seat the sampler into undisturbed material, the sampler is then driven an additional 2 or 3 six inch increments. Refusal is defined as a resistance greater than 50 blows per 6" of penetration.

5. "N" Value or RQD – "N" VALUE – The sum of the second and third sample blow increments is generally termed the Standard Penetration Test (SPT) "N" value. Refusal (R) is defined as a resistance greater than 50 blows for 6 inches of penetration. CORE RQD – Core Rock Quality Designation, RQD, is defined as the summed length of all pieces of core equal to or longer than 4 inches divided by the total length of the coring run. Fresh, irregular breaks distinguishable as being caused by drilling or recovery operations are ignored and the pieces are counted as intact lengths. RQD values are valid only for cores obtained with NX size core barrels.

6. SAMPLE – Graphical presentation of sample type and advance or core run length. See Table 1.

7. DEPTH – Depth as measured from the ground surface in feet.

8. GRAPHICS – Graphical presentation of subsurface materials. See Table 4. Dual soil classification and rock graphics may vary and are not shown on Table 4.

9. DESCRIPTION AND CLASSIFICATION – SOIL – Recovered samples are visually classified in the field by the supervising geologist or engineer unless otherwise noted. Particle size and plasticity classification is based on field observations, and using the AASHTO soil classification system. See Table 4. AASHTO symbols are presented in parentheses following the soil description. Where necessary, dual symbols may be used for combinations of soil types. Relative proportions, by weight and/or plasticity, are described in general accordance with "Suggested Methods of Test for Identification of Soils" by D.M. Burmister, ASTM Special Publication 479, 6-1970. See Table 2. Soil density or consistency description is based on the penetration resistance. See Table 3. Soil moisture description is based on the observed wetness of the soil recovered being dry, moist, wet, or saturated. Water introduced into the boring during drilling may affect the moisture content of the materials. Other geologic terms may also be used to further describe the subsurface materials. ROCK – Rock core descriptions are based on the inspector's observations and may be examined and described in greater detail by the project engineer or geologist. Terms used in the description of rock core are presented in Table 5.

10. DIVISION LINES – Division lines between deposits are based on field observations and changes in recovered material. Solid lines depict contacts between two deposits of different geologic depositional environment of known elevation. Dashed lines represent estimated elevation of contacts between two deposits of different geologic depositional environment. Dotted lines depict transitions of deposits within the same depositional environment, such as grain size or density.

11. ELEVATION – Elevation of strata changes in feet.

12. REMARKS – Miscellaneous observations.

13. WATER LEVELS & WELL DATA – Hollow water level symbol, if present, represents level at which first saturated sample or water level was encountered. Solid water level symbol, if present, depicts the most probable static water elevation at the time of drilling or as measured in an installed observation well at a later date. Subsurface water conditions are influenced by factors such as precipitation, stratigraphic composition, and drilling/coring methods. Conditions at other times may differ from those described on the logs. For graphical presentation of observation/monitoring well construction, see Table 6. Elevations of changes in construction are noted at the bottom of each section.

LEGEND TO SUBSURFACE LOGS

TABLE 1
TYPICAL SAMPLE TYPES

SPLIT SPOON (1 3/8" I.D.)

NX SIZE ROCK CORE

SHELBY TUBE "UNDISTURBED"

AUGER SAMPLE

TABLE 2
SAMPLE MATERIAL PROPORTIONS

ADJECTIVE	PERCENTAGE OF SAMPLE
"and"	35% – 50%
"some"	20% – 35%
"little"	10% – 20%
"trace"	< 10%

Standard split spoon samples may not recover particles with any dimension larger than 1 3/8". Therefore, reported gravel percentages may not reflect actual conditions.

TABLE 3
DENSITY/CONSISTENCY

GRANULAR SOILS		COHESIVE SOILS	
Blows/ft.	Density	Blows/ft.	Consistency
< 5	Very Loose	< 3	Very Soft
5–10	Loose	3–4	Soft
11–24	Med. Dense	5–8	Med. Stiff
25–50	Dense	9–15	Stiff
> 50	Very Dense	16–30	Very Stiff
		31–60	Hard

TABLE 4
AASHTO CLASSIFICATION, PARTICLE SIZE, & GRAPHICS

	MAJOR PARTICLE SIZE DIVISION	AASHTO SYMBOL	GRAPHIC SYMBOL	GENERAL DESCRIPTION
COARSE GRAINED SOILS	GRAVEL Coarse: 3"–3/4" Fine: 3/4"–#10 Classification based on > 50% being gravel	A–1–a		Well graded gravels, gravel & sand mix.
		A–1–b		Poorly graded gravels, gravel & sand mix.
		A–2–4		Sand and silt mix.
		A–2–5		Sand and silt mix.
	SAND Coarse: #10–#20 Med.: #20–#40 Fine: #40–#200 Classification based on > 50% being sand	A–2–6		Sand and clay mix.
		A–2–7		Sand and clay mix.
		A–3		Sand and gravel mix.
FINE GRAINED SOILS	SILT & CLAY Classification based on > 50% passing #200 sieve.	A–4		Inorganic silt, low plasticity.
		A–5		Inorganic silt, high plasticity.
		A–6		Inorganic clay, low plasticity.
		A–7–5		Inorganic clay, moderate plasticity.
		A–7–6		Inorganic clay, high plasticity.
ORGANIC SOILS		A–8		Peat and other highly organic soils.
FILL		Fill		Miscellaneous fill materials.

TABLE 5
ROCK CLASSIFICATION TERMS

HARDNESS:

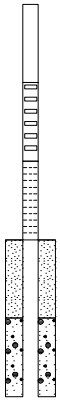
Very Soft	Carves
Soft	Groves with knife
Med. Hard	Scratched easily with knife
Hard	Scatched with difficulty
Very Hard	Cannot be scratched with knife

WEATHERING:

Fresh	Slight or no staining of fractures, little or no discoloration, few fractures.
Slightly	Fractures stained, discoloration may extend into rock 1", some soil in fractures.
Moderately	Significant portions of rock stained and discolored, soil in fractures, loss of strength.
Highly	Entire rock discolored and dull except quartz grains, severe loss of strength.
Complete	Weathered to a residual soil.

BEDDING:		FRACTURE SPACING:		RQD:	
Massive	> 40"	Massive/V. Wide	> 6'	Excellent	> 90%
Thick	12' – 40"	Thick/Wide	2' – 6'	Good	76% – 90%
Medium	4" – 12"	Med./Med.	8" – 24"	Fair	51% – 75%
Thin	< 4"	Thin/Close	2 1/2" – 8"	Poor	25% – 50%
		V. Thin/V. Close	< 2 1/2"	V. Poor	< 25%

TABLE 6
WELL CONSTRUCTION



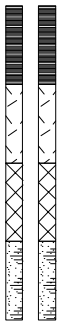
SOLID PVC PIPE

SCREENED PVC PIPE

STAINLESS STEEL SCREENED PIPE

FINE GRAINED WASHED SAND

WASHED SAND



BENTONITE PLUG

AIR ENTRAINED CEMENT

NATURAL SOIL/ ROCK FILL

BENTONITE/ CEMENT GROUT

STATE OF VERMONT
AGENCY OF TRANSPORTATION
MATERIALS & RESEARCH SECTION
SUBSURFACE INFORMATION

BORING LOG

Johnson BF 0248(4)
VT 100C Bridge Improvements
Johnson, VT

Boring No.: **B-101**
Page No.: 1 of 1
Pin No.: 13c066
Checked By: CWS

Boring Crew: G. Leavitt, K. Owens
Date Started: 8/11/14 Date Finished: 8/11/14
VTSPG NAD83: N 780143.69 ft E 1598806.35 ft
Station: 16+69 Offset: 6L
Ground Elevation: 581.0 ft

	Casing	Sampler
Type:	FJC	SS
I.D.:	4 in	1.38 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	N.A.	30 in.
Hammer/Rod Type:	Manual/NW	
Rig:	SS 15 TRUCK	C_E = 0.85

Groundwater Observations

Date	Depth (ft)	Notes
08/11/14	5.0	During Drilling

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Run (Dip deg.)	Core Rec. % (RQD %)	Drill Rate minutes/ft	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
		Asphalt Pavement, 0.0 ft - 0.7 ft								
		Subbase, 0.7 ft - 1.0 ft								
		(FILL), **f.c. SAND**, little f.c. gravel, trace silt, dense, brown, Moist, Rec. = 1.1 ft				26-24-19-17 (43)				
		(FILL), **f.c. GRAVEL**, Some f.c. Sand, little silt, dense, brown, Moist, Rec. = 0.6 ft				18-25-25-20 (50)				
5		A-2-4, **f.c. SAND**, Some Silt, medium dense, gray, Wet, Rec. = 0.5 ft, Mild organic odor				10-6-7-5 (13)				
		A-2-4, **f.c. SAND**, Some Silt, very loose, brown, Wet, Rec. = 1.6 ft, Mild organic odor				2-1-1-1 (2)	24.3	0.2	74.3	25.4
		A-2-4, becomes very dense, Rec. = 0.4 ft				7-40/2" (R)				
10		9.5 ft - 10.0 ft				Top of Bedrock @ 9.5 ft				
		10.0 ft - 15.0 ft, Gray, Phyllite, quartz seams, close fracture spacing. Hard, Slightly weathered, Good rock, NXDC, 3" highly weathered seam 10.5'-10.8'	R-1	98 (87)	0.16					
15		15.0 ft - 20.0 ft, Gray, Phyllite, quartz seams, close fracture spacing. Hard, Slightly weathered, Good rock, NXDC, 1" highly weathered seam 15.9'-16.4'	R-2	96 (82)	0.18					
20		20.0 ft - 25.0 ft, Gray, Phyllite, quartz seams, medium close fracture spacing. Hard, Slightly weathered, Excellent Rock, NXDC	R-3	98 (95)	0.14					
25		Hole stopped @ 25.0 ft								
30										
35										
40										

Remarks:
0': Rollerbit through asphalt pavement and subbase. Layers identified through visual observation of borehole sidewalls.

9.5': Rollerbit grinding 9.5' - 10', interpreted as top of bedrock.

11.5': Uniaxial Compressive Strength, qu=5,060 psi

AASHTO classifications are based on visual description of sample recovery at depths where lab testing not performed.

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. C_E is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the time measurements were made.

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AGENCY OF TRANSPORTATION
MATERIALS & RESEARCH SECTION
SUBSURFACE INFORMATION

BORING LOG

Johnson BF 0248(4)
VT 100C Bridge Improvements
Johnson, VT

Boring No.: **B-102**
Page No.: 1 of 1
Pin No.: 13c066
Checked By: CWS

Boring Crew: G. Leavitt, K. Owens
Date Started: 8/11/14 Date Finished: 8/12/14
VTSPG NAD83: N 780148.21 ft E 1598830.66 ft
Station: 16+94 Offset: 8L
Ground Elevation: 582.0 ft

	Casing	Sampler
Type:	FJC	SS
I.D.:	4 in	1.38 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	N.A.	30 in.
Hammer/Rod Type:	Manual/NW	
Rig:	SS 15 TRUCK	C_E = 0.85

Groundwater Observations

Date	Depth (ft)	Notes
08/11/14	8.0	During Drilling

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Run (Dip deg.)	Core Rec. % (RQD %)	Drill Rate minutes/ft	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
		Asphalt Pavement, 0.0 ft - 0.7 ft								
		Subbase, 0.7 ft - 1.0 ft								
		(FILL), **f.c. GRAVEL**, Some f.c. Sand, trace silt, dense, brown, Moist, Rec. = 0.9 ft				18-27-14-16 (41)				
		(FILL), **Similar Soil**, Rec. = 0.9 ft				12-16-15-4 (31)				
5		A-2-4, **f. SAND**, little silt, dense, brown, Moist								
		A-4, **SILT**, Some f.c. Sand, little f.m.c. gravel, medium dense, brown, Moist, Rec. = 0.8 ft				10-8-5-7 (13)	10.5	13.5	28.0	50.8
		A-2-4, **f. SAND**, Some Silt, medium dense, brown, MTW, Rec. = 1.3 ft, Mild organic odor				10-8-5-3 (13)				
10		A-2-4, **f.c. SAND**, little silt, trace f. gravel, very loose, brown, Wet, Rec. = 1.0 ft, Mild organic odor				2-3-1-1 (4)	27.0	6.1	77.8	13.7
		A-2-4, **f.c. SAND**, Some Silt, little f.c. gravel, very dense, brown, Wet, Rec. = 0.3 ft				50/4" (R)				
		Top of Bedrock @ 11.3 ft								
		11.3 ft - 12.0 ft	R-1	82 (73)	0.15					
15		12.0 ft - 17.0 ft, Gray, Phyllite, close fracture spacing. Hard, Slightly weathered, Fair rock, NXDC								
		17.0 ft - 21.5 ft, Gray, Phyllite, close fracture spacing. Hard, Slightly weathered, Good rock, NXDC	R-2	93 (89)	0.13					
20										
		21.5 ft - 22.0 ft								
		22.0 ft - 27.0 ft, Gray, Phyllite, medium close fracture spacing. Hard, Slightly weathered, Excellent Rock, NXDC	R-3	100 (100)	0.2					
25										
		Hole stopped @ 27.0 ft								
30										
35										
40										

Remarks:
0': Rollerbit through asphalt pavement and subbase. Layers identified through visual observation of borehole sidewalls.

14.5': Uniaxial Compressive Strength, qu=4,860 psi

21.5': Core barrel jammed at 21.5', rollerbit to 22' to ream out borehole prior to advancement of rock core R-3.

22.4': Uniaxial Compressive Strength, qu=5,950 psi

27': R-3 had 72% recovery initially, core barrel returned to hole and recovered the remaining portion of the core sample.

AASHTO classifications are based on visual description of sample recovery at depths where lab testing not performed.

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. C_E is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the time measurements were made.

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BORING LOG

Johnson BF 0248(4)
VT 100C Bridge Improvements
Johnson, VT

Boring No.: **B-103**
Page No.: 1 of 1
Pin No.: 13c066
Checked By: CWS

Boring Crew: J. Leonhardt, K. Owens
Date Started: 6/16/15 Date Finished: 6/16/15
VTSPG NAD83: N 780158.78 ft E 1598870.02 ft
Station: 17+39 Offset: 10L
Ground Elevation: 584.5 ft

	Casing	Sampler
Type:	FJC	SS
I.D.:	4 in	1.38 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	N.A.	30 in.
Hammer/Rod Type:	Auto/NW	
Rig:	CME 550 ATV	C_E = 1.4

Groundwater Observations		
Date	Depth (ft)	Notes
06/16/15		None observed

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Run (Dip deg.)	Core Rec. % (RQD %)	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
		Asphalt Pavement, 0.0 ft - 0.4 ft							
		(FILL), **f.c. GRAVEL**, Some f.c. Sand, little silt, dense, brown/gray, Moist Rec. = 0.7 ft			26-18-13 (31)	3.2	55.1	33.8	11.1
		(FILL)							
		A-2-4							
5		A-2-4, **f.c. SAND**,little silt, trace f.c. gravel, medium dense, brown, Moist, Rec. = 0.4 ft			8-8-9-8 (17)				
		A-2-4							
		(Completely Weathered Bedrock)							
10		(Completely Weathered Bedrock), **f.c. GRAVEL**, Some f.c. Sand, little silt, very dense, gray, Moist, Rec. = 0.8 ft			18-67-100 (167)	6.7	53.1	29.2	17.7
		11.0 ft - 14.5 ft, Gray, Phyllite, very close fracture spacing. Hard, Severely weathered, Poor rock, NXDC	R-1	71 (37)	Top of Bedrock @ 11.0 ft				
15		14.5 ft - 17.0 ft, Gray, Phyllite, very close fracture spacing. Hard, Slightly weathered, Very Poor rock, NXDC	R-2	80 (20)					
		17.0 ft - 22.0 ft, Gray, Phyllite, close fracture spacing. Hard, Slightly weathered, Good rock, NXDC	R-3	100 (90)					
20									
		22.0 ft - 27.0 ft, Gray, Phyllite, close fracture spacing. Hard, Moderately weathered, Fair rock, NXDC, 24'-25' mineral/quartz seam	R-4	100 (70)					
25									
		Hole stopped @ 27.0 ft							
30									
35									
40									

Remarks:
0': Auger through asphalt pavement. Layers identified through visual observation of borehole sidewalls.

14.5': Core barrel jam at 14.5' and 17'.

19.1': Uniaxial Compressive Strength, qu= 4,090 psi

25.7': Uniaxial Compressive Strength, qu= 5,680 psi

AASHTO classifications are based on visual description of sample recovery at depths where lab testing not performed.

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. C_E is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the time measurements were made.

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MATERIALS & RESEARCH SECTION
SUBSURFACE INFORMATION

BORING LOG

Johnson BF 0248(4)
VT 100C Bridge Improvements
Johnson, VT

Boring No.: **B-201**
Page No.: 1 of 1
Pin No.: 13c066
Checked By: CWS

Boring Crew: J. Leonhardt, K. Owens
Date Started: 6/17/15 Date Finished: 6/17/15
VTSPG NAD83: N 780161.59 ft E 1598573.68 ft
Station: 14+35 Offset: 5L
Ground Elevation: 581.0 ft

	Casing	Sampler
Type:	FJC	SS
I.D.:	4 in	1.38 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	N.A.	30 in.
Hammer/Rod Type:	Auto/NW	
Rig:	CME 550 ATV	C_E = 1.4

Groundwater Observations

Date	Depth (ft)	Notes
06/17/15	4.0	During drilling

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Run (Dip deg.)	Core Rec. % (RQD %)	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
		Asphalt Pavement, 0.0 ft - 0.5 ft							
		(FILL), **f.c. GRAVEL**, little f.c. sand, trace silt, dense, brown/gray, Moist, Rec. = 0.6 ft			87-18-15 (33)				
5		(FILL), **f.c. SAND**, trace silt, trace f.c. gravel, loose, brown, Wet, Rec. = 0.5 ft			6-4-4-3 (8)	5.9	2.1	90.5	7.4
		(FILL)							
10		(FILL), **Similar Soil**, Rec. = 0.3 ft			5-3-2-2 (5)				
		(FILL)							
15		(FILL), Grades to little silt,becomes very loose, Rec. = 0.3 ft			3-1-0-1 (1)				
		(FILL), **f.c. SAND**, little f.c. gravel, trace silt, very loose, brown, Wet, Rec. = 0.6 ft			2-1-1-2 (2)				
		Rec. = 1.3 ft18.0 ft - 19.5 ft, Concrete, NXDC	R-1	87	50/0" (R)				
20		(Completely Weathered Bedrock), **SILT**, Some f.c. Sand,little f. gravel, very dense, gray, Wet, Rec. = 1.0 ft 21.0 ft - 23.0 ft			22-27-36 (63)	14.0	18.9	29.4	51.7
25		23.0 ft - 28.0 ft, Gray, Phyllite, close fracture spacing, "6"" quartz seam at 25.5 ft.. Hard, Slightly weathered, Good rock, NXDC	R-2	94 (90)	Top of Bedrock @ 23.0 ft				
		Hole stopped @ 28.0 ft							
30									
35									
40									

Remarks:
0': Auger through asphalt pavement. Layers identified through visual observation of borehole sidewalls.

18': Uniaxial Compressive Strength, qu= 2,298 psi

19.3': Sudden decrease in drilling resistances at 19.3', interpreted as bottom of concrete.

19.3'-23': Consistent drilling resistance, interpreted as completely weathered bedrock.

23': Very hard drilling

24': Uniaxial Compressive Strength, qu= 5,200 psi

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. C_E is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the time measurements were made.

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MATERIALS & RESEARCH SECTION
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BORING LOG

Johnson BF 0248(4)
VT 100C Bridge Improvements
Johnson, VT

Boring No.: **B-202**
Page No.: 1 of 1
Pin No.: 13c066
Checked By: CWS

Boring Crew: J. Leonhardt, K. Owens
Date Started: 6/17/15 Date Finished: 6/18/15
VTSPG NAD83: N 780145.56 ft E 1598614.70 ft
Station: 14+80 Offset: 4R
Ground Elevation: 580.0 ft

	Casing	Sampler
Type:	FJC	SS
I.D.:	4 in	1.38 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	N.A.	30 in.
Hammer/Rod Type:	Auto/NW	
Rig:	CME 550 ATV	C_E = 1.4

Groundwater Observations

Date	Depth (ft)	Notes
06/17/15	8.0	During drilling

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Run (Dip deg.)	Core Rec. % (RQD %)	Drill Rate minutes/ft	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
		Asphalt Pavement, 0.0 ft - 0.4 ft								
		Subbase, 0.4 ft - 2.0 ft				14-10-12 (22)				
		Rec. = 0.5 ft								
		(FILL)								
		(FILL), **f.c. SAND**, trace silt, loose, brown, Moist, Rec. = 0.7 ft				4-3-3-4 (6)				
5		(FILL)								
		(FILL), becomes very loose, wet, Rec. = 0.5 ft				2-2-2-2 (4)				
10		(FILL)								
		(FILL), grades to trace f. gravel, Rec. = 0.3 ft				1-1-1-1 (2)				
15		(FILL), becomes very dense, Rec. = 0.6 ft, Spoon bouncing at 16.0', interpreted as top of concrete.				2-2-50/0" (R)	18.4	7.2	86.7	6.1
		16.0 ft - 19.0 ft, Gray, Concrete. Severely weathered, NXDC, 0.8' of recovery was moderately weathered concrete, 0.3' of recovery was concrete aggregate.	R-1	37	0.25					
20		(Completely Weathered Bedrock), **SILT**, Some f.c. Sand, trace f. gravel, very dense, gray, Moist, Rec. = 1.1 ft				26-56-100 (R)	9.1	11.7	31.3	57.0
		(Completely Weathered Bedrock), **Similar Soil**, Rec. = 1.3 ft				61-88-100/3" (R)				
		22.5 ft - 27.5 ft, Gray, Phyllite, medium fracture spacing. Medium hard, Moderately to severely weathered, Poor rock, NXDC, Severely weathered and very close fracture spacing 25.0-25.5'.	R-2	60 (50)	0.42	Top of Bedrock @ 22.5 ft				
25		27.5 ft - 30.5 ft, Gray, Phyllite, very close fracture spacing. Medium hard, Severely weathered, Very Poor rock, NXDC	R-3	83 (20)	0.5					
30		Hole stopped @ 30.5 ft								
35										
40										

Remarks:
0': Auger through asphalt pavement. Layer identified through visual observation of borehole sidewalls.

16': Uniaxial Compressive Strength, qu= 4,810 psi

19': Drilling resistance suddenly decreased at 19', interpreted as bottom of concrete.

22.5': Rollerbit refusal at 22.5', interpreted as top of competent bedrock.

25': Less drilling resistance 25-26', 29-30'.

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
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3. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the time measurements were made.

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SUBSURFACE INFORMATION

BORING LOG

Johnson BF 0248(4)
VT 100C Bridge Improvements
Johnson, VT

Boring No.: **B-203**
Page No.: 1 of 1
Pin No.: 13c066
Checked By: CWS

Boring Crew: J. Leonhardt, K. Owens
Date Started: 6/16/15 Date Finished: 6/17/15
VTSPG NAD83: N 780147.13 ft E 1598710.46 ft
Station: 15+74 Offset: 4L
Ground Elevation: 580.0 ft

	Casing	Sampler
Type:	FJC	SS
I.D.:	4 in	1.38 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	N.A.	30 in.
Hammer/Rod Type:	Auto/NW	
Rig:	CME 550 ATV	C_E = 1.4

Groundwater Observations

Date	Depth (ft)	Notes
06/16/15	14.0	During drilling

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Run (Dip deg.)	Core Rec. % (RQD %)	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
		Asphalt Pavement, 0.0 ft - 0.5 ft							
		(FILL), **f.c. GRAVEL**, AND f.c. SAND, trace silt, dense, Moist, Rec. = 0.6 ft			27-14-12 (26)				
		(FILL)							
5		(FILL), **f.c. GRAVEL and f.c. SAND**, little silt, loose, brown, Moist, Rec. = 0.8 ft			5-3-3-2 (6)	11.2	43.1	44.8	12.1
		(FILL)							
10		(FILL), **Similar Soil**, Rec. = 0.6 ft			11-6-4-3 (10)				
		(FILL)							
15		(FILL), **f.c. SAND**, trace silt, trace f. gravel, loose, brown, Wet, Rec. = 0.4 ft			7-4-2-2 (6)				
		(FILL)							
20		(Completely Weathered Bedrock), **SILT**, Some f.c. Sand,Some f.c. Gravel, medium dense, gray, Moist, Rec. = 0.9 ft			6-6-14-51 (20)	14.3	30.9	30.7	38.4
		21.0 ft - 22.0 ft, Gray, Phyllite, Hard, Fair rock, NXDC	R-1	60					
		(Completely Weathered Bedrock), **f. SAND**, Some Silt, trace f. gravel very dense, gray, Moist, Rec. = 0.5 ft			44-100/2"				
		(Completely Weathered Bedrock)							
25		24.0 ft - 26.0 ft, Gray, Phyllite, Hard, Good rock, NXDC	R-2	100 (80)	Top of Bedrock @ 24.0 ft				
		(Completely Weathered Bedrock)							
30		28.5 ft - 31.5 ft, Gray, Phyllite, highly weathered, very close fracture spacing. Hard, Poor rock, NXDC	R-3	100 (49)					
		31.5 ft - 32.8 ft, becomes slightly weathered at 31.5'							
		Hole stopped @ 32.8 ft							
35									
40									

Remarks:
0': Auger through asphalt pavement. Layers identified through visual observation of borehole sidewalls.

19': Casing driving resistance increased at 19' interpreted as top of completely weathered bedrock.

26': Core barrel jammed at 26' and 32.8', likely due to high degree of weathering. Advanced roller bit 26'-28.5' to obtain rock core. Interpreted as completely weathered bedrock.

31.5': Uniaxial Compressive Strength, qu= 3,430 psi

Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. C_E is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the time measurements were made.

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MATERIALS & RESEARCH SECTION
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BORING LOG

Johnson BF 0248(4)
VT 100C Bridge Improvements
Johnson, VT

Boring No.: **B-204**
Page No.: 1 of 1
Pin No.: 13c066
Checked By: CWS

Boring Crew: J. Leonhardt, K. Owens
Date Started: 6/18/15 Date Finished: 6/18/15
VTSPG NAD83: N 780130.84 ft E 1598773.40 ft
Station: 16+35 Offset: 5R
Ground Elevation: 580.1 ft

	Casing	Sampler
Type:	FJC	SS
I.D.:	4 in	1.38 in
Hammer Wt:	300 lb.	140 lb.
Hammer Fall:	N.A.	30 in.
Hammer/Rod Type:	Auto/NW	
Rig:	CME 550 ATV	C_E = 1.4

Groundwater Observations

Date	Depth (ft)	Notes
06/18/15	4.0	During drilling

Depth (ft)	Strata (1)	CLASSIFICATION OF MATERIALS (Description)	Run (Dip deg.)	Core Rec. % (RQD %)	Blows/6" (N Value)	Moisture Content %	Gravel %	Sand %	Fines %
		Asphalt Pavement, 0.0 ft - 0.5 ft							
		(FILL), **f.c. GRAVEL**, little f.c. sand,trace silt, dense, brown, Moist, Rec. = 0.7 ft			27-19-15 (34)				
		(FILL)							
5		(FILL), **f.c. SAND**,trace silt, trace f. gravel, loose, brown, Wet, Rec. = 0.7 ft			5-3-4-2 (7)	17.4	2.4	90.2	7.4
		(FILL)							
10		(FILL), **f.c. SAND**, little silt, little f.c. gravel, dense, brown, Wet, Rec. = 0.9 ft			6-15-10-17 (25)				
		(FILL)							
15		(FILL), **f.c. SAND**, trace silt, very loose, brown, Wet, Rec. = 0.6 ft			1-1-0-25 (1)				
		Wood fragments, 14.3 ft - 14.5 ft							
		(Completely Weathered Bedrock), **f.c. GRAVEL**, little silt, little f.c. sand, very loose, gray, Moist	R-1	86 (46)	Top of Bedrock @ 16.0 ft				
20		16.0 ft - 21.0 ft, Gray, Phyllite, very close fracture spacing. Medium hard, Slightly to moderately weathered, Poor rock, NXDC							

Hole stopped @ 21.0 ft

Remarks:
0': Auger through asphalt pavement. Layers identified through visual observation of borehole sidewalls.

9': Hard casing during 9'-14'.

16': Spoon bouncing at 16'.

18.5': Uniaxial Compressive Strength, qu= 3,720 psi

25
30
35
40

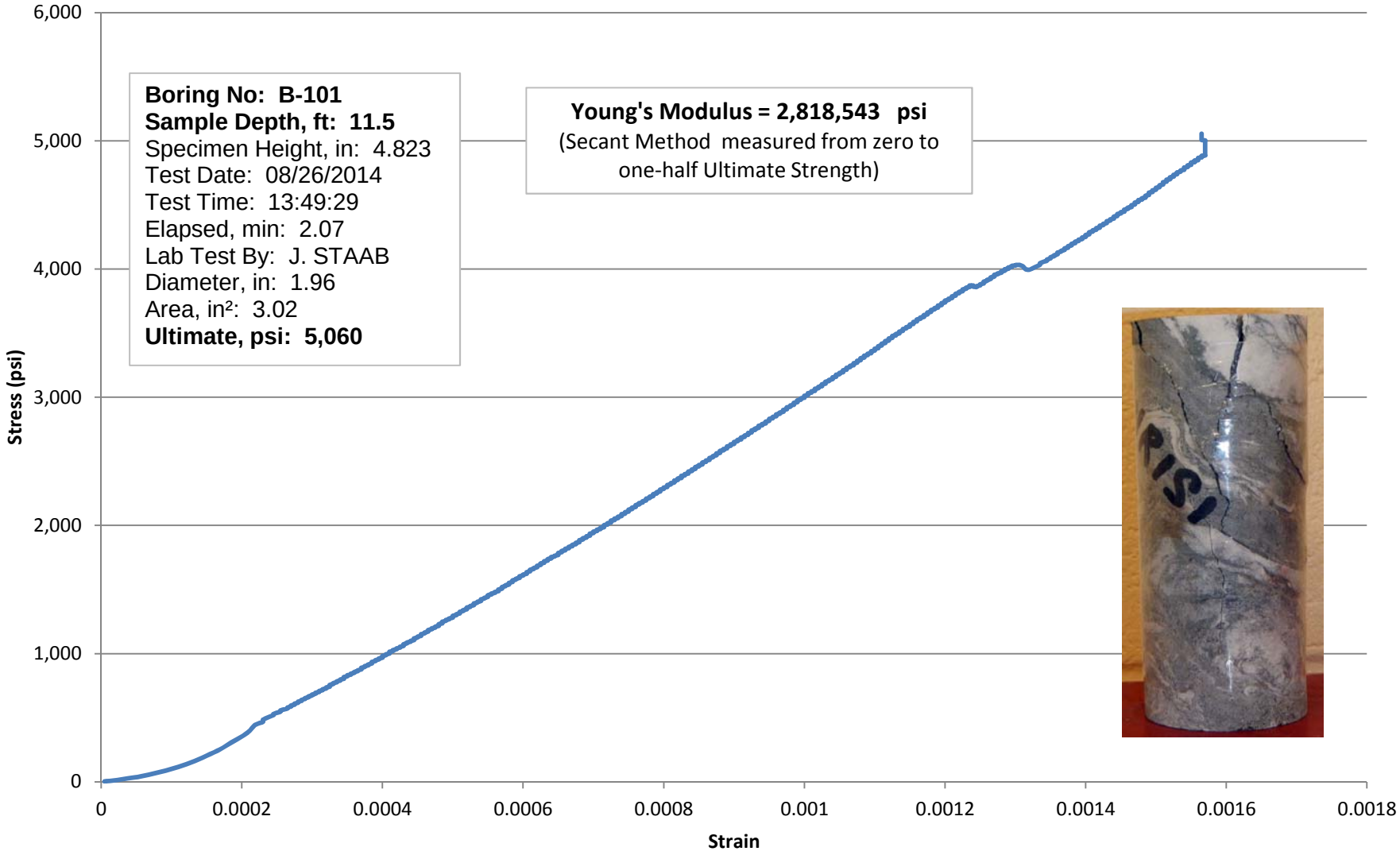
Notes:
1. Stratification lines represent approximate boundary between material types. Transition may be gradual.
2. N Values have not been corrected for hammer energy. C_E is the hammer energy correction factor.
3. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the time measurements were made.

APPENDIX D

LABORATORY TEST RESULTS

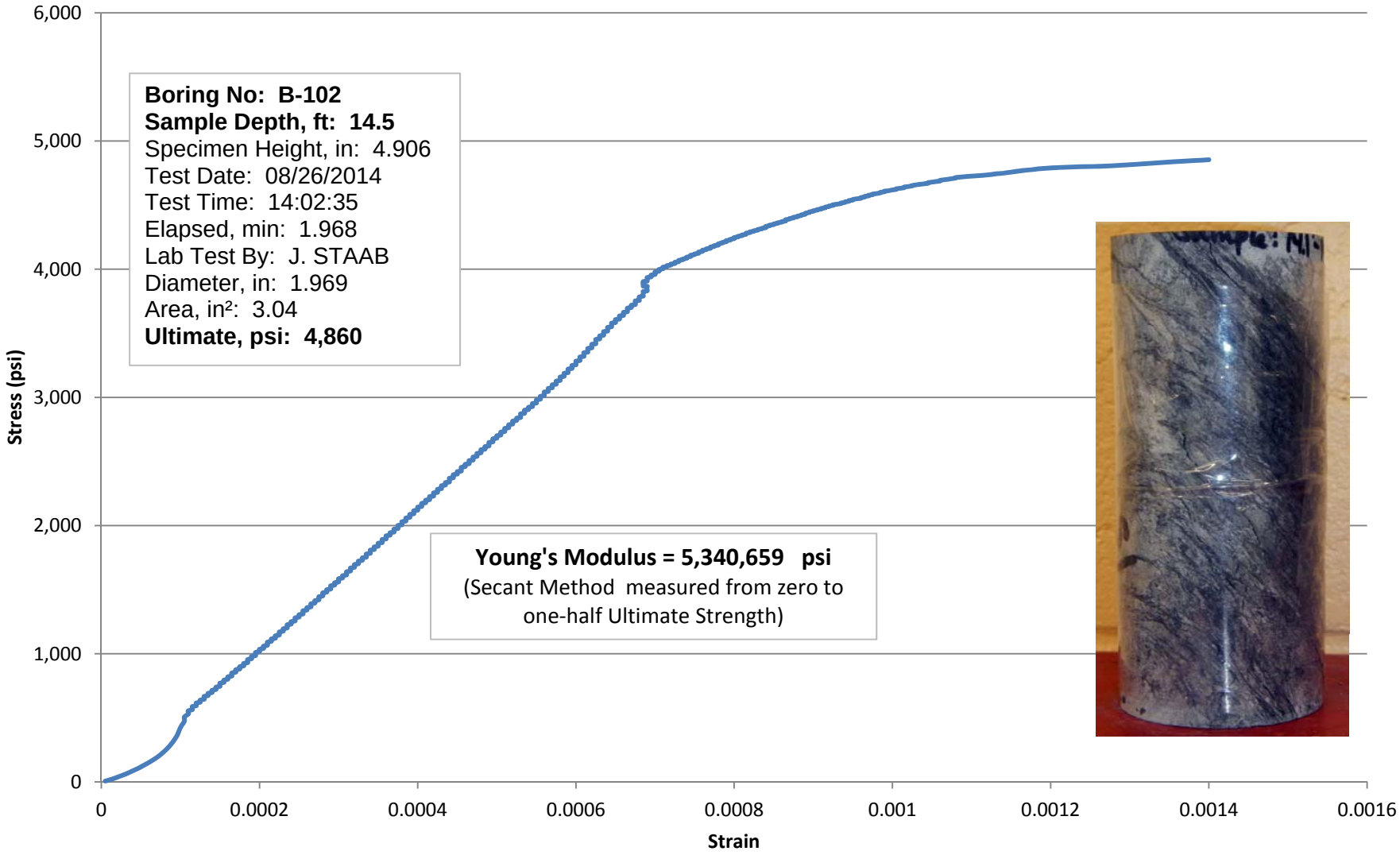
Checked By: CEE

JOHNSON BF 0248(4)
Uniaxial Compressive Strength
Laboratory Sample Number B101R1S1



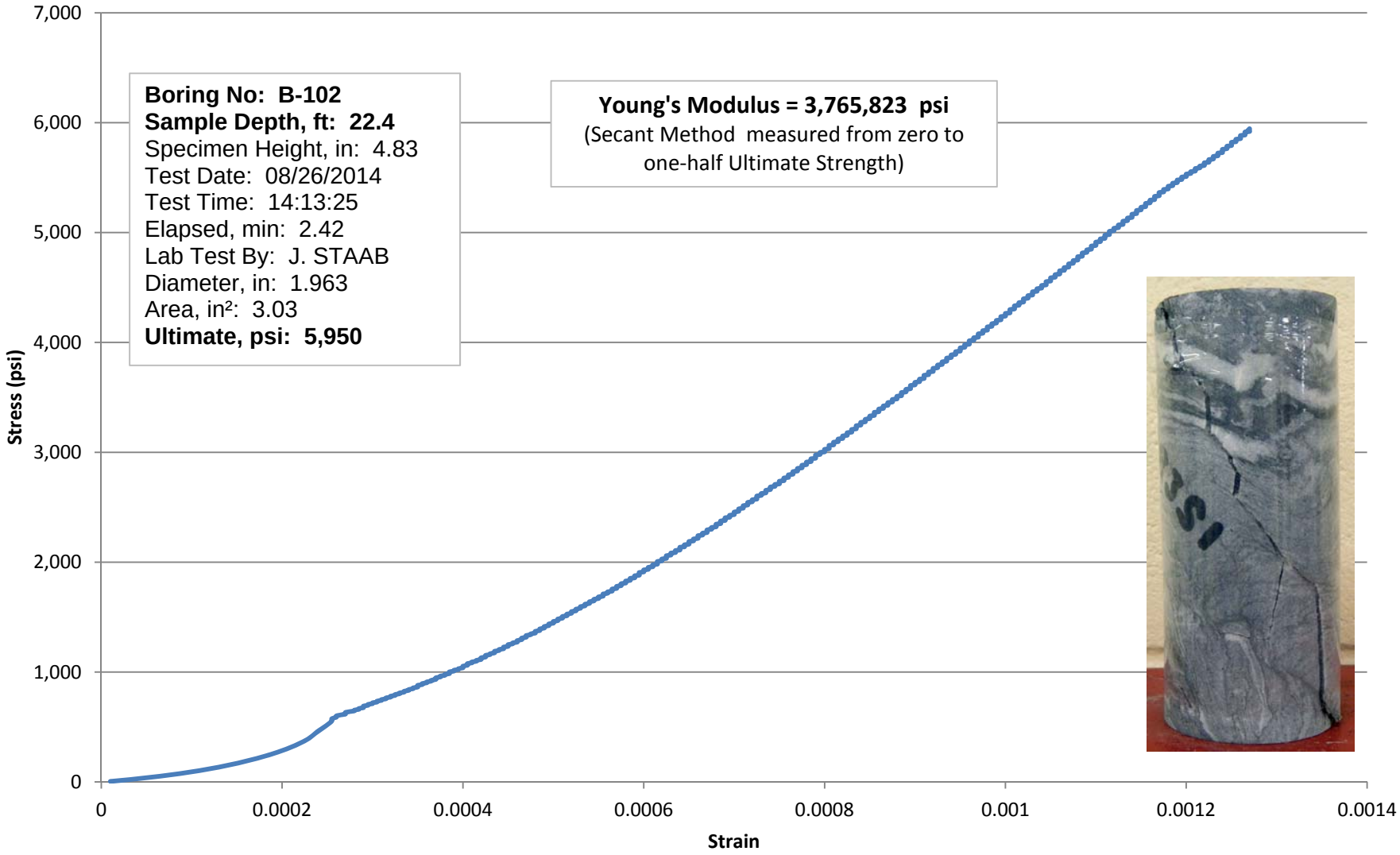
Checked By: CEE

JOHNSON BF 0248(4)
Uniaxial Compressive Strength
Laboratory Sample Number B102R1S1



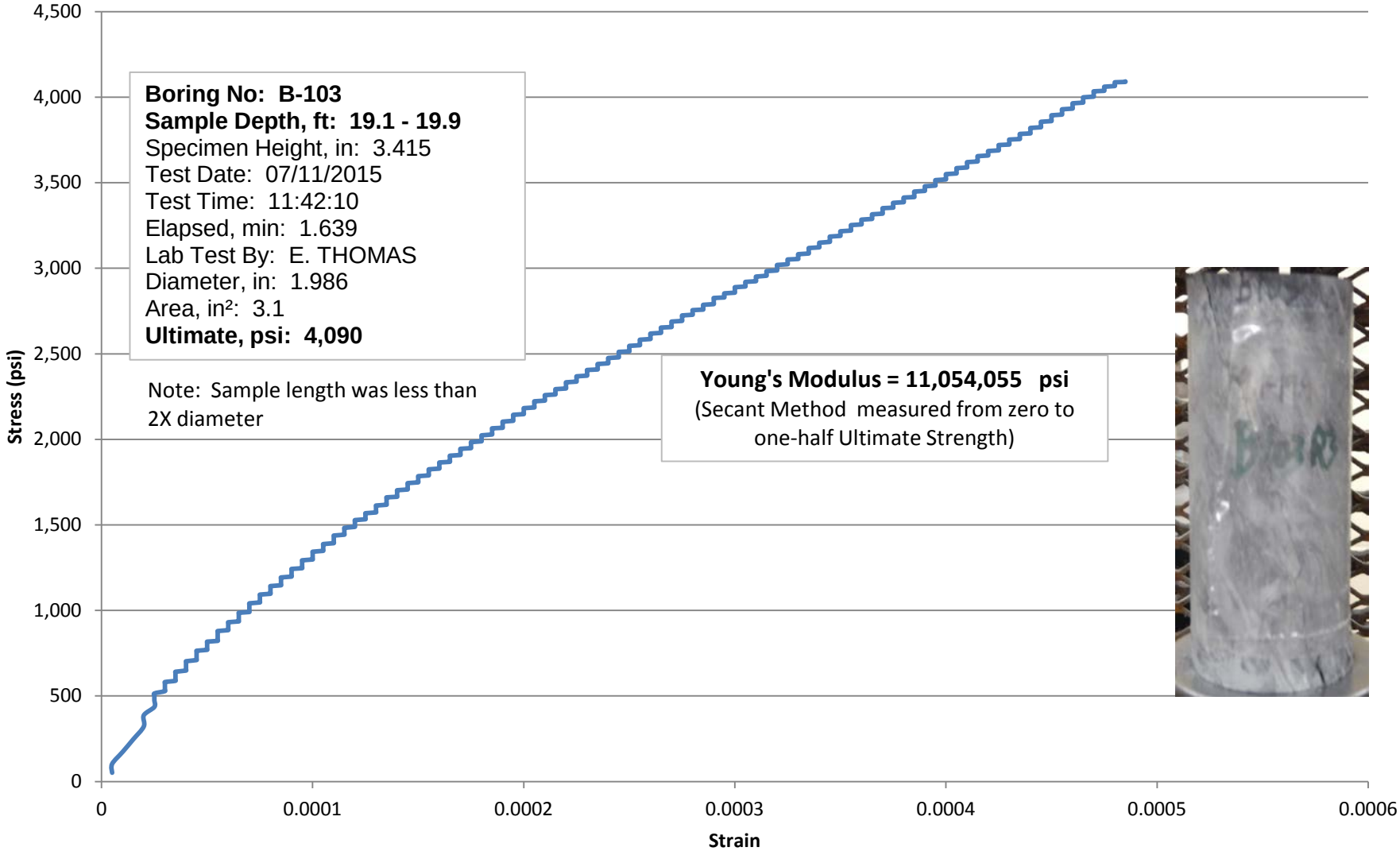
Checked By: CEE

JOHNSON BF 0248(4)
Uniaxial Compressive Strength
Laboratory Sample Number B102R3S1



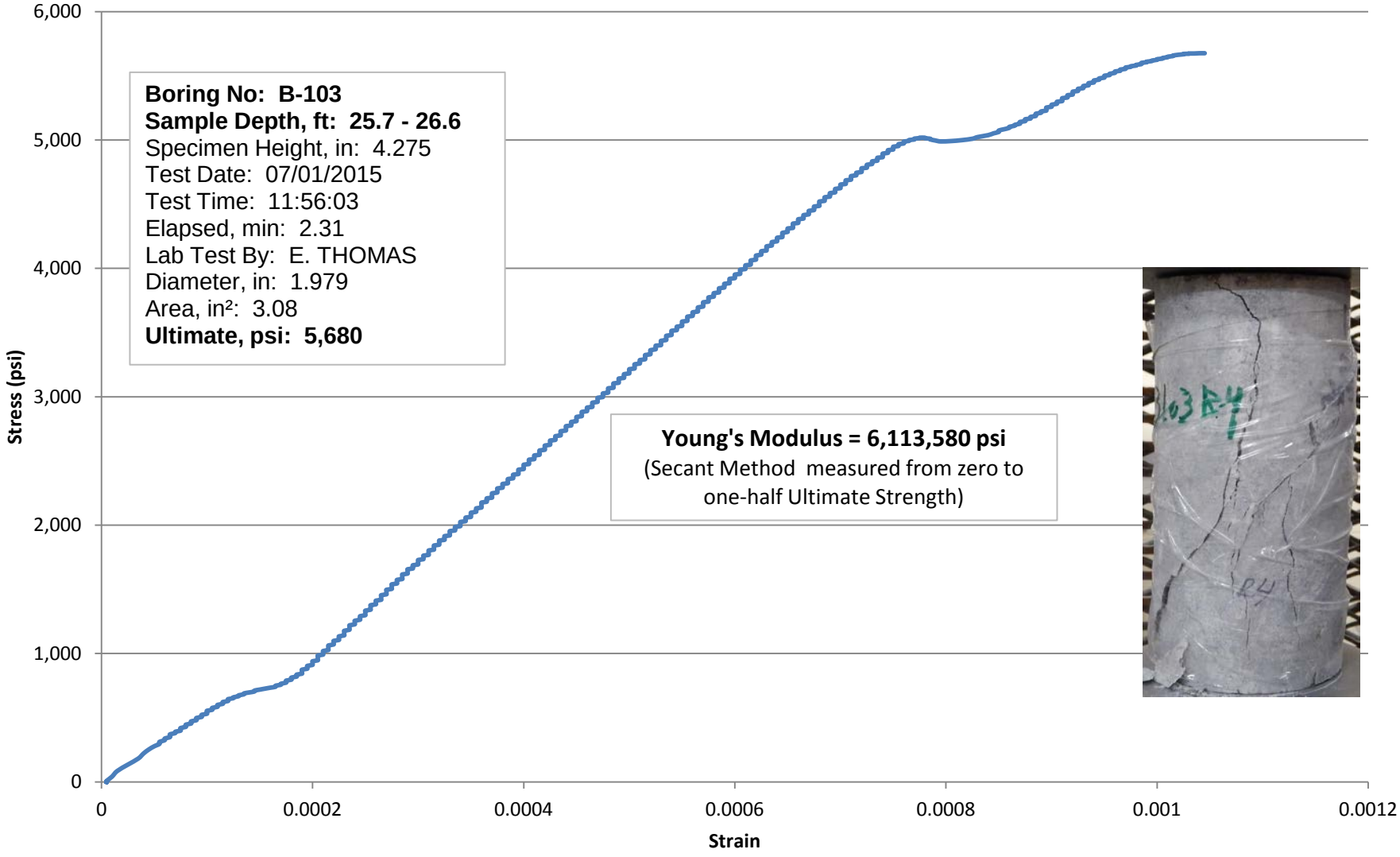
Checked By: TDE

JOHNSON BF 0248(4)
Uniaxial Compressive Strength
Laboratory Sample Number B103R3S1



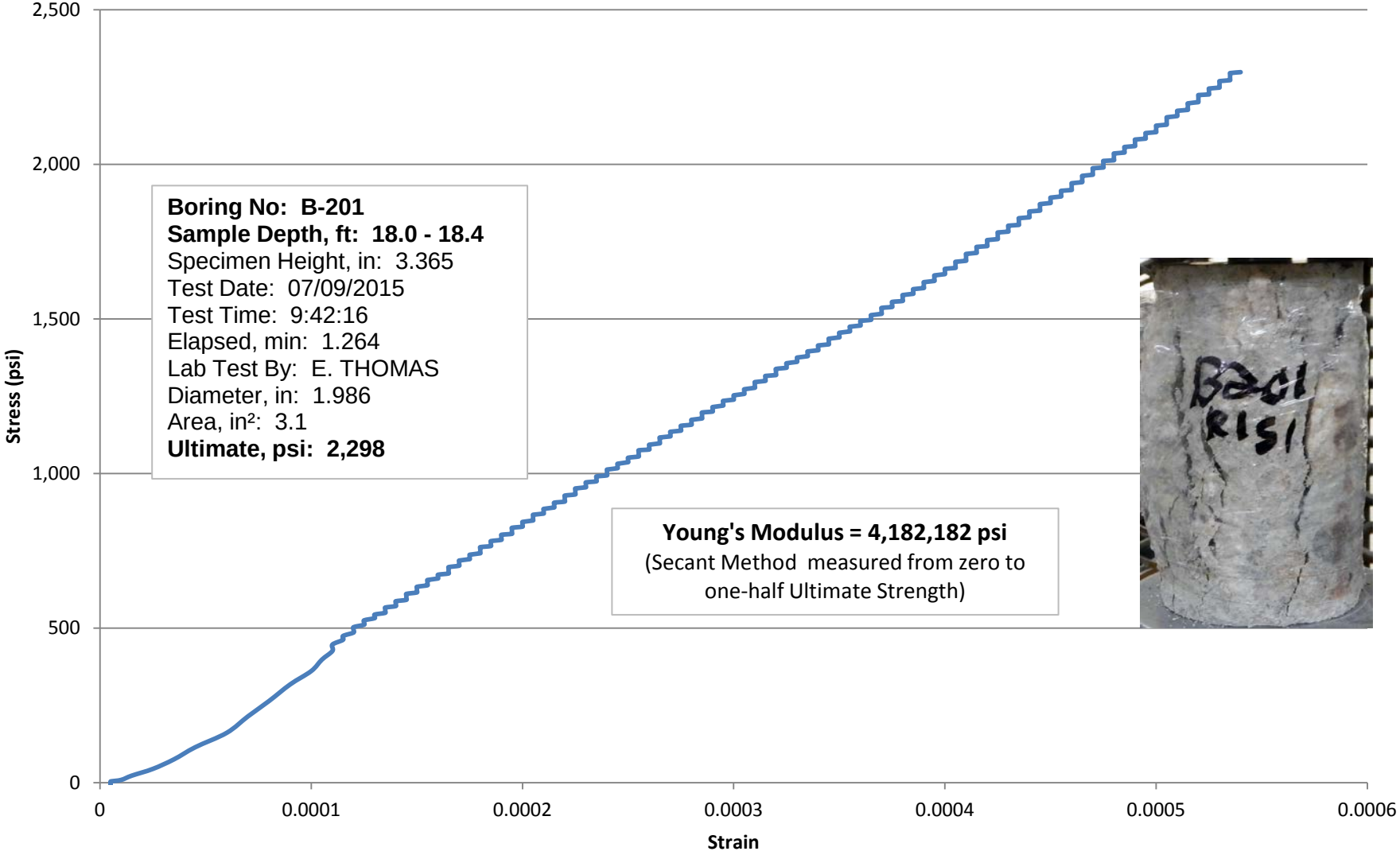
Checked By: TDE

JOHNSON BF 0248(4)
Uniaxial Compressive Strength
Laboratory Sample Number B103R4S1



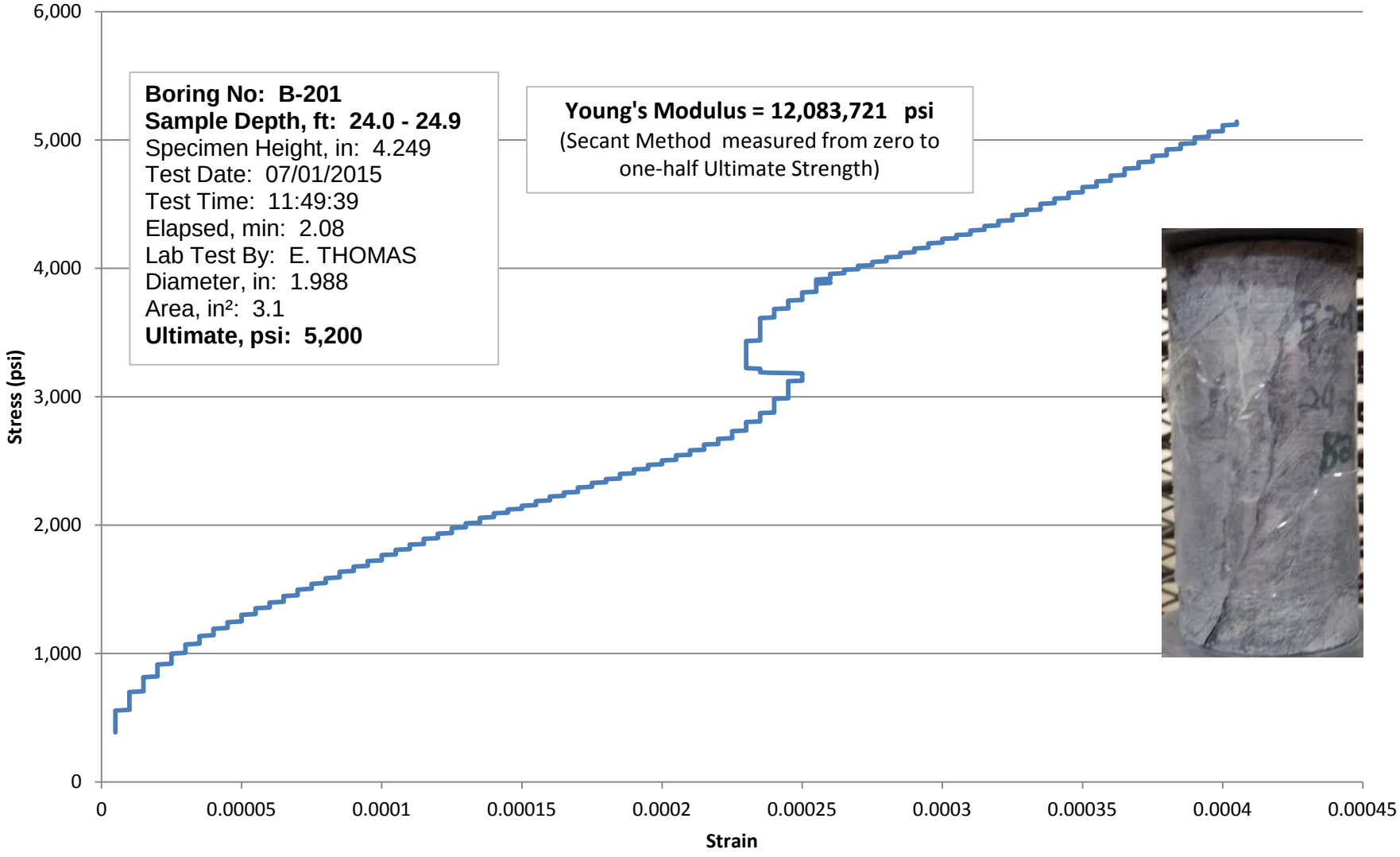
Checked By: TDE

JOHNSON BF 0248(4)
Uniaxial Compressive Strength
Laboratory Sample Number B201R1S1



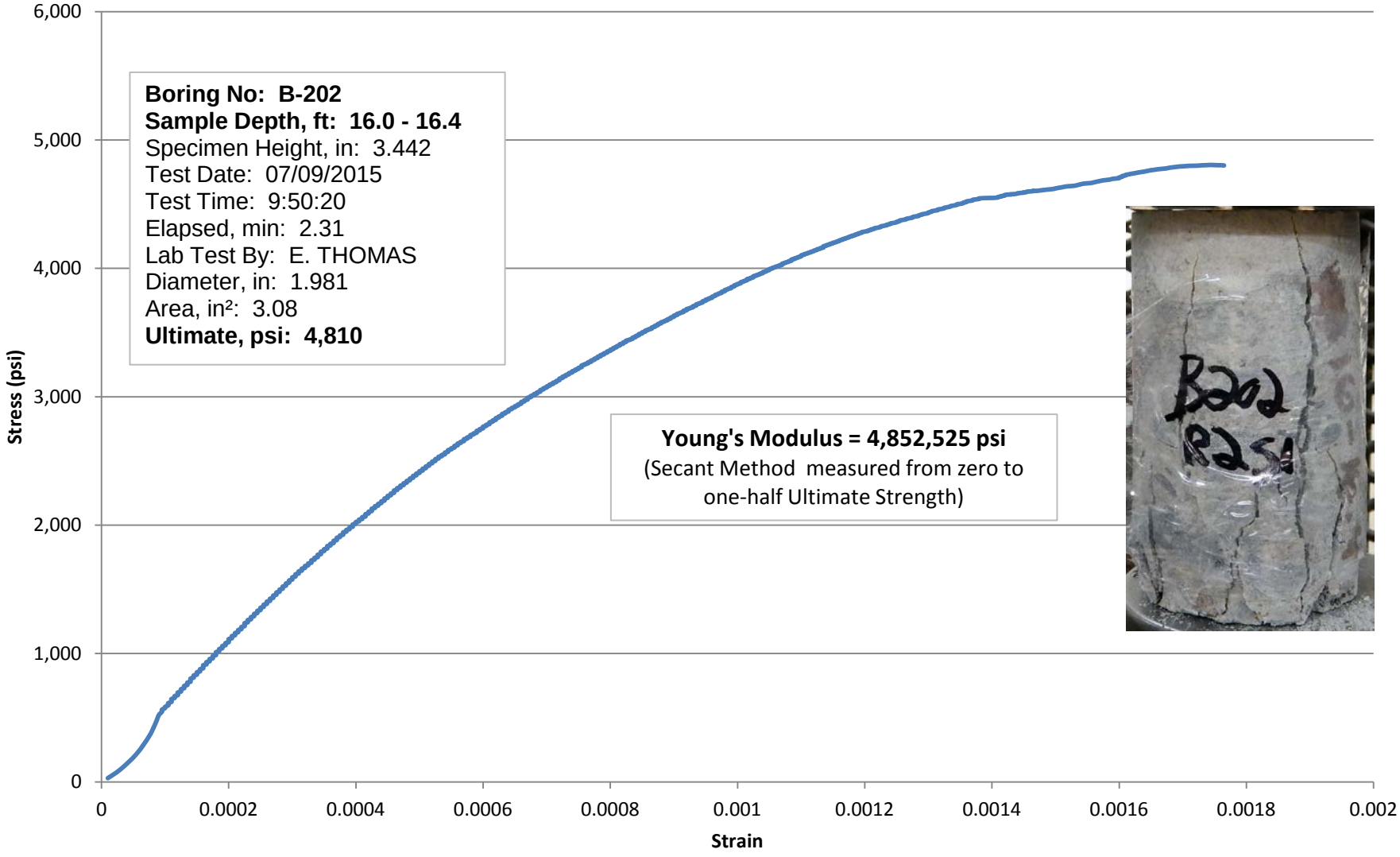
Checked By: TDE

JOHNSON BF 0248(4)
Uniaxial Compressive Strength
Laboratory Sample Number B201R2S1



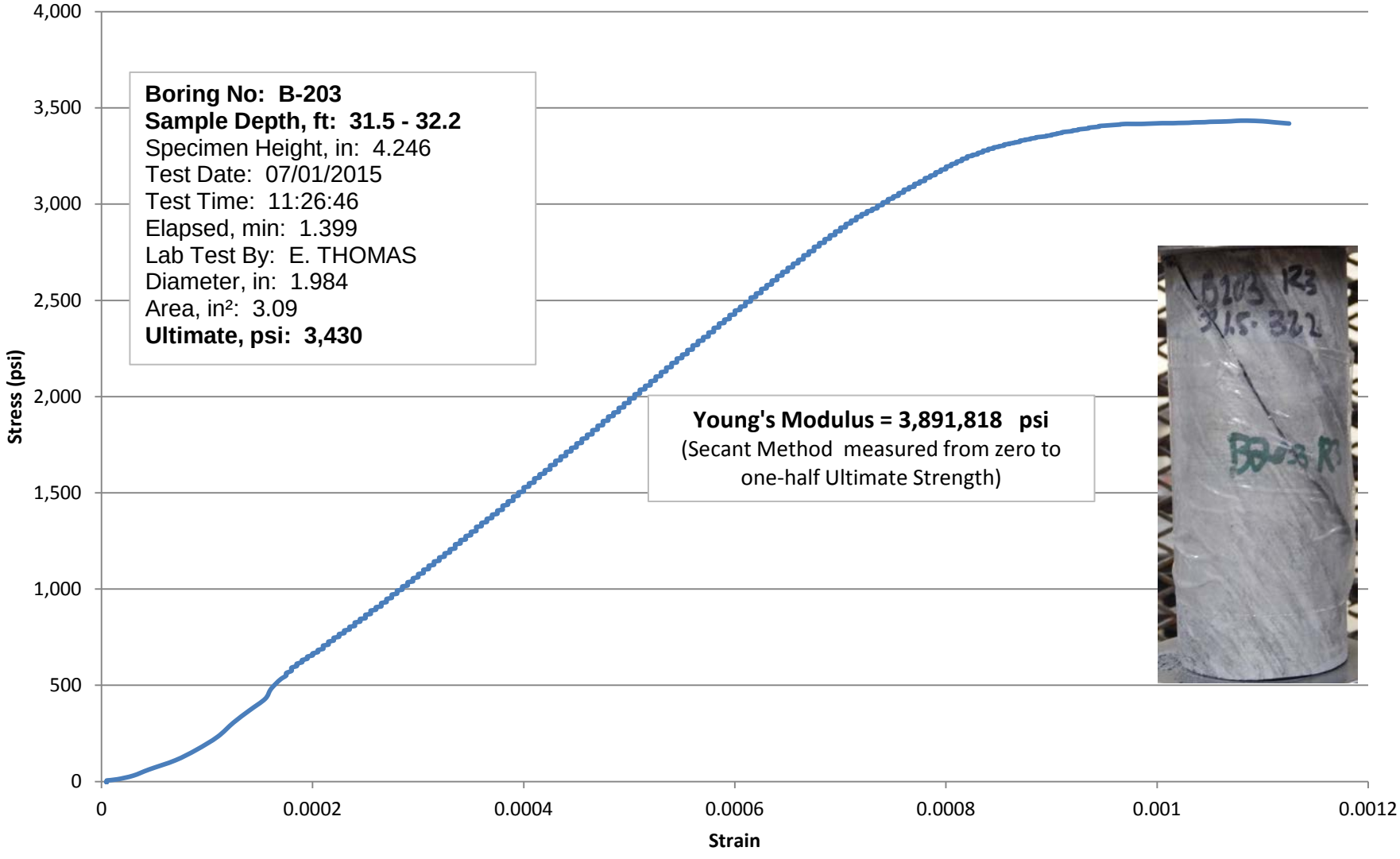
Checked By: TDE

JOHNSON BF 0248(4)
Uniaxial Compressive Strength
Laboratory Sample Number B202R2S1



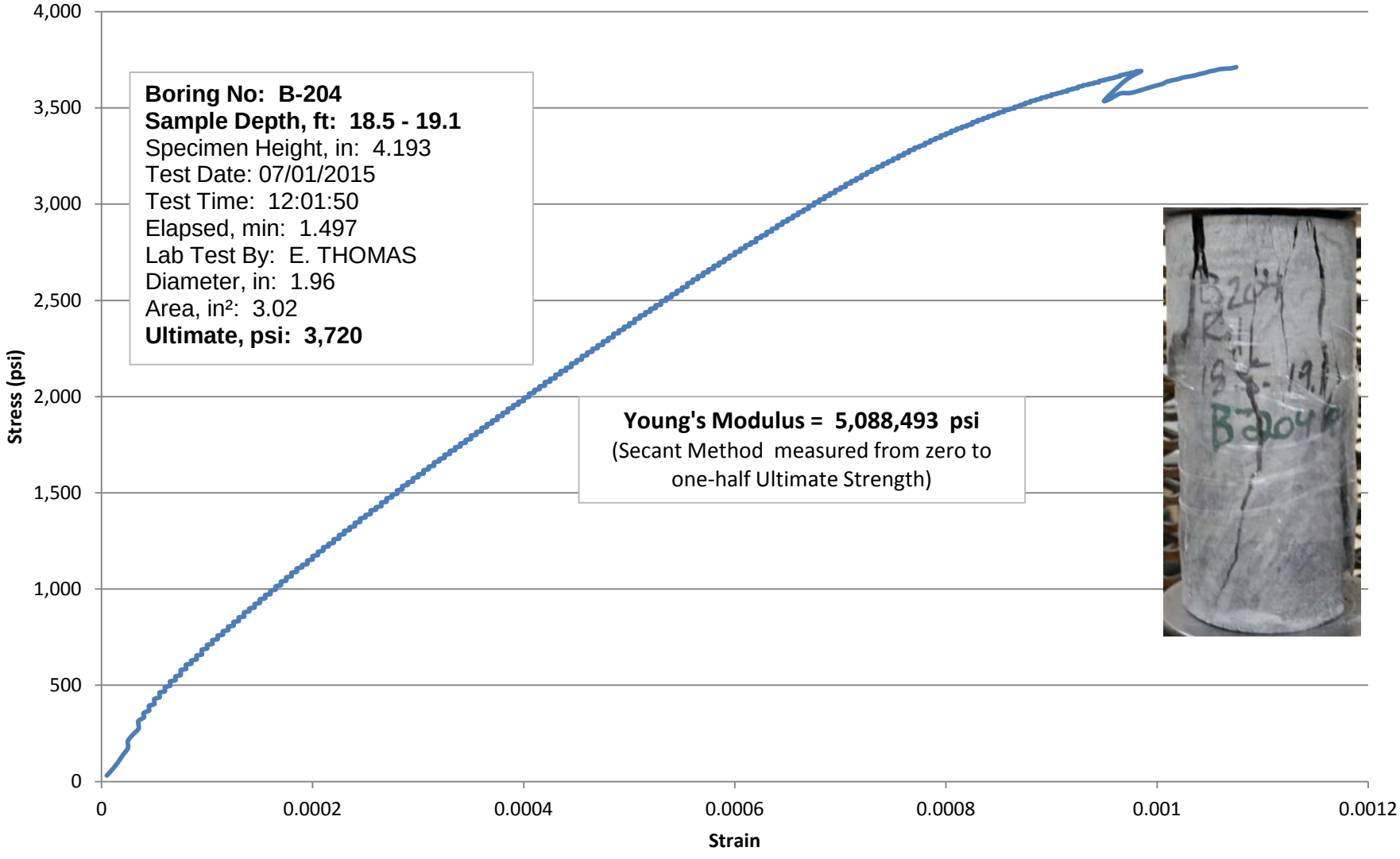
Checked By: TDE

JOHNSON BF 0248(4)
Uniaxial Compressive Strength
Laboratory Sample Number B203R3S1



Checked By: TDE

JOHNSON BF 0248(4)
Uniaxial Compressive Strength
Laboratory Sample Number B204R1S1



Vermont Agency of Transportation
Materials and Research Section
1 National Life Drive
Montpelier, VT 05633-5001

Distribution list
CHA
T. ELIASSEN
J. TOUCHETTE

Report on Soil Sample

Lab number: E141358 **Corrected copy:** N/A **Report Date:** 8/25/2014 2:55:49 P
Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR-2

Date sampled: 8/11/2014 **Received:** 8/22/2014 **Tested:** 8/22/2014 **Tested by:** J. TOUCHETTE
Station: **Offset:** **Hole:** B-101 **Depth:** 7 FT **to:** 9 FT
Field description: Sa & Si
Submitted by: CHA **Address:**
Sample type: SPLIT BARREL **Quantity:**
Sample source/Outside agency name:
Location used: **Examined for:** MC, GS
Comment:

Test Results

T-88	Sieve Analysis % Passing Total Sample
75 mm (3.0"):	
37.5 mm (1.5"):	
19 mm (3/4"):	
9.5 mm (3/8"):	
4.75 mm (#4):	99.9%
2.00 mm (#10):	99.7%
850 µm (#20):	98.7%
425 µm (#40):	92.3%
250 µm (#60):	75.4%
150 µm (#100):	52.5%
75 µm (#200):	25.4%

Hydrometer Analysis

Particles smaller	% total sample
0.05 mm:	
0.02 mm:	
0.005 mm:	
0.002 mm:	
0.001 mm:	

Limits

T-265 Moisture content: 24.3%
T-89 Liquid Limit:
T-90 Plastic Limit:
T-90 Plasticity Index: NP

Moisture Density

Test method: T-180 **Method:**
Maximum density: **pcf**
Optimum moisture:
T-100 Specific Gravity:

Gr: 0.3% **D2487:** SM
Sa: 74.4% **M145:** A-2-4 Silty Sand
Si: 25.4%

Comments:

Reviewed by: T. Eliassen, P.G., Transportation Geologist

Distribution list

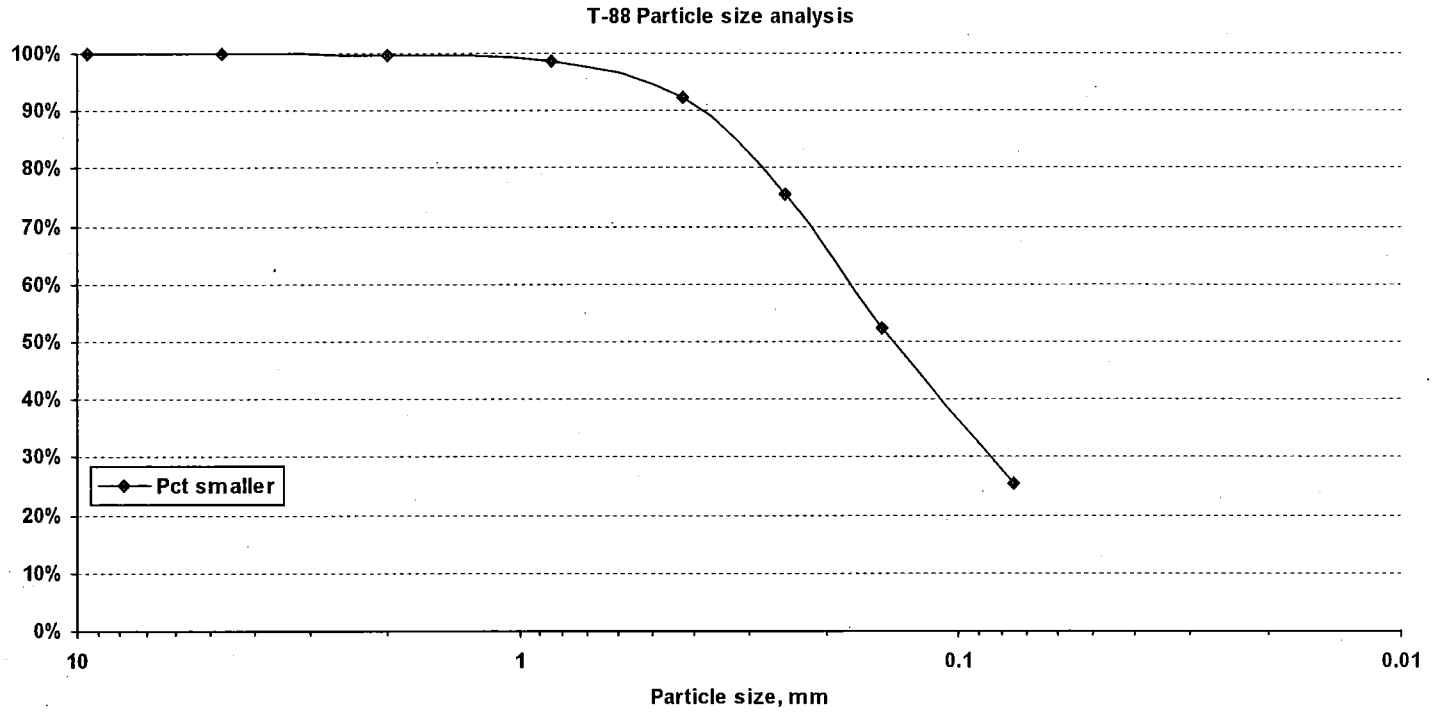
Vermont Agency of Transportation
Materials and Research Section
1 National Life Drive
Montpelier, VT 05633-5001

Report on Soil Sample

Lab number: E141358 **Corrected copy:** N/A **Report Date:** 8/25/2014 2:55:53 P

Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR-2

Hole: B-101 **Depth:** 7 FT - 9 FT



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Distribution list
CHA
T. ELIASSEN
J. TOUCHETTE

Report on Soil Sample

Lab number: E141359 **Corrected copy:** N/A **Report Date:** 8/25/2014 2:58:06 P
Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR-2

Date sampled: 8/11/2014 **Received:** 8/22/2014 **Tested:** 8/22/2014 **Tested by:** J. TOUCHETTE
Station: **Offset:** **Hole:** B-102 **Depth:** 5 FT **to:** 7 FT
Field description: Sa & Si
Submitted by: CHA **Address:**
Sample type: SPLIT BARREL **Quantity:**
Sample source/Outside agency name:
Location used: **Examined for:** MC, GS
Comment:

Test Results

Sieve Analysis

T-88	% Passing Total Sample
75 mm (3.0"):	
37.5 mm (1.5"):	
19 mm (3/4"):	92.3%
9.5 mm (3/8"):	91.5%
4.75 mm (#4):	88.4%
2.00 mm (#10):	78.8%
850 µm (#20):	69.1%
425 µm (#40):	61.3%
250 µm (#60):	55.4%
150 µm (#100):	52.3%
75 µm (#200):	50.8%

Hydrometer Analysis

Particles smaller	% total sample
0.05 mm:	
0.02 mm:	
0.005 mm:	
0.002 mm:	
0.001 mm:	

Limits

T-265 Moisture content: 10.5%
T-89 Liquid Limit:
T-90 Plastic Limit:
T-90 Plasticity Index: NP

Moisture Density

Test method: T-180 **Method:**
Maximum density: **pcf**
Optimum moisture:
T-100 Specific Gravity:

Gr: 21.2% **D2487:** ML
Sa: 27.9% **M145:** A-4 Gravelly Sandy Silt
Si: 50.8%

Comments:

Reviewed by: T. Eliassen, P.G., Transportation Geologist TE

Distribution list

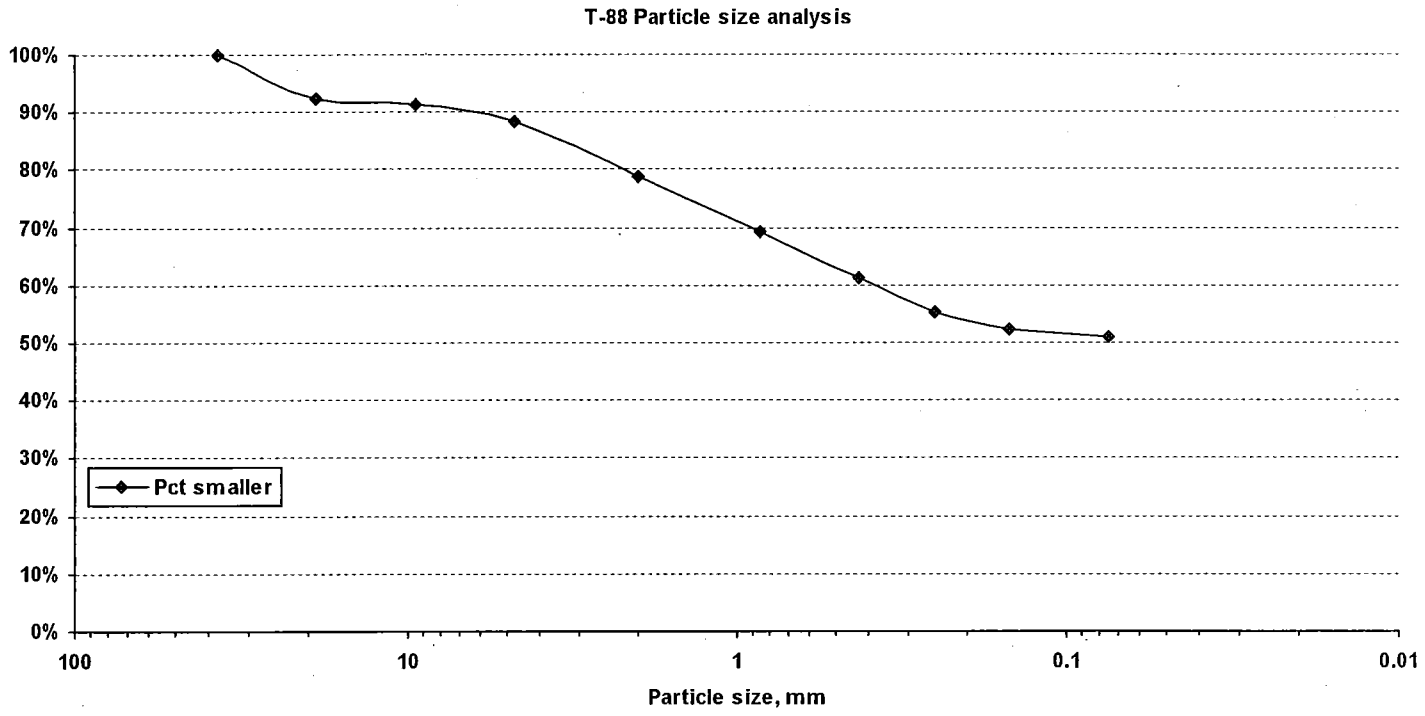
Vermont Agency of Transportation
Materials and Research Section
1 National Life Drive
Montpelier, VT 05633-5001

Report on Soil Sample

Lab number: E141359 **Corrected copy:** N/A **Report Date:** 8/25/2014 2:58:11 P

Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR-2

Hole: B-102 **Depth:** 5 FT - 7 FT



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Distribution list
CHA
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J. TOUCHETTE

Report on Soil Sample

Lab number: E141360 **Corrected copy:** N/A **Report Date:** 8/25/2014 2:59:47 P
Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR-2

Date sampled: 8/11/2014 **Received:** 8/22/2014 **Tested:** 8/22/2014 **Tested by:** J. TOUCHETTE
Station: **Offset:** **Hole:** B-102 **Depth:** 9 FT **to:** 11 FT
Field description: Sa & Si
Submitted by: CHA **Address:**
Sample type: SPLIT BARREL **Quantity:**
Sample source/Outside agency name:
Location used: **Examined for:** MC, GS
Comment:

Test Results

Sieve Analysis

T-88	% Passing Total Sample
75 mm (3.0"):	
37.5 mm (1.5"):	
19 mm (3/4"):	
9.5 mm (3/8"):	
4.75 mm (#4):	97.6%
2.00 mm (#10):	91.5%
850 µm (#20):	84.2%
425 µm (#40):	76.1%
250 µm (#60):	63.9%
150 µm (#100):	47.4%
75 µm (#200):	13.7%

Hydrometer Analysis

Particles smaller	% total sample
0.05 mm:	
0.02 mm:	
0.005 mm:	
0.002 mm:	
0.001 mm:	

Limits

T-265 Moisture content: 27.0%
T-89 Liquid Limit:
T-90 Plastic Limit:
T-90 Plasticity Index: NP

Moisture Density

Test method: T-180 **Method:**
Maximum density: **pcf**
Optimum moisture:
T-100 Specific Gravity:

Gr: 8.5% **D2487:** SM
Sa: 77.9% **M145:** A-2-4 Sand
Si: 13.7%

Comments:

Reviewed by: T. Eliassen, P.G., Transportation Geologist

Distribution list

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Materials and Research Section
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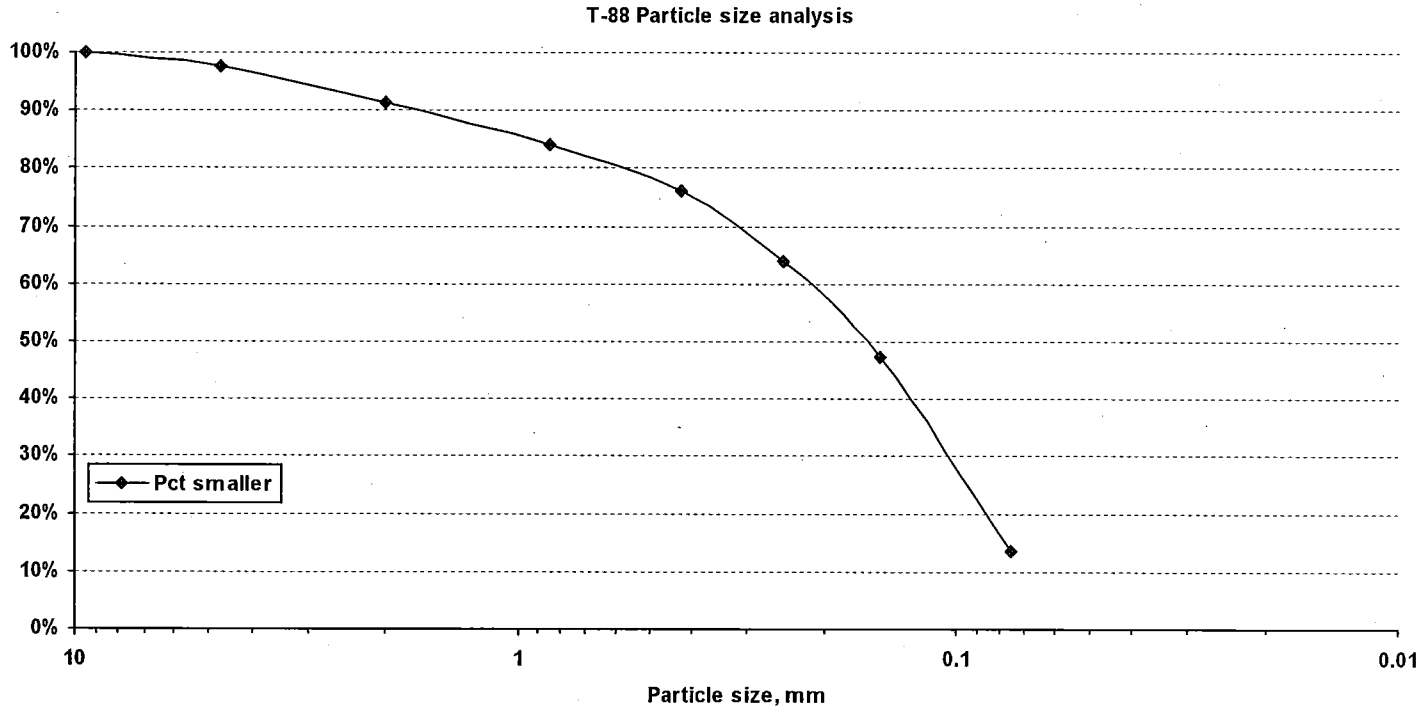
Montpelier, VT 05633-5001

Report on Soil Sample

Lab number: E141360 **Corrected copy:** N/A **Report Date:** 8/25/2014 2:59:53 P

Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR-2

Hole: B-102 **Depth:** 9 FT - 11 FT



State of Vermont
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Distribution list
CHA
T. ELIASSEN
J. TOUCHETTE

Report on Soil Sample

Lab number: E151005 **Corrected copy:** N/A **Report Date:** 7/7/2015 2:22:36 P
Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR- 1&2

Date sampled: 6/16/2015 **Received:** 7/6/2015 **Tested:** 7/6/2015 **Tested by:** J. TOUCHETTE
Station: **Offset:** **Hole:** B-103 **Depth:** 0.5 FT **to:** 2 FT
Field description: Sa & Gr
Submitted by: CHA **Address:**
Sample type: SPLIT BARREL **Quantity:**
Sample source/Outside agency name:
Location used: **Examined for:** MC, GS
Comment: S-1

Test Results

Sieve Analysis

T-88	% Passing Total Sample
75 mm (3.0"):	
37.5 mm (1.5"):	
19 mm (3/4"):	90.8%
9.5 mm (3/8"):	76.5%
4.75 mm (#4):	59.8%
2.00 mm (#10):	44.9%
850 µm (#20):	33.0%
425 µm (#40):	25.3%
250 µm (#60):	20.5%
150 µm (#100):	16.7%
75 µm (#200):	11.1%

Hydrometer Analysis

Particles smaller	% total sample
0.05 mm:	
0.02 mm:	
0.005 mm:	
0.002 mm:	
0.001 mm:	

Limits

T-265 Moisture content: 3.2%
T-89 Liquid Limit:
T-90 Plastic Limit:
T-90 Plasticity Index: NP

Moisture Density

Test method: T-180 **Method:**
Maximum density: **pcf**
Optimum moisture:
T-100 Specific Gravity:

Gr: 55.1% **D2487:** SP-SM
Sa: 33.8% **M145:** A-1-a Sandy Gravel
Si: 11.1%

Comments:

Reviewed by: T. Eliassen, P.G., Transportation Geologist

CEE

For TDE

Distribution list

State of Vermont
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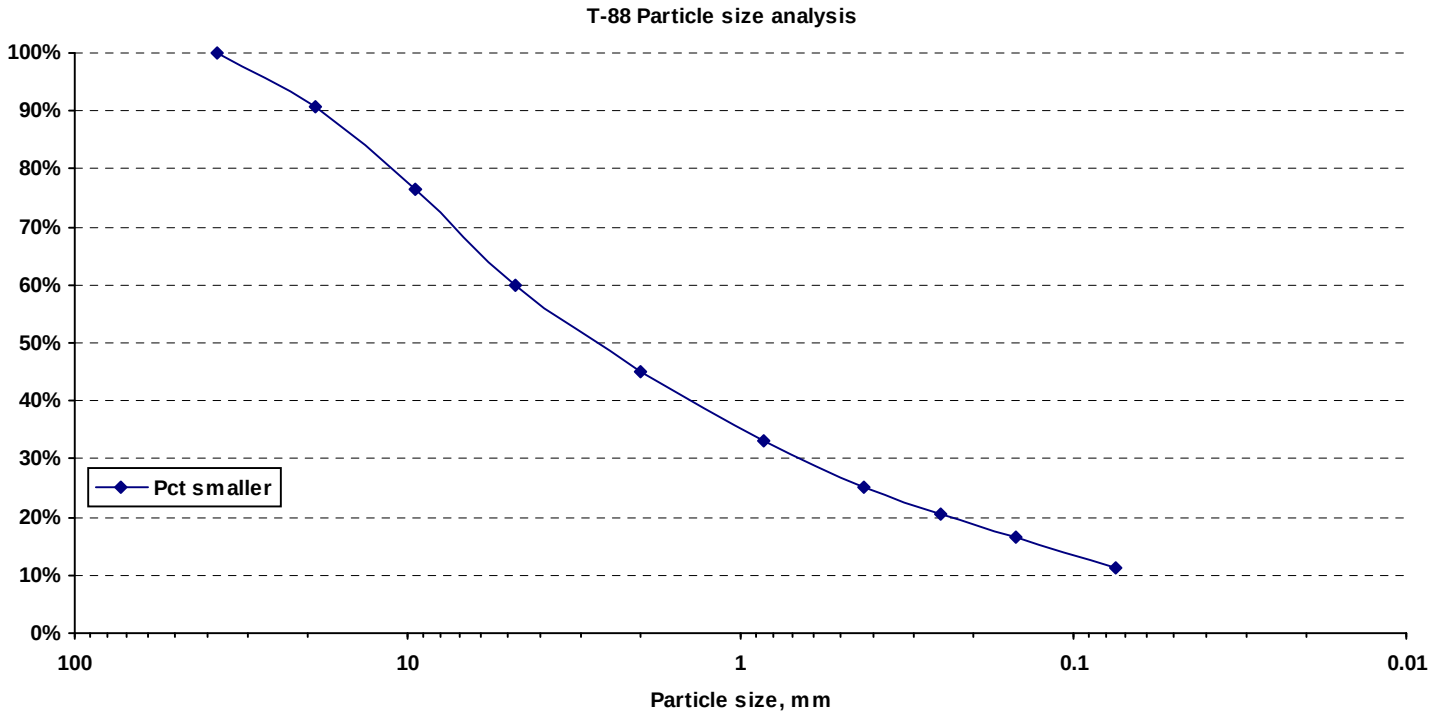
Construction and Materials Bureau
Central Laboratory

Report on Soil Sample

Lab number: E151005 **Corrected copy:** N/A **Report Date:** 7/7/2015 2:24:28 P

Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR- 1&2

Hole: B-103 **Depth:** 0.5 FT - 2 FT



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Distribution list
CHA
T. ELIASSEN
J. TOUCHETTE

Report on Soil Sample

Lab number: E151006 **Corrected copy:** N/A **Report Date:** 7/7/2015 2:22:36 P
Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR- 1&2

Date sampled: 6/16/2015 **Received:** 7/6/2015 **Tested:** 7/6/2015 **Tested by:** J. TOUCHETTE
Station: **Offset:** **Hole:** B-103 **Depth:** 9 FT **to:** 10.5 FT
Field description: Weathered Rock/ Si & Sa
Submitted by: CHA **Address:**
Sample type: SPLIT BARREL **Quantity:**
Sample source/Outside agency name:
Location used: **Examined for:** MC, GS
Comment: S-3

Test Results

Sieve Analysis

T-88	% Passing Total Sample
75 mm (3.0"):	
37.5 mm (1.5"):	
19 mm (3/4"):	89.4%
9.5 mm (3/8"):	78.1%
4.75 mm (#4):	58.8%
2.00 mm (#10):	46.9%
850 µm (#20):	37.0%
425 µm (#40):	31.4%
250 µm (#60):	27.4%
150 µm (#100):	23.3%
75 µm (#200):	17.7%

Hydrometer Analysis

Particles smaller	% total sample
0.05 mm:	
0.02 mm:	
0.005 mm:	
0.002 mm:	
0.001 mm:	

Limits

T-265 Moisture content: 6.7%
T-89 Liquid Limit:
T-90 Plastic Limit:
T-90 Plasticity Index: NP

Moisture Density

Test method: T-180 **Method:**
Maximum density: **pcf**
Optimum moisture:
T-100 Specific Gravity:

Gr: 53.1% **D2487:** SM
Sa: 29.3% **M145:** A-1-b Sandy Gravel
Si: 17.7%

Comments: LAB NOTE: SAMPLE WAS MOSTLY BROKEN ROCK WITH SOME WEATHERED ROCK.

Reviewed by: T. Eliassen, P.G., Transportation Geologist

CEE
For TDE

Distribution list

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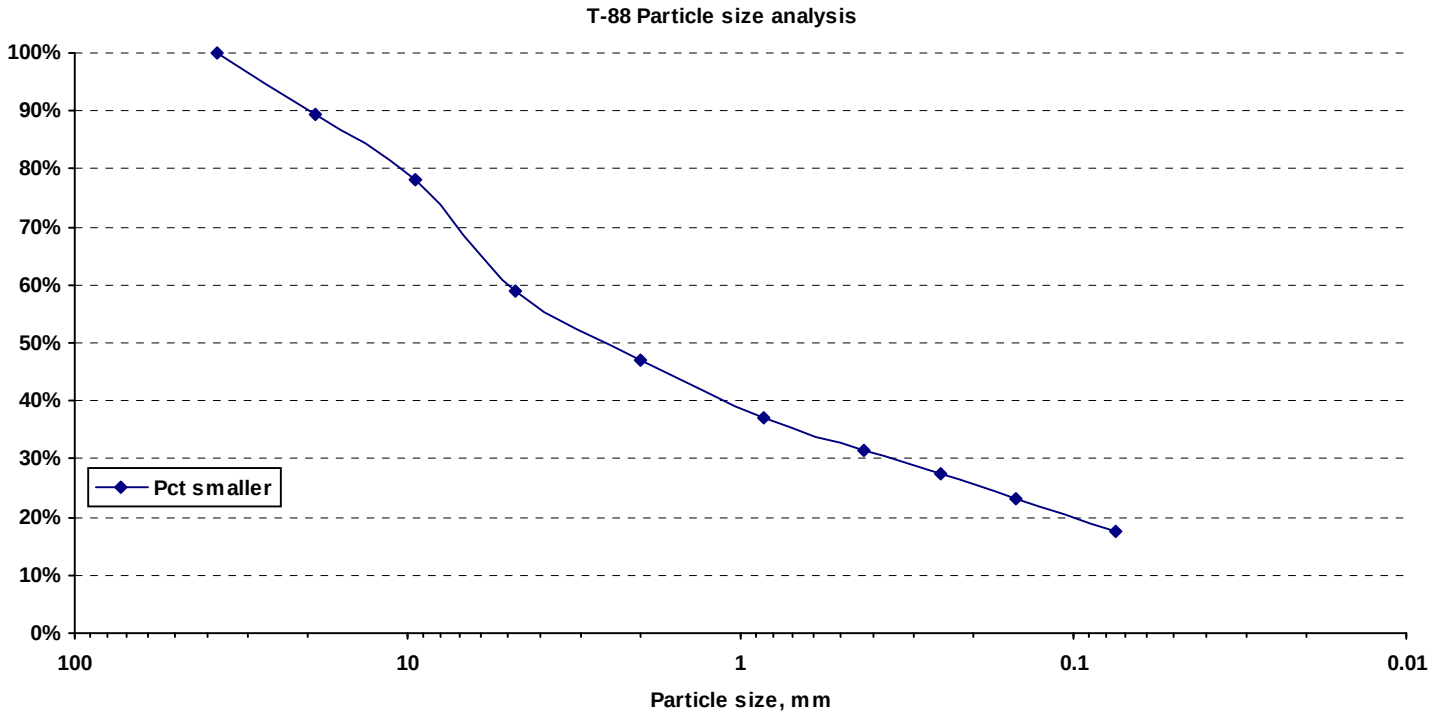
Report on Soil Sample

Lab number: E151006 **Corrected copy:** N/A **Report Date:** 7/7/2015 2:24:28 P

Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR- 1&2

Hole: B-103 **Depth:** 9 FT - 10.5 FT



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Distribution list
CHA
T. ELIASSEN
J. TOUCHETTE

Report on Soil Sample

Lab number: E150994 **Corrected copy:** N/A **Report Date:** 6/30/2015 9:17:46 A
Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR- 1&2

Date sampled: 6/17/2015 **Received:** 6/29/2015 **Tested:** 6/29/2015 **Tested by:** J. TOUCHETTE
Station: **Offset:** **Hole:** B-201 **Depth:** 4 FT **to:** 6 FT
Field description: Sa Gr
Submitted by: CHA **Address:**
Sample type: SPLIT BARREL **Quantity:**
Sample source/Outside agency name:
Location used: **Examined for:** MC, GS
Comment: S-2

Test Results

Sieve Analysis

T-88	% Passing Total Sample
75 mm (3.0"):	
37.5 mm (1.5"):	
19 mm (3/4"):	
9.5 mm (3/8"):	
4.75 mm (#4):	100.0%
2.00 mm (#10):	97.9%
850 µm (#20):	90.2%
425 µm (#40):	63.1%
250 µm (#60):	33.2%
150 µm (#100):	17.6%
75 µm (#200):	7.4%

Hydrometer Analysis

Particles smaller	% total sample
0.05 mm:	
0.02 mm:	
0.005 mm:	
0.002 mm:	
0.001 mm:	

Limits

T-265 Moisture content: 5.9%
T-89 Liquid Limit:
T-90 Plastic Limit:
T-90 Plasticity Index: NP

Moisture Density

Test method: T-180 **Method:**
Maximum density: **pcf**
Optimum moisture:
T-100 Specific Gravity:

Gr: 2.1% **D2487:** SP-SM
Sa: 90.4% **M145:** A-3 Sand
Si: 7.4%

Comments:

Reviewed by: T. Eliassen, P.G., Transportation Geologist TDE

Distribution list

State of Vermont
Agency of Transportation
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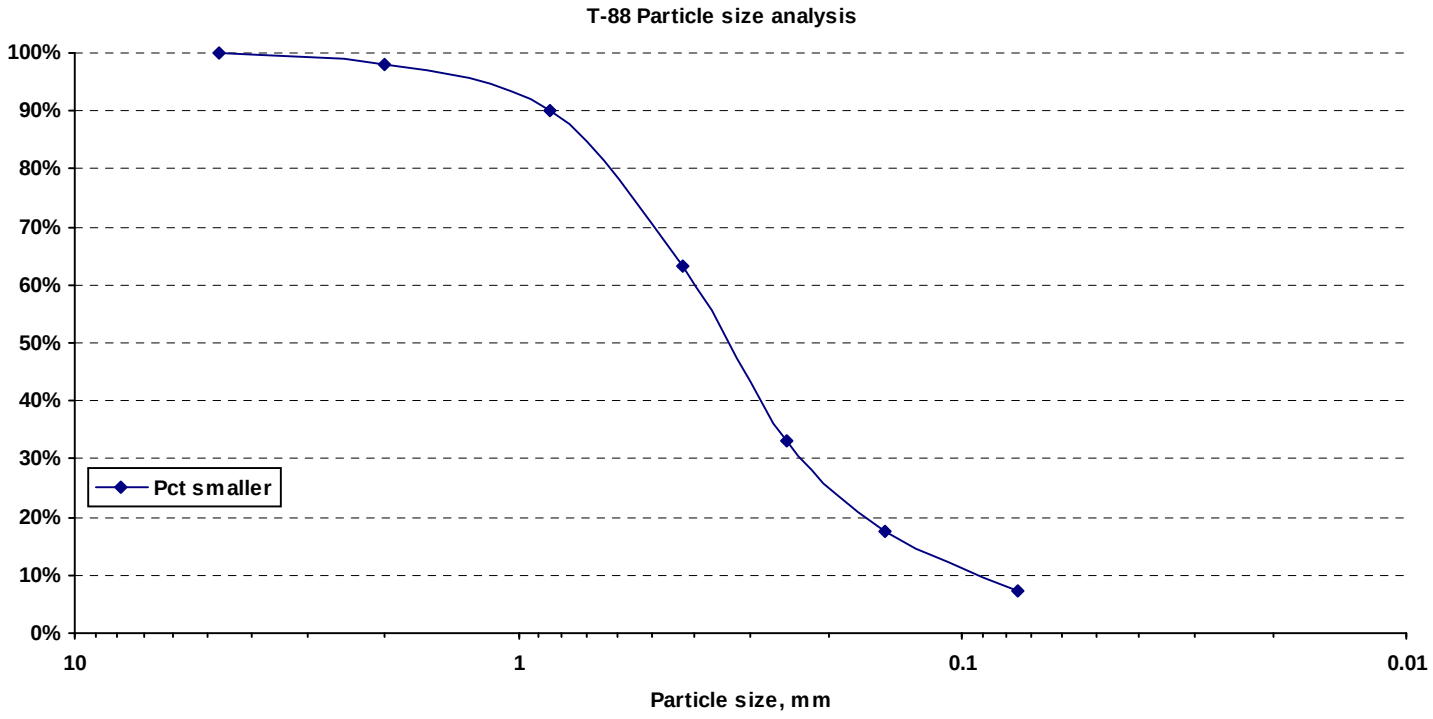
Report on Soil Sample

Lab number: E150994 **Corrected copy:** N/A **Report Date:** 6/30/2015 9:19:17 A

Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR- 1&2

Hole: B-201 **Depth:** 4 FT - 6 FT



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Central Laboratory

Distribution list
CHA
T. ELIASSEN
J. TOUCHETTE

Report on Soil Sample

Lab number: E150995 **Corrected copy:** N/A **Report Date:** 6/30/2015 9:17:47 A
Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR- 1&2

Date sampled: 6/17/2015 **Received:** 6/29/2015 **Tested:** 6/29/2015 **Tested by:** J. TOUCHETTE
Station: **Offset:** **Hole:** B-201 **Depth:** 19.5 FT **to:** 21 FT
Field description: Weathered Rock (Si & Sa)
Submitted by: CHA **Address:**
Sample type: SPLIT BARREL **Quantity:**
Sample source/Outside agency name:
Location used: **Examined for:** MC, GS
Comment: S-7

Test Results

Sieve Analysis

T-88	% Passing Total Sample
75 mm (3.0"):	
37.5 mm (1.5"):	
19 mm (3/4"):	
9.5 mm (3/8"):	95.6%
4.75 mm (#4):	88.3%
2.00 mm (#10):	81.1%
850 µm (#20):	75.2%
425 µm (#40):	70.6%
250 µm (#60):	66.6%
150 µm (#100):	62.0%
75 µm (#200):	51.7%

Hydrometer Analysis

Particles smaller	% total sample
0.05 mm:	
0.02 mm:	
0.005 mm:	
0.002 mm:	
0.001 mm:	

Limits

T-265 Moisture content: 14.0%
T-89 Liquid Limit:
T-90 Plastic Limit:
T-90 Plasticity Index: NP

Moisture Density

Test method: T-180 **Method:**
Maximum density: **pcf**
Optimum moisture:
T-100 Specific Gravity:

Gr: 18.9% **D2487:** ML
Sa: 29.4% **M145:** A-4 Sandy Silt
Si: 51.7%

Comments:

Reviewed by: T. Eliassen, P.G., Transportation Geologist TDE

Distribution list

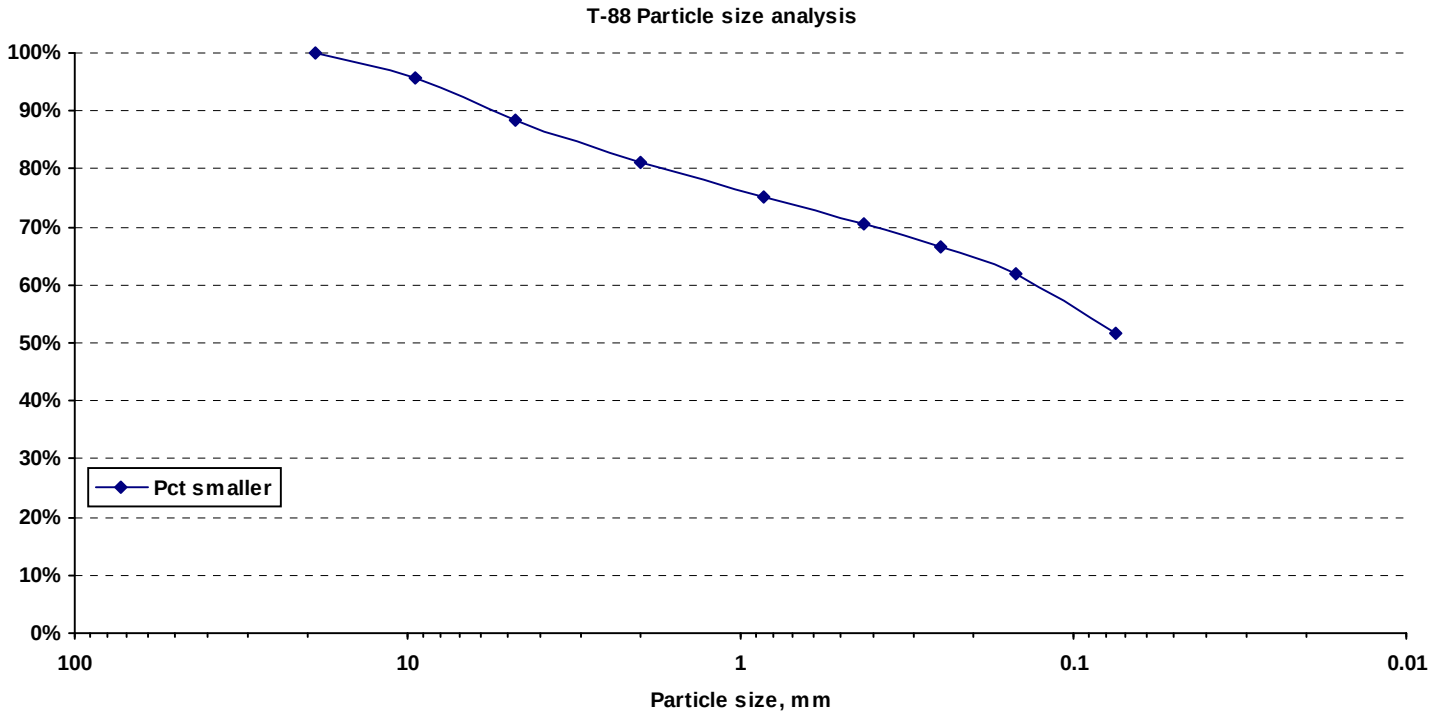
State of Vermont
Agency of Transportation
Construction and Materials Bureau
Central Laboratory

Report on Soil Sample

Lab number: E150995 **Corrected copy:** N/A **Report Date:** 6/30/2015 9:19:17 A

Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR- 1&2

Hole: B-201 **Depth:** 19.5 FT - 21 FT



State of Vermont
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Central Laboratory

Distribution list
CHA
T. ELIASSEN
J. TOUCHETTE

Report on Soil Sample

Lab number: E150996 **Corrected copy:** N/A **Report Date:** 6/30/2015 9:20:23 A
Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR- 1&2

Date sampled: 6/17/2015 **Received:** 6/29/2015 **Tested:** 6/29/2015 **Tested by:** J. TOUCHETTE
Station: **Offset:** **Hole:** B-202 **Depth:** 15 FT **to:** 16 FT
Field description: Sa & Gr
Submitted by: CHA **Address:**
Sample type: SPLIT BARREL **Quantity:**
Sample source/Outside agency name:
Location used: **Examined for:** MC, GS
Comment: S-5

Test Results

Sieve Analysis

T-88	% Passing Total Sample
75 mm (3.0"):	
37.5 mm (1.5"):	
19 mm (3/4"):	
9.5 mm (3/8"):	98.0%
4.75 mm (#4):	96.8%
2.00 mm (#10):	92.8%
850 µm (#20):	82.8%
425 µm (#40):	51.6%
250 µm (#60):	23.0%
150 µm (#100):	11.0%
75 µm (#200):	6.1%

Hydrometer Analysis

Particles smaller	% total sample
0.05 mm:	
0.02 mm:	
0.005 mm:	
0.002 mm:	
0.001 mm:	

Limits

T-265 Moisture content: 18.4%
T-89 Liquid Limit:
T-90 Plastic Limit:
T-90 Plasticity Index: NP

Moisture Density

Test method: T-180 **Method:**
Maximum density: **pcf**
Optimum moisture:
T-100 Specific Gravity:

Gr: 7.2% **D2487:** SP-SM
Sa: 86.7% **M145:** A-3 Sand
Si: 6.1%

Comments:

Reviewed by: T. Eliassen, P.G., Transportation Geologist TDE

Distribution list

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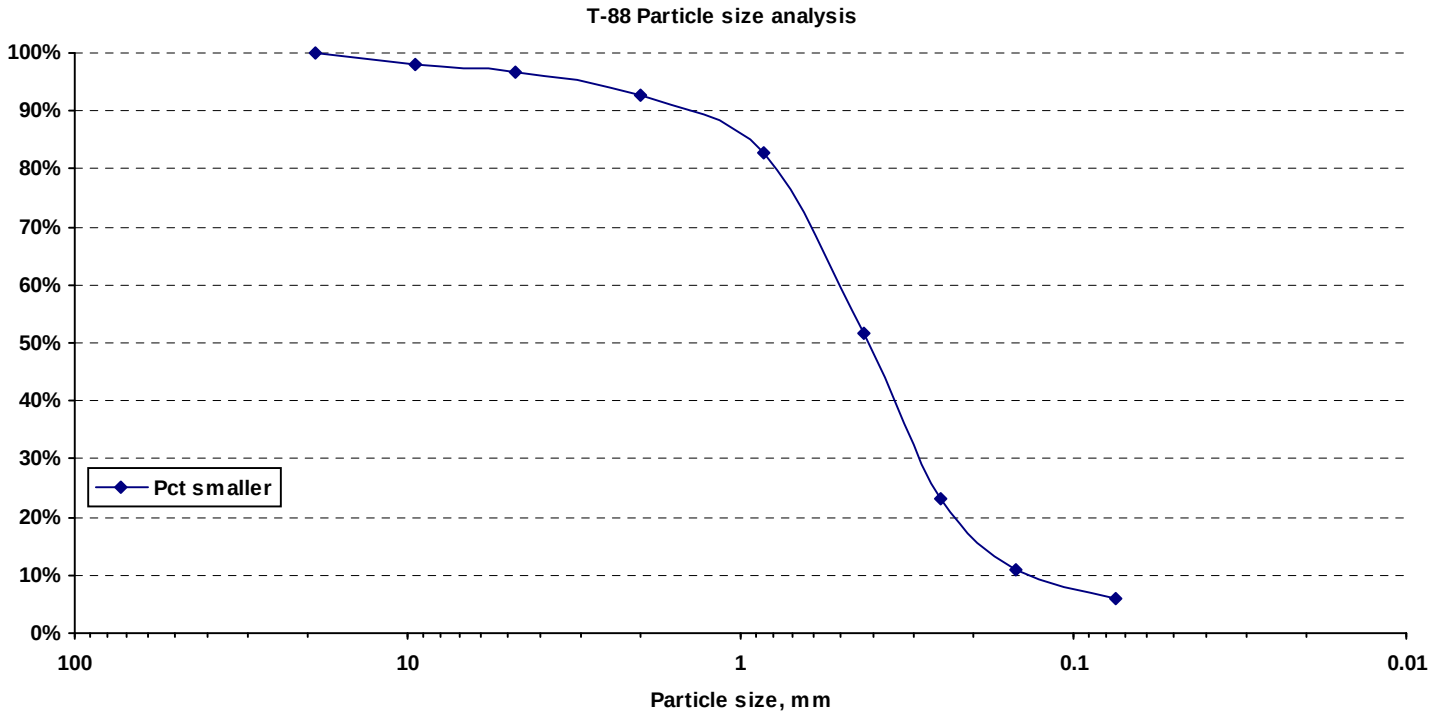
Central Laboratory

Report on Soil Sample

Lab number: E150996 **Corrected copy:** N/A **Report Date:** 6/30/2015 9:21:16 A

Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR- 1&2

Hole: B-202 **Depth:** 15 FT - 16 FT



State of Vermont
Agency of Transportation
Construction and Materials Bureau
Central Laboratory

Distribution list
CHA
T. ELIASSEN
J. TOUCHETTE

Report on Soil Sample

Lab number: E150997 **Corrected copy:** N/A **Report Date:** 6/30/2015 9:20:23 A
Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR- 1&2

Date sampled: 6/17/2015 **Received:** 6/29/2015 **Tested:** 6/29/2015 **Tested by:** J. TOUCHETTE
Station: **Offset:** **Hole:** B-202 **Depth:** 19 FT **to:** 20.5 FT
Field description: CWR (Si & Sa)
Submitted by: CHA **Address:**
Sample type: SPLIT BARREL **Quantity:**
Sample source/Outside agency name:
Location used: **Examined for:** MC, GS
Comment: S-6

Test Results

Sieve Analysis

T-88	% Passing Total Sample
75 mm (3.0"):	
37.5 mm (1.5"):	
19 mm (3/4"):	
9.5 mm (3/8"):	97.8%
4.75 mm (#4):	92.8%
2.00 mm (#10):	88.3%
850 µm (#20):	84.2%
425 µm (#40):	80.5%
250 µm (#60):	76.3%
150 µm (#100):	70.6%
75 µm (#200):	57.0%

Hydrometer Analysis

Particles smaller	% total sample
0.05 mm:	
0.02 mm:	
0.005 mm:	
0.002 mm:	
0.001 mm:	

Limits

T-265 Moisture content: 9.1%
T-89 Liquid Limit:
T-90 Plastic Limit:
T-90 Plasticity Index: NP

Moisture Density

Test method: T-180 **Method:**
Maximum density: **pcf**
Optimum moisture:
T-100 Specific Gravity:

Gr: 11.7% **D2487:** ML
Sa: 31.3% **M145:** A-4 Sandy Silt
Si: 57.0%

Comments:

Reviewed by: T. Eliassen, P.G., Transportation Geologist TDE

Distribution list

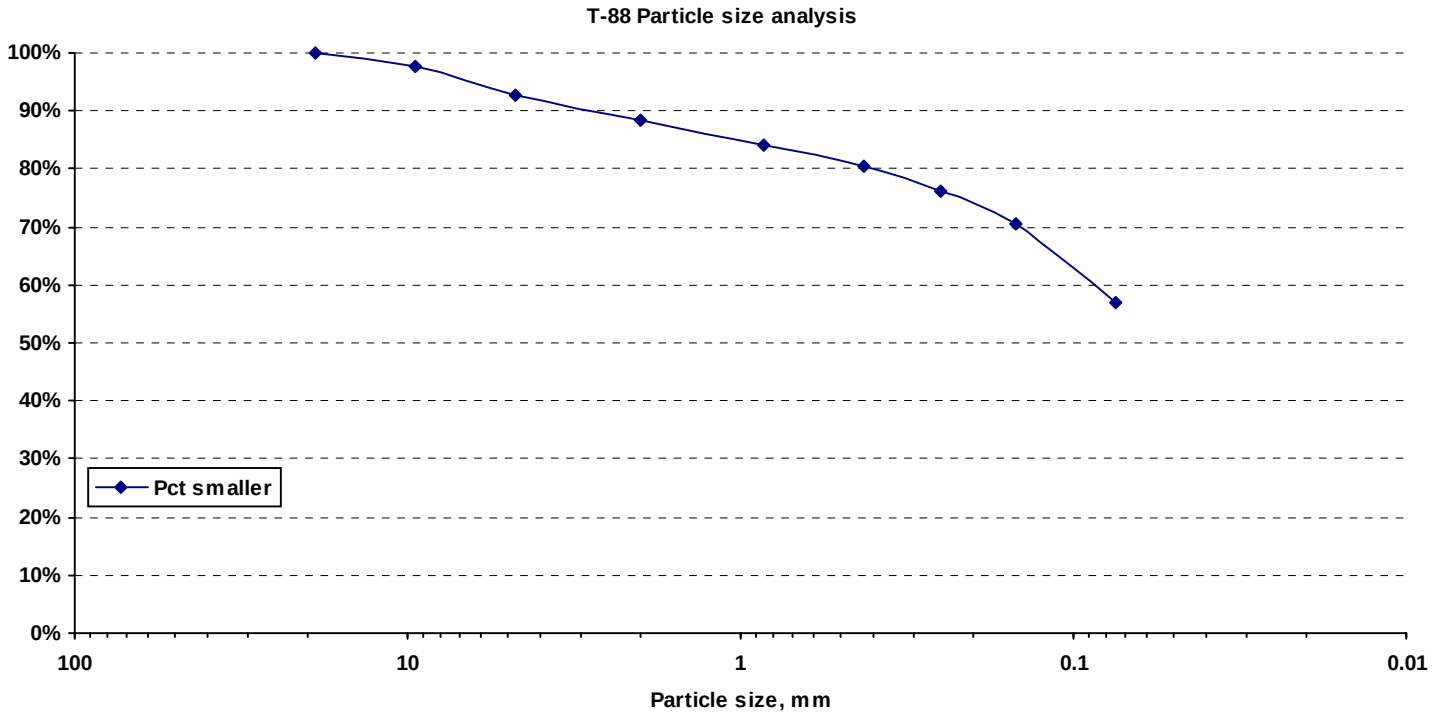
State of Vermont
Agency of Transportation
Construction and Materials Bureau
Central Laboratory

Report on Soil Sample

Lab number: E150997 **Corrected copy:** N/A **Report Date:** 6/30/2015 9:21:16 A

Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR- 1&2

Hole: B-202 **Depth:** 19 FT - 20.5 FT



State of Vermont
Agency of Transportation
Construction and Materials Bureau
Central Laboratory

Distribution list
CHA
T. ELIASSEN
J. TOUCHETTE

Report on Soil Sample

Lab number: E150998 **Corrected copy:** N/A **Report Date:** 6/30/2015 9:22:13 A
Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR- 1&2

Date sampled: 6/17/2015 **Received:** 6/29/2015 **Tested:** 6/29/2015 **Tested by:** J. TOUCHETTE
Station: **Offset:** **Hole:** B-203 **Depth:** 4 FT **to:** 6 FT
Field description: Sa & Gr
Submitted by: CHA **Address:**
Sample type: SPLIT BARREL **Quantity:**
Sample source/Outside agency name:
Location used: **Examined for:** MC, GS
Comment: S-2

Test Results

Sieve Analysis

T-88	% Passing Total Sample
75 mm (3.0"):	
37.5 mm (1.5"):	
19 mm (3/4"):	89.4%
9.5 mm (3/8"):	83.7%
4.75 mm (#4):	71.4%
2.00 mm (#10):	56.9%
850 µm (#20):	42.3%
425 µm (#40):	32.1%
250 µm (#60):	24.9%
150 µm (#100):	18.8%
75 µm (#200):	12.1%

Hydrometer Analysis

Particles smaller	% total sample
0.05 mm:	
0.02 mm:	
0.005 mm:	
0.002 mm:	
0.001 mm:	

Limits

T-265 Moisture content: 11.2%
T-89 Liquid Limit:
T-90 Plastic Limit:
T-90 Plasticity Index: NP

Moisture Density

Test method: T-180 **Method:**
Maximum density: **pcf**
Optimum moisture:
T-100 Specific Gravity:

Gr: 43.1% **D2487:** SM
Sa: 44.8% **M145:** A-1-b Gravelly Sand
Si: 12.1%

Comments: LAB NOTE: BROKEN ROCK WAS WITHIN SAMPLE.

Reviewed by: T. Eliassen, P.G., Transportation Geologist TDE

Distribution list

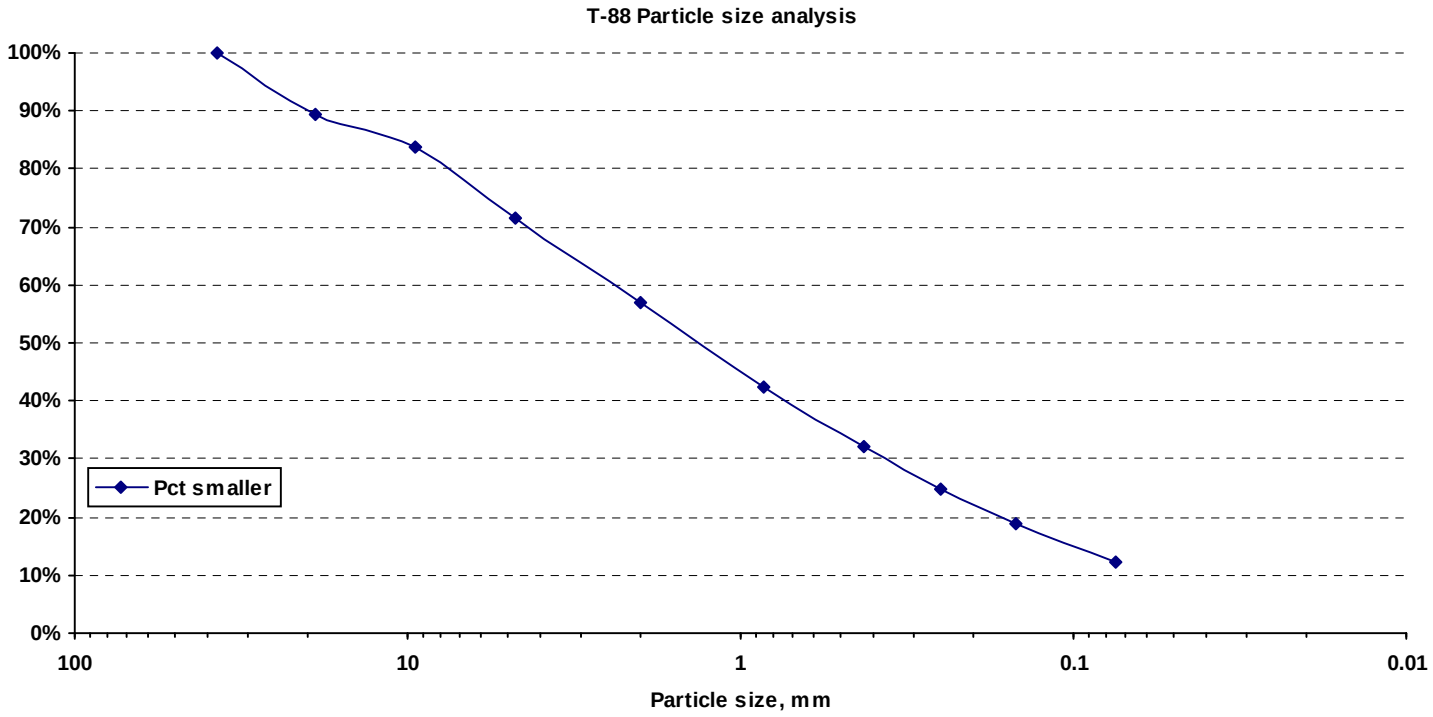
State of Vermont
Agency of Transportation
Construction and Materials Bureau
Central Laboratory

Report on Soil Sample

Lab number: E150998 **Corrected copy:** N/A **Report Date:** 6/30/2015 9:22:51 A

Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR- 1&2

Hole: B-203 **Depth:** 4 FT - 6 FT



State of Vermont
Agency of Transportation
Construction and Materials Bureau
Central Laboratory

Distribution list
CHA
T. ELIASSEN
J. TOUCHETTE

Report on Soil Sample

Lab number: E150999 **Corrected copy:** N/A **Report Date:** 6/30/2015 9:22:13 A
Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR- 1&2

Date sampled: 6/17/2015 **Received:** 6/29/2015 **Tested:** 6/29/2015 **Tested by:** J. TOUCHETTE
Station: **Offset:** **Hole:** B-203 **Depth:** 19 FT **to:** 21 FT
Field description: CWR (Si & Sa)
Submitted by: CHA **Address:**
Sample type: SPLIT BARREL **Quantity:**
Sample source/Outside agency name:
Location used: **Examined for:** MC, GS
Comment: S-5

Test Results

Sieve Analysis

T-88	% Passing Total Sample
75 mm (3.0"):	
37.5 mm (1.5"):	
19 mm (3/4"):	90.1%
9.5 mm (3/8"):	83.5%
4.75 mm (#4):	77.4%
2.00 mm (#10):	69.1%
850 µm (#20):	62.6%
425 µm (#40):	57.9%
250 µm (#60):	53.5%
150 µm (#100):	48.6%
75 µm (#200):	38.4%

Hydrometer Analysis

Particles smaller	% total sample
0.05 mm:	
0.02 mm:	
0.005 mm:	
0.002 mm:	
0.001 mm:	

Limits

T-265 Moisture content: 14.3%
T-89 Liquid Limit:
T-90 Plastic Limit:
T-90 Plasticity Index: NP

Moisture Density

Test method: T-180 **Method:**
Maximum density: **pcf**
Optimum moisture:
T-100 Specific Gravity:

Gr: 30.9% **D2487:** SM
Sa: 30.7% **M145:** A-4 Sandy Gravelly Silt
Si: 38.4%

Comments: LAB NOTE: SOME WOOD FIBERS WERE WITHIN SAMPLE.

Reviewed by: T. Eliassen, P.G., Transportation Geologist TDE

Distribution list

State of Vermont
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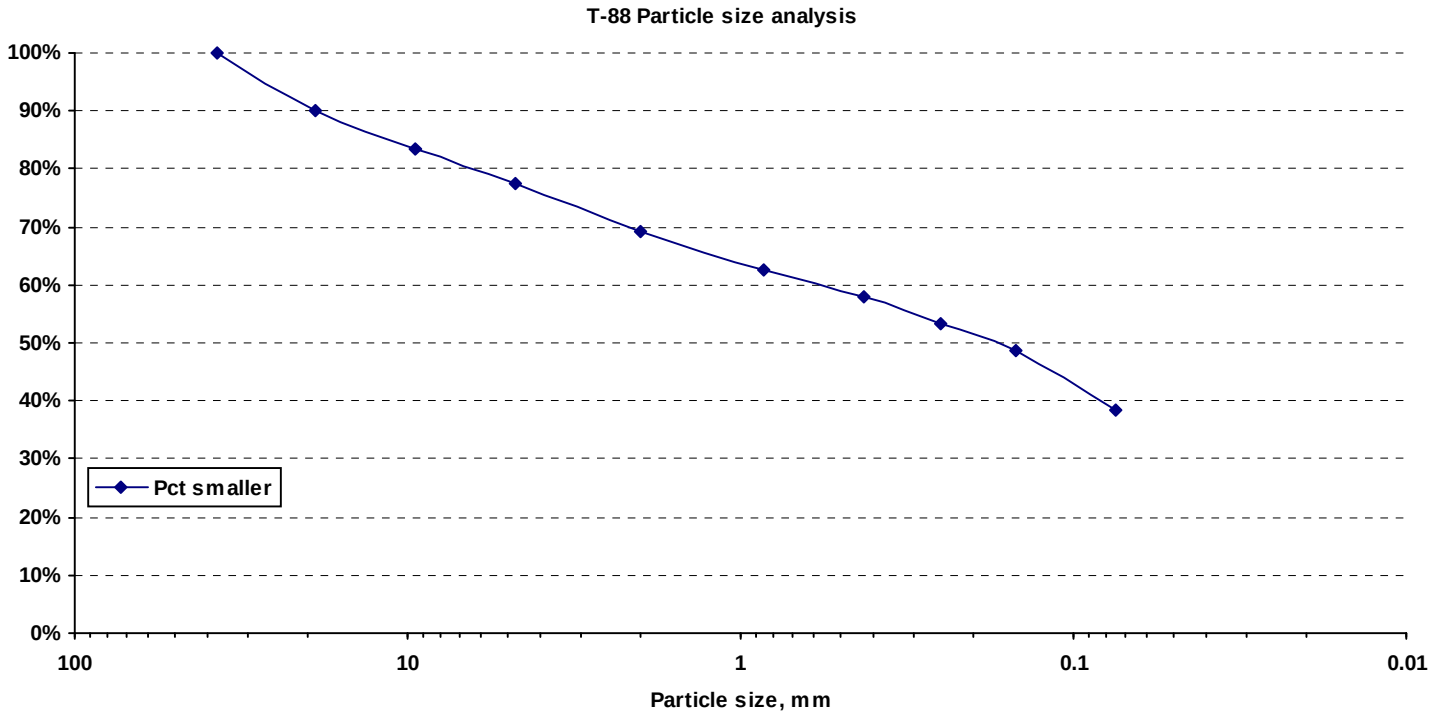
Central Laboratory

Report on Soil Sample

Lab number: E150999 **Corrected copy:** N/A **Report Date:** 6/30/2015 9:22:51 A

Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR- 1&2

Hole: B-203 **Depth:** 19 FT - 21 FT



State of Vermont
Agency of Transportation
Construction and Materials Bureau
Central Laboratory

Distribution list
CHA
T. ELIASSEN
J. TOUCHETTE

Report on Soil Sample

Lab number: E151000 **Corrected copy:** N/A **Report Date:** 6/30/2015 9:23:39 A
Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR- 1&2

Date sampled: 6/18/2015 **Received:** 6/29/2015 **Tested:** 6/29/2015 **Tested by:** J. TOUCHETTE
Station: **Offset:** **Hole:** B-204 **Depth:** 4 FT **to:** 6 FT
Field description: Sa & Gr
Submitted by: CHA **Address:**
Sample type: SPLIT BARREL **Quantity:**
Sample source/Outside agency name:
Location used: **Examined for:** MC, GS
Comment: S-2

Test Results

Sieve Analysis

T-88	% Passing Total Sample
75 mm (3.0"):	
37.5 mm (1.5"):	
19 mm (3/4"):	
9.5 mm (3/8"):	
4.75 mm (#4):	99.2%
2.00 mm (#10):	97.6%
850 µm (#20):	92.0%
425 µm (#40):	64.3%
250 µm (#60):	32.9%
150 µm (#100):	16.8%
75 µm (#200):	7.4%

Hydrometer Analysis

Particles smaller	% total sample
0.05 mm:	
0.02 mm:	
0.005 mm:	
0.002 mm:	
0.001 mm:	

Limits

T-265 Moisture content: 17.4%
T-89 Liquid Limit:
T-90 Plastic Limit:
T-90 Plasticity Index: NP

Moisture Density

Test method: T-180 **Method:**
Maximum density: **pcf**
Optimum moisture:
T-100 Specific Gravity:

Gr: 2.4% **D2487:** SP-SM
Sa: 90.2% **M145:** A-3 Sand
Si: 7.4%

Comments:

Reviewed by: T. Eliassen, P.G., Transportation Geologist TDE

Distribution list

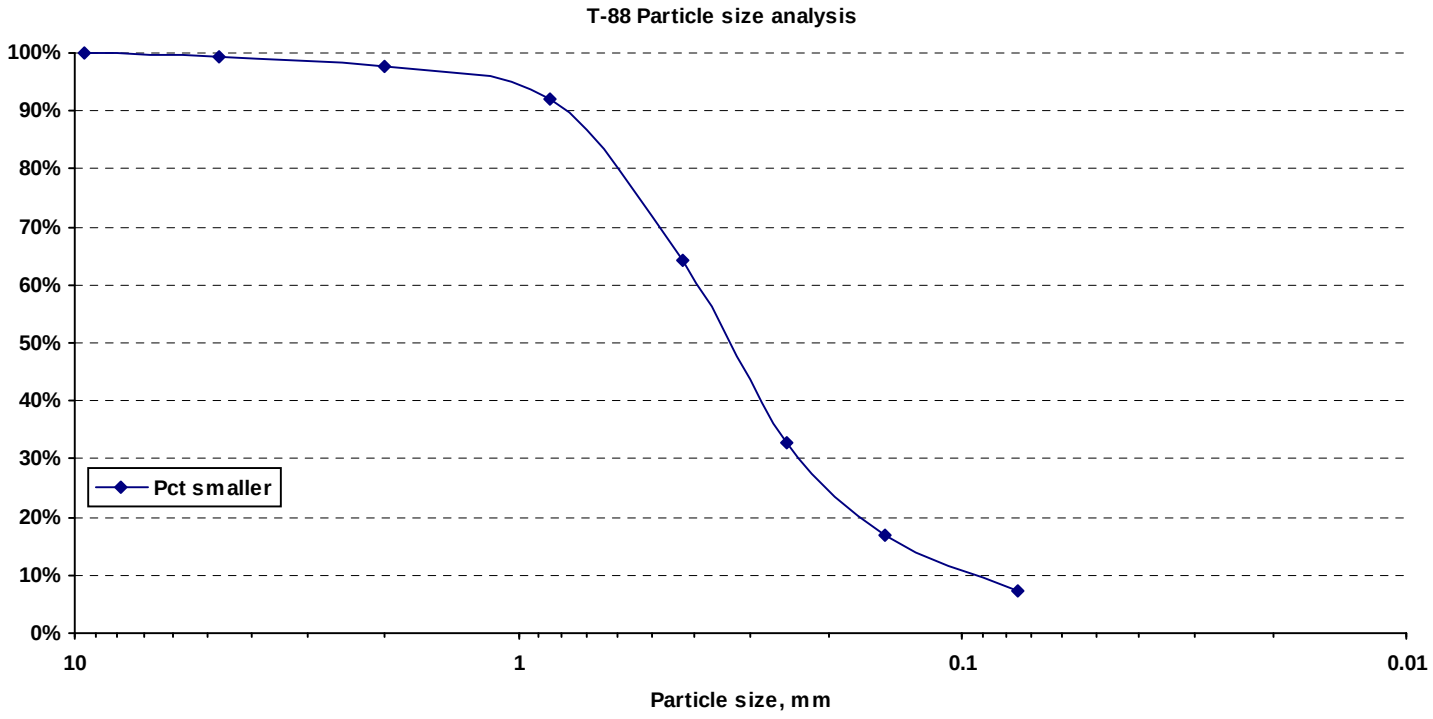
State of Vermont
Agency of Transportation
Construction and Materials Bureau
Central Laboratory

Report on Soil Sample

Lab number: E151000 **Corrected copy:** N/A **Report Date:** 6/30/2015 9:24:18 A

Project: JOHNSON **Number:** BF 0248(4) **Site:** VT-100 BR- 1&2

Hole: B-204 **Depth:** 4 FT - 6 FT



VTRANS SOIL LABORATORY TEST REQUEST FORM

PROJECT Johnson BF 0248(4) PROJECT NUMBER _____ HOLE NO. B103 B201 B202 B203 B204

DATE _____ REQUEST BY _____ APPROVED BY _____ STATION NO. _____ OFFSET _____

REQUESTOR CONTACT INFO Kelly Owens 860-257-4557 ORIGINAL REQUEST ☐ SUPPLEMENTAL REQUEST ☒

	SAMPLE DATA			INDEX TESTS					CONSOL.		STRENGTH TESTS								
	SAMPLE NUMBER/ID	SAMPLE DEPTH (ft)	SAMPLE TYPE (Auger, SS, Tube, Hand)	MOISTURE CONTENT (T 265)	GRADATION (T 88)	ATTERBURG LIMITS (T89/90)	HYDROMETER (T 88)	ORGANICS (T 267)	ONE-DIM CONSOL. (T 216)	NUMBER OF TESTS	TYPE OF TEST AND CONSOLIDATION PRESSURE								*STATUS
											U (T 208)	UU (T 296)	CONF. PRESS (PSI)	CU (T 297)	DS (T 236)	CONS. PRESS. (PSI) 1	2	3	
*B201	S-2	46	SS		X														
B201	S-7	19.5-21	↓	X	X														
B202	S-5	15-16			X														
B202	S-6	19-20.5		X	X														
B203	S-2	4-6			X														
B203	S-5	19-21		X	X														
B204	S-2	4-6	↓		X														
B103	R-3	19-19.1	NX Core																
B103	R-4	25.7-26.6'	↓		UNIAXIAL COMPRESSIVE TESTS														
B201	R-2	24-24.9'																	
B203	R-3	31.5-32.2'																	
B204	R-4	18.5-19.1	↓																
Bridge 1	C-1	0-6.25"	Core		UNIAXIAL COMPRESSIVE														
Bridge 2	C-2	0-6.25"	↓																

Per T. Eliassen 7/2/15
2 tests each:
Bridge 1 - 6130psi & 5600psi (5600 likely no good because of testing error)
Bridge 2 - 6170psi & 6400psi

*CHECK MARK WHEN TEST IS IN PROGRESS ✓ MAKE X WHEN COMPLETED ✕ CIRCLE WHEN COMPUTED ●

REV. NOV. 16, 2010

DeFlorio, Nicholas

From: Tillberg, David <David.Tillberg@vermont.gov>
Sent: Thursday, November 19, 2015 4:08 PM
To: Wild, Jim; DeFlorio, Nicholas
Subject: RE: Johnson BF 0248(4) Concrete Core Test Results
Attachments: 20150702114915791.pdf

Hi Nicholas,

I looked at this in our system was never entered into our system for reporting. I cannot remember the exact details of the testing, but this was request came from Geotechnical engineering Section.

Found the original card (attached). This made me realize I missed a couple of results (added tests #3 & #4 below):
I would say "corrected PSI" on the first email from Jim does not mean there was any height correction force adjustment. A height correction force adjustment should be applied as these are less than 1.75 L/d ratio. I have applied height adjustments (correction factor) and given correct psi below.

Cylinders #1 & #2 were logged in under lab number C150629 and broken on 7/2/15
Cylinders #3 & #4 were logged in under lab number C150630 and broken on 7/2/15

Cylinder #1
The diameter of the cylinder #1 was 3.64"
The length was average 5.77"
Pounds 58227, uncorrected psi 5595
Correction factor = 0.967, corrected psi = 5410

Cylinders #2
The diameter of the cylinder #1 was 3.64"
The length was average 6.14"
Pounds 63793, uncorrected psi 6130
Correction factor = 0.975, corrected psi = 5980

Cylinder #3
The diameter of the cylinder #3 was 3.60"
The length was average 5.23"
Pounds 62823, uncorrected psi 6170
Correction factor = 0.954, corrected psi = 5890

Cylinder #4
The diameter of the cylinder #4 was 3.71"
The length was average 5.76"
Pounds 69146, uncorrected psi 6400
Correction factor = 0.964, corrected psi = 6170

I think I just did the testing and gave results (card) to them, that's why it is not in the system. This kind of thing happens infrequently.

TJ Davison might be able to produce a report if we really need one. If this is the case, let us know.

APPENDIX E

NDT NONDESTRUCTIVE AND GEOPHYSICAL SURVEY REPORT

NONDESTRUCTIVE AND GEOPYSICAL SURVEY
JOHNSON BF 0248 (4) BRIDGE 1 AND BRIDGE 2
VT 100C OVER GIHON RIVER
EAST JOHNSON, VERMONT

Prepared for

Clough Harbour & Associates

JULY, 2015

NDT CORPORATION

NDT CORPORATION

153 Clinton Road • Sterling, MA 01564
Tel: 978-563-1327
Fax: 978-563-1340

July 10, 2015

Mr. Charles W. Symmes, P.E.
Clough Harbour & Associates
III Winners Circle
Albany, NY 12205

Subject: Nondestructive testing and geophysical survey to concrete conditions, subsurface and fill conditions, and abutment geometry for Johnson BF 0248 (4) Bridge 1 and Bridge 2; VT 100C over the Gihon River in East Jonson, Vermont.

Dear Mr. Symmes:

In accordance with your authorization to proceed, NDT Corporation conducted nondestructive sonic/ultrasonic measurements, ground penetrating radar (GPR) and geophysical seismic refraction measurements on and near the abutment walls of Johnson BF 0248 (4) Bridge 1 and Bridge 2; VT 100C over the Gihon River in East Jonson, Vermont. Field work was conducted on June 17th and 18th, 2015. The objectives of the survey were:

- The horizontal geometry (thickness) of the existing concrete abutments and wing walls (primary objective);
- The vertical geometry (height) of the concrete abutments (secondary objective)
- The spacing, depth of cover and number of layers of reinforcing in the concrete (secondary objective);
- The strength of the concrete (secondary objective);
- Depth and attitude of the bedrock surface at the abutments (secondary objective).

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 978-573-1327.

Sincerely

NDT Corporation

Paul S Fisk

Paul S Fisk

TABLE OF CONTENTS

1.0 INTRODUCTION AND PURPOSE

NDT Corporation conducted nondestructive sonic/ultrasonic measurements, ground penetrating radar (GPR) and geophysical seismic refraction measurements on and near the abutment walls of Johnson BF 0248 (4) Bridge 1 and Bridge 2; VT 100C over the Gihon River in East Johnson, Vermont. Field work was conducted on June 17th and 18th, 2015. The objectives of the survey were:

- The horizontal geometry (thickness) of the existing concrete abutments and wing walls (primary objective);
- The vertical geometry (height) of the concrete abutments (secondary objective)
- The spacing, depth of cover and number of layers of reinforcing in the concrete (secondary objective);
- The strength of the concrete (secondary objective);
- Depth and attitude of the bedrock surface at the abutments (secondary objective).

2.0 LOCATION AND SURVEY CONTROL

The general location of Johnson BF 0248 (4) Bridge 1 and Bridge 2; VT 100C over the Gihon River in East Jonson, Vermont is shown on Figure 1. Lateral measurements were referenced to up-stream and down-stream, and measurements from the abutment faces, while vertical measurements were referenced to the road surface and the bottom of the beams shown on Figure 2. Reference elevations were provided by CHA and are listed below:

Bridge 1:	Bridge 2:
low t/deck approx. 580.7	low t/deck approx. 580
low beam approx. 578	low beam approx. 576
stream bed at low beam approx. 566	stream bed at low beam approx. 566

The bridges are referenced as Bridge 1 (west) over the Left Branch of the Gihon River and Bridge 2 (east) over the Right Branch of the Gihon River. Abutments are referenced as begin (west) and end (east).

3.0 METHODS OF INVESTIGATION

3.1 Sonic/Ultrasonic Measurements

The integrity/strength of concrete is evaluated with sonic/ultrasonic pulse velocity measurements. Sonic/ultrasonic testing is the most definitive NDT testing technique for the assessment of concrete. Sonic/ultrasonic NDT measurements determine the characteristics of concrete by creating a stress wave generated by a relatively low energy projectile impact. Stress wave measurements in the sonic/ultrasonic frequency band are used to make direct measurements of the compressional and shear wave transmission velocity and to measure reflected phases of the compressional wave. The transmission velocity values determine the elastic deformational characteristics of the concrete, including Young's, bulk, and shear moduli, as well as Poisson's ratio, and strength values. Low velocity areas are indicative of low strength concrete, high velocity areas are indicative of higher strength concrete and locations with no or poorly developed wave arrivals are indicative of weakened concrete due to open cracking, voiding, delamination or honey combing.

The height/depth of walls, piles and shafts can be determined using sonic/ultrasonic pulse-echo measurements. The energy source produces stress waves that are detected by receiving sensors which measure the amplitude of the stress wave in time (time domain data). Compressional wave velocity values are calculated using the travel times and the distance between the impact point and sensors. Reflected compressional wave arrivals are used to determine the time required for a compressional wave to travel from the impact point to a reflective boundary and back to the receiving sensors. Data recorded at a sensor next to the impact point is used to establish "zero" time (instant energy is introduced into the pile). The two-way travel time of the impact signal is the time difference between zero time and reflected signal. The wall height/ length is determined using ½ the measured time to the reflector multiplied times the measured velocity. Data for multiple "shots" and reflectors are used to determine the average wall height/ length. Testing results using this method is expected to be within 10% of the actual height/depth. Cracked, broken or severely deteriorated zones in the structure can/will disrupt the energy propagation causing a reflection that may be interpreted as an "end" reflector.

The sonic/ultrasonic data were acquired with a system designed and built by NDT Corporation specifically for testing concrete structures. For this project the system used a projectile impact energy source and a three sensor array for recording the generated signals. The signal from the sensor is pre-amplified, amplified, conditioned and input to a portable PC for field quality control and archiving. A more detailed discussion of the sonic/ultrasonic method is in Appendix 1.

3.2 Ground Penetrating Radar (GPR)

The GPR method uses a pulsed electromagnetic signal that is transmitted to and reflected by a target back to the point of transmission. The electromagnetic wave transmission and reflection is dependent on the dielectric constant and conductivity (electrical) properties of the material(s) being investigated. These electrical properties are highly dependent on moisture content; saturated or concentrated moist conditions provide both strong reflections and high attenuation limiting signal penetration. GPR can be used to determine thickness of structures, assess fill and soil layering for settlement/voiding, and or locate utilities/rebar. A detailed discussion of the GPR Survey Method is included in Appendix 2.

GPR can be used to determine the thickness of structures like walls, and slabs. Typically there is a change in the dielectric constant and conductivity (electrical) properties of the structure and the fill/air/water behind or below the structure. This change in conductivity will cause a change in signal character or produce a strong reflective surface indicating the interface between the changes in materials. Data collected on the vertical face of the wall indicted a strong contrast between the concrete wall and the fill materials behind the structure.

GPR data were acquired using a digital system coupled with a 400 MHz antenna. The 400 MHz antenna is high resolution with an approximate depth of investigation of up to 15+/- feet. The actual depth of investigation is dependent on the material types and moisture conditions.

3.3 Seismic Refraction

Seismic refraction data was acquired with a 12 channel system with 5 and 10 foot geophone spacing and seismic energy generated approximately every 50 feet with a "seisgun" or sledge hammer. Seismic Refraction utilizes 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. Refracted compressional wave data are used to evaluate material types and thickness, profile top of bedrock, and to determine the approximate depth to layer interfaces. A more complete discussion of the seismic refraction survey method is included in Appendix 1.

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. Delayed arrivals indicate thicker overburden over the bedrock.

Overburden, with a 1,000+/- ft. /sec velocity are consistent with normally consolidated soils/sands/fill material typical of natural soils, fluvial deposits, and/or construction fill. Overburden with a 2,000 to 2,500 ft. /sec velocity range is associated 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.

Seismic compressional wave velocities in unconsolidated deposits are significantly affected by water saturation. The compressional seismic velocity values of unsaturated overburden materials such as gravels, sands, silts and unconsolidated tills generally fall in the range of 1,000 to 2,600 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 wave velocity of sound in water. This is because the seismic wave assumes the velocity of the faster medium, that of water.

Bedrock velocity ranges are determined by rock type and the degree of fracturing, bedding, joints, and weathering. Bedrock with a velocity of 12,000 +/- ft. /sec is indicative of competent bedrock that will require drilling and blasting for removal. This velocity range is typical of competent sedimentary and metamorphic rocks such as sandstones, limestones, schists, and gneisses.

Top of bedrock surface shown on the profile sections 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 in the rock surface and materials layers of 3+/- feet are not accurately profiled, particularly in shallow (less than 10 feet deep) bedrock areas.

4.0 RESULTS

The results of the nondestructive and geophysical testing program are summarized on Figures 3 and 4 for abutment geometry, and Figures 5 through 11 for abutment height, overburden and rock and preliminary CHA boring data.

Wall Geometry

Sonic/ultrasonic and GPR data were collected on horizontal and vertical lines on the four (4) abutments/wing walls, and the down-stream U-wall at the end (east) abutment of Bridge 2 in the center of the 18 inch tall concrete levels. Concrete levels were numbered from 1 to 9 for Bridge 1 and 1 to 7 for Bridge 2 – shown on Figure 2. Data indicates the Abutments, wing walls and U-wall are battered. The wing-walls also taper from the abutment towards the ends. Figure 4 shows GPR data representing typical horizontal cross-sections from the End (East) Abutment of Bridge 1 with the concrete shape highlighted with a solid blue line.

Figure 3 shows plan elevations provided to NDT by CHA with the sonic/ultrasonic and GPR results for concrete thickness (inches) shown in green at the concrete levels. On average the Begin and End abutments of Bridge 1 are 18-20 inches at the top level (below the beams) and slope back to 34-36 inches at the bottom level near the water surface. The Begin and End abutments of Bridge 2 are 25-27 inches at the top level (below the beams) and slope back to 34-36 inches at the bottom level near the water surface. The down-stream U-wall of Bridge 2 is 16-18 inches at the top level and slope back to 28-30 inches at the bottom level near the water surface.

Concrete Strength

Sonic/ultrasonic measurements measure the velocity of an energy wave as it passes through the concrete. Concrete compressional and shear wave velocity are used to calculate an average in-situ concrete strength.

Bridge 1 Begin (west) Abutment and Wing-walls

Average compressional wave velocity	= 13,000 ft. /sec
Average shear wave velocity	= 7,300 ft. /sec
Average in-situ compressional strength	= 6,500 PSI

Bridge 1 End (east) Abutment and Wing-walls

Average compressional wave velocity	= 13,700 ft. /sec
Average shear wave velocity	= 7,500 ft. /sec
Average in-situ compressional strength	= 6,500 PSI

Bridge 2 Begin (west) Abutment and Wing-walls

Average compressional wave velocity	= 13,300 ft. /sec
Average shear wave velocity	= 7,200 ft. /sec
Average in-situ compressional strength	= 5,500 PSI

Bridge 2 End (east) Abutment and Wing-walls

Average compressional wave velocity	= 13,700 ft. /sec
Average shear wave velocity	= 7,500 ft. /sec
Average in-situ compressional strength	= 6,500 PSI

Bridge 2 Down-steam U-wall

Average compressional wave velocity	= 13,700 ft. /sec
Average shear wave velocity	= 7,500 ft. /sec
Average in-situ compressional strength	= 6,500 PSI

Concrete Reinforcing Abutment and Wing-walls

The Abutments for Bridge 1 and Bridge 2 have the same basic reinforcing layout, shown as circles in Figure 4.

The front set of vertical bars (shown in open blue circles) have a 2+/- foot spacing for the top 2 to 3 feet of the abutments and a 4+/- foot spacing for the lower levels with a depth of cover of 6+/- inches from the front face. Centered at the transition from wing-wall to abutment are three (3) vertical bars (shown in as red circles) with a 2+/- foot spacing and 3+/- inch depth of cover from the front face. The front set of horizontal bars has a spacing of 4+/- feet with a depth of cover of 4+/- inches.

The back set of vertical bars (shown in solid blue circles) in the abutment have a 1.5+/- foot spacing with a depth of cover of 6+/- inches from the back face, these bars have the same slope as the back of wall. The back set of vertical bars (shown as solid orange circles) in the wing-walls have a 4+/ foot spacing with a depth of cover of 6+/- inches from the back face, these bars have the same slope as the back of wall. The back set of horizontal bars (abutment and wing walls) has a spacing of 2+/- feet with a depth of cover of 6-8+/- inches from the back face.

The back set of vertical bars in the down-stream wing wall of Bridge 2 (west) have a 2+/- foot spacing.

Concrete Reinforcing Bridge 2 Down-stream U-wall

The down-stream U-wall of Bridge 2 has does not have a front set of reinforcing, the back vertical bars have a spacing of 2+/- feet with a depth of cover of 6+/- inches from the back face, and these bars have the same slope as the back of wall. The back set of horizontal bars (abutment and wing walls) has a spacing of 2+/- feet with a depth of cover of 6-8+/- inches from the back face.

Seismic Refraction

Seismic refraction was conducted along six (6) 100 foot lines of coverage shown on Figure 5 the CHA "Boring Location Plan" with the elevation of top of competent bedrock listed in green and the preliminary CHA boring results top of rock elevations listed in blue. In general, Lines 1 through 5 (conducted at the river level) indicate 5-8 feet of overburden with a seismic velocity of 5,000 to 6,500 ft. /sec over competent bedrock with a seismic velocity of 12,000-14,000 ft. /sec.

Included in the 5 to 8 feet of overburden there are indications of materials with a seismic velocity:

5,000+/- ft. /sec would indicate water-saturated soils, sands, gravels, etc.

6,500+/- ft. /sec would indicate consolidated till or weathered rock

Variations in the rock surface and materials layers of 3+/- feet are not accurately profiled, particularly in shallow (less than 10 feet deep) bedrock areas.

Seismic data indicates the top of rock is relatively flat (elevation 556 to 560) west of Bridge 2 End (east) abutment but slopes up to the east of this abutment (elevation 575) with outcrop in the Bell Brook tributary just north-east of the end of the down-stream U-wall.

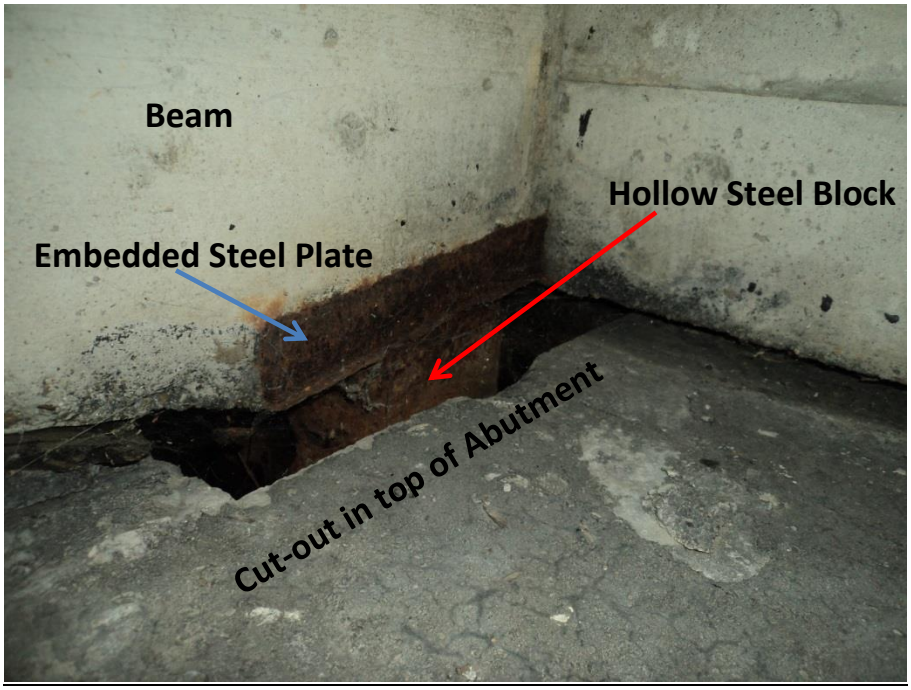
Concrete Abutment Height

Sonic/ultrasonic pulse echo measurements were conducted between the beams on the abutments and the top of the Bridge 2 down-stream U-wall. The results of these measurements are shown as red arrows on Figures 6 through 11 which also show the seismic refraction and CHA preliminary boring results.

Structure	Height	Bottom Elev.	Competent Rock Elev.
Bridge 1 Begin Abut.	16-18 feet	560-562	556+/-
Bridge 1 End Abut.	16-18 feet	560-562	558+/-
Bridge 2 Begin Abut.	16-17 feet	559-560	557+/-
Bridge 2 End Abut.	14-16 feet	560-562	560+/-
Bridge 2 U-wall	17+/- feet	562+/-	561+/-

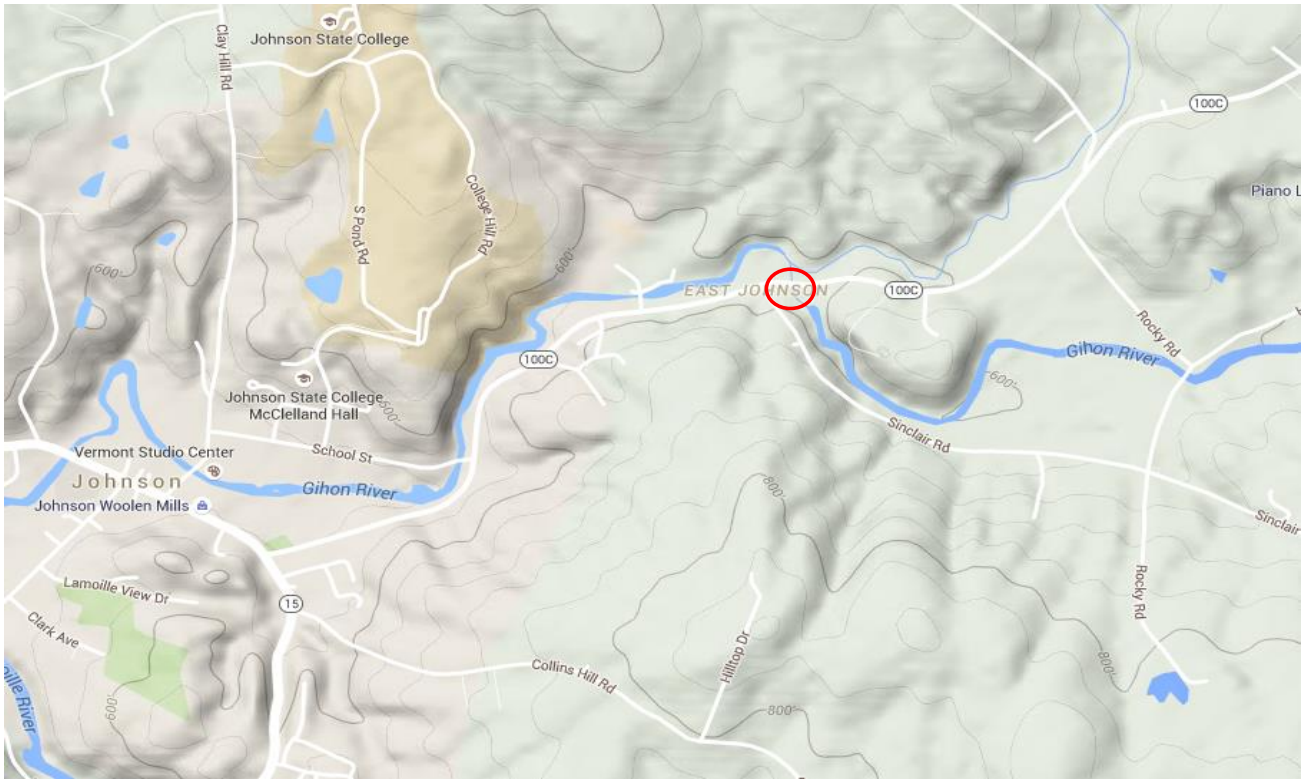
Bridge 2 End (East) Abutment Beam Support Structure

Fieldwork observations noted that the concrete beams for Bridge 1 (both abutments) and Bridge 2 Begin (west) abutments were supported on the concrete beam seat of the abutments. The concrete beams in Bridge 2 End (East) abutment were supported on a 1.5 inch thick X 8" long X full beam width steel plate embedded in the beam over rectangular steel hollow blocks approximately 10-12 inches high by 8 inches wide by 4 inches deep set in a 12 inch X 12 inch X 12 inch cut-out in the concrete abutment. These structures were discussed with CHA field engineers and are shown in the photograph below.



FIGURES

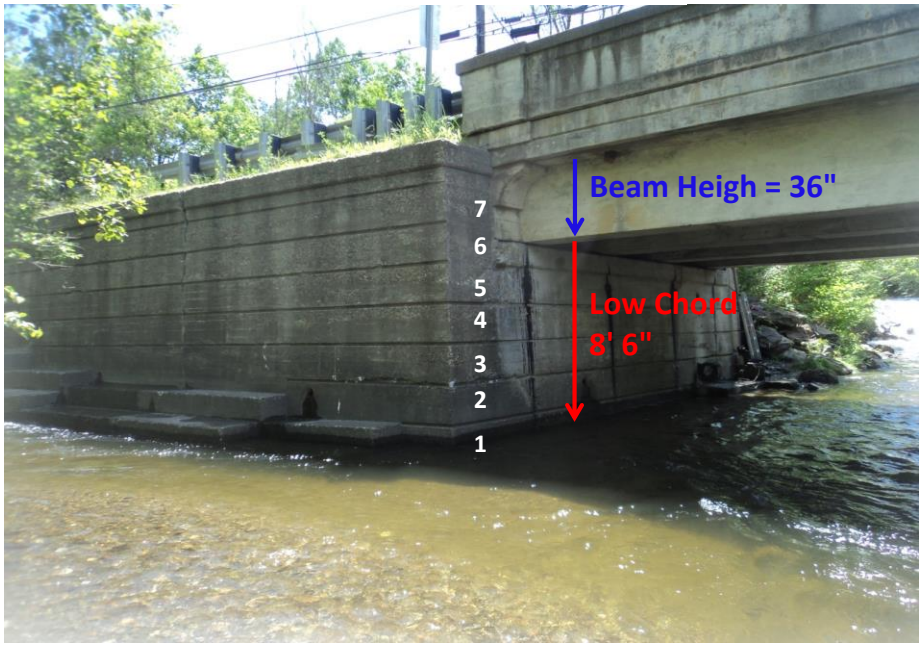
Johnson BF 0248(4)
Bridge 1 and Bridge 2
Johnson, VT
VT 100C OVER
Gihon River



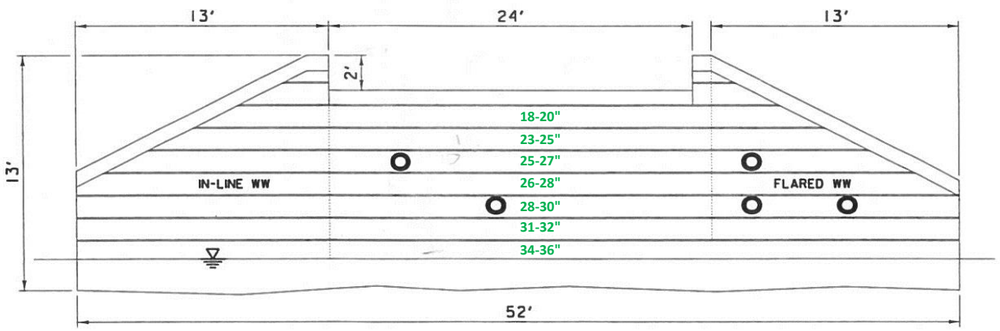
Johnson BF 0248(4) VT 100C Bridge 1 and 2 Johnson, VT prepared for CHA, Inc. by NDT Corporation	AREA OF INVESTIGATION	
	July 2015	Figure 1

Bridge 1 Begin (West) Abutment

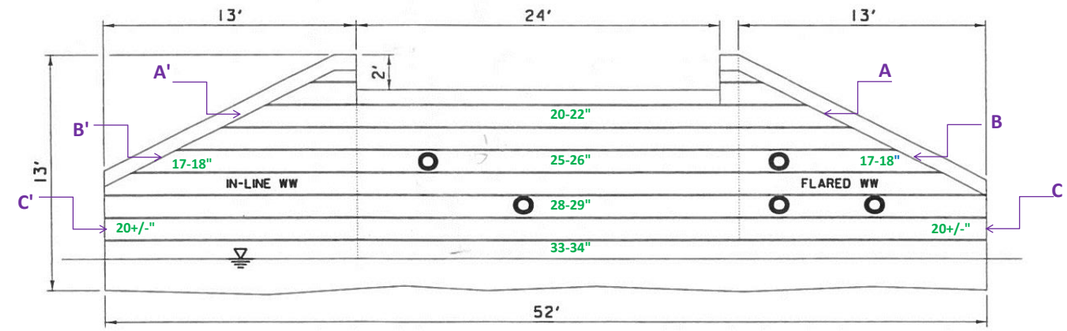
Bridge 2 End (East) Abutment



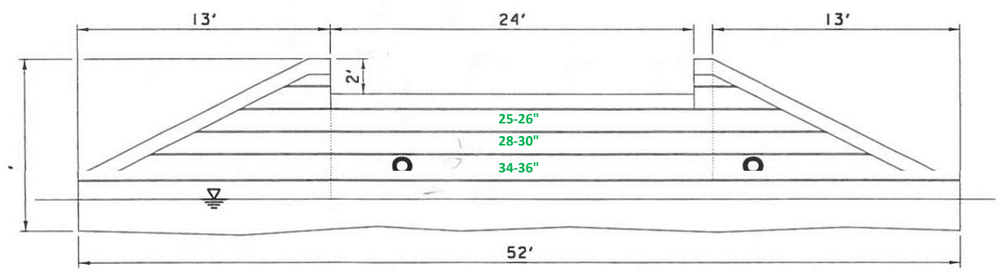
Johnson BF 0248(4) VT 100C Bridge 1 and 2 Johnson, VT prepared for CHA, Inc. by NDT Corporation	Measurement Refference Locations	
	July 2015	Figure 2



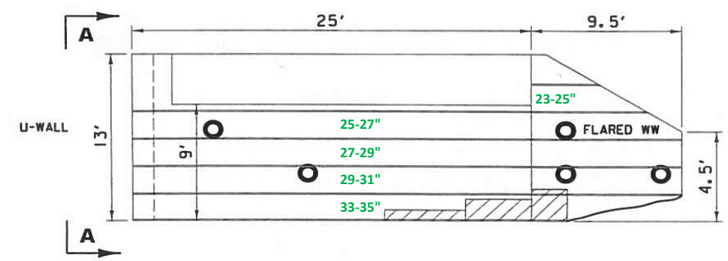
Bridge 1 Begin (West) Abutment



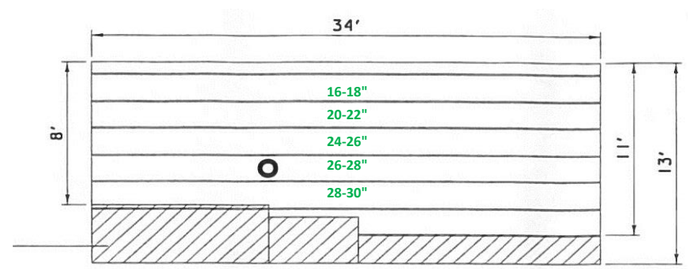
Bridge 1 End (East) Abutment



Bridge 2 Begin (West) Abutment



Bridge 2 End (East) Abutment



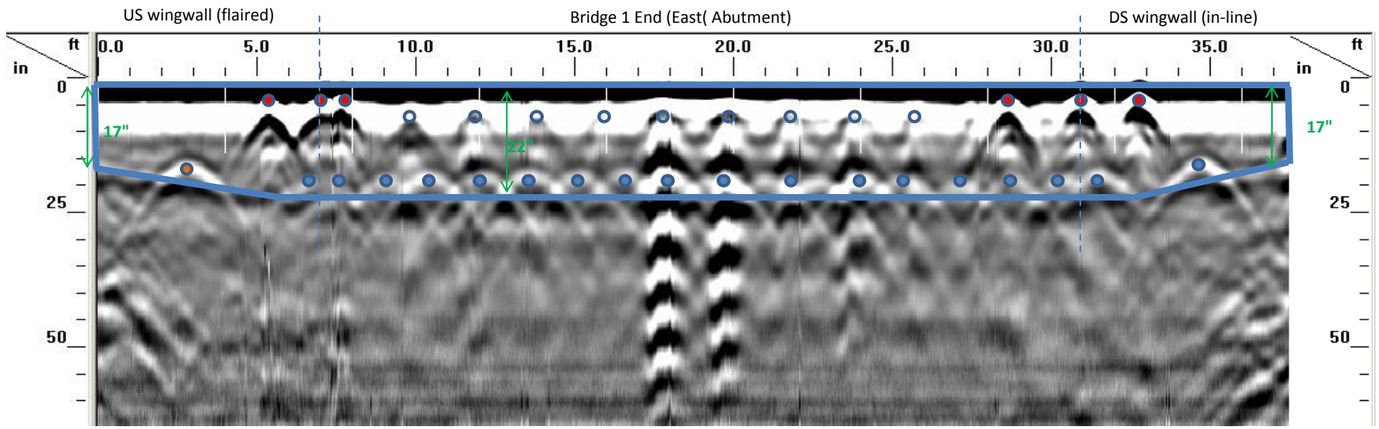
Bridge 2 End (East) Abutment

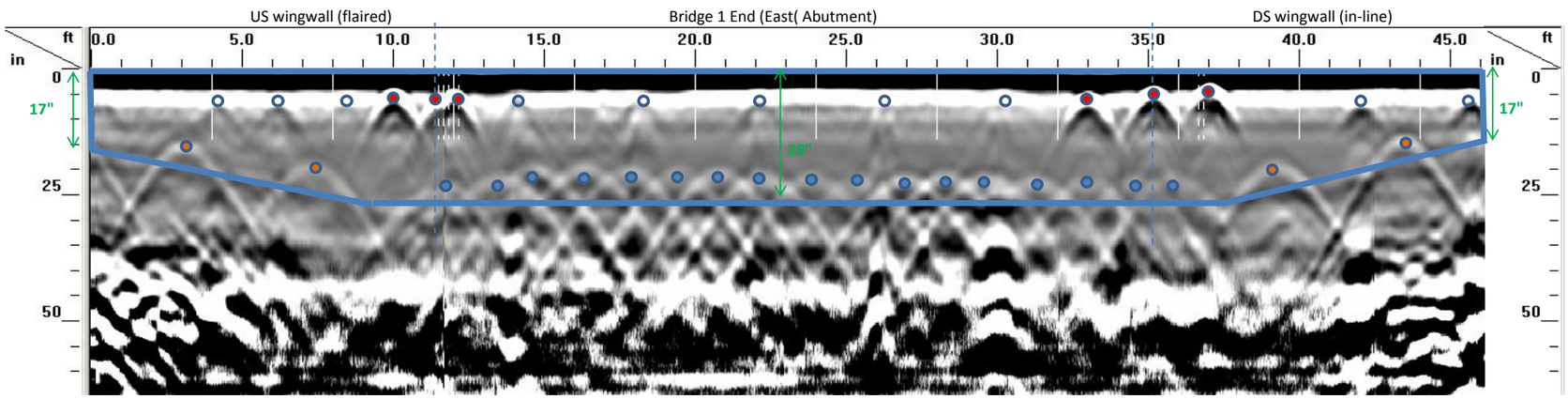
33-34" **GPR and Sonic/ultrasonic Frequency thickness Results**

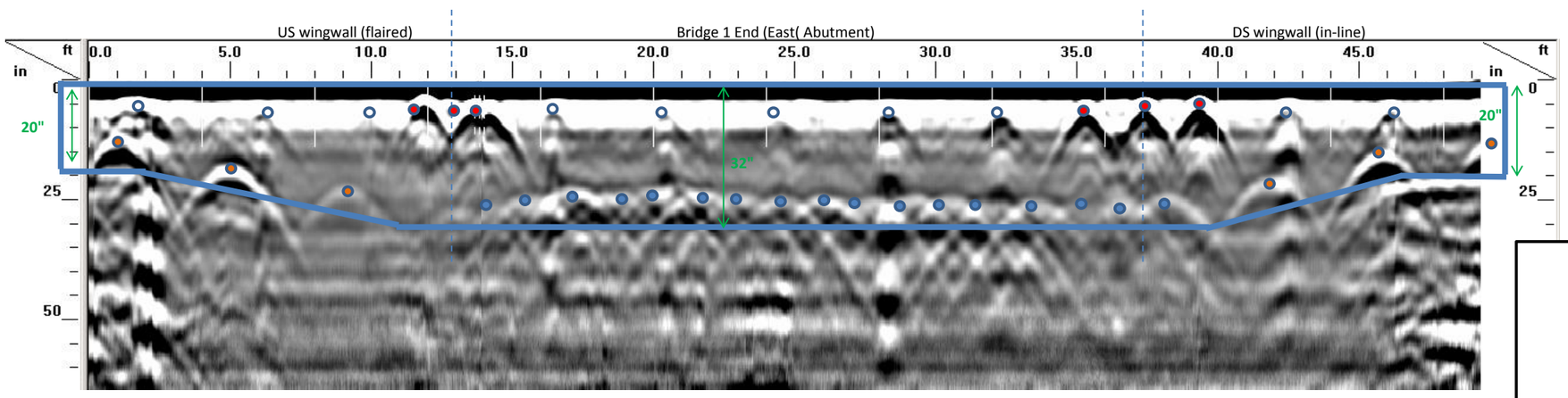
A' A

GPR Cross-section shown in Figure XXX

Johnson BF 0248(4) VT 100C Bridge 1 and 2 Johnson, VT prepared for CHA, Inc. by NDT Corporation	Abutment Geometry	
	July 2015	Figure 3



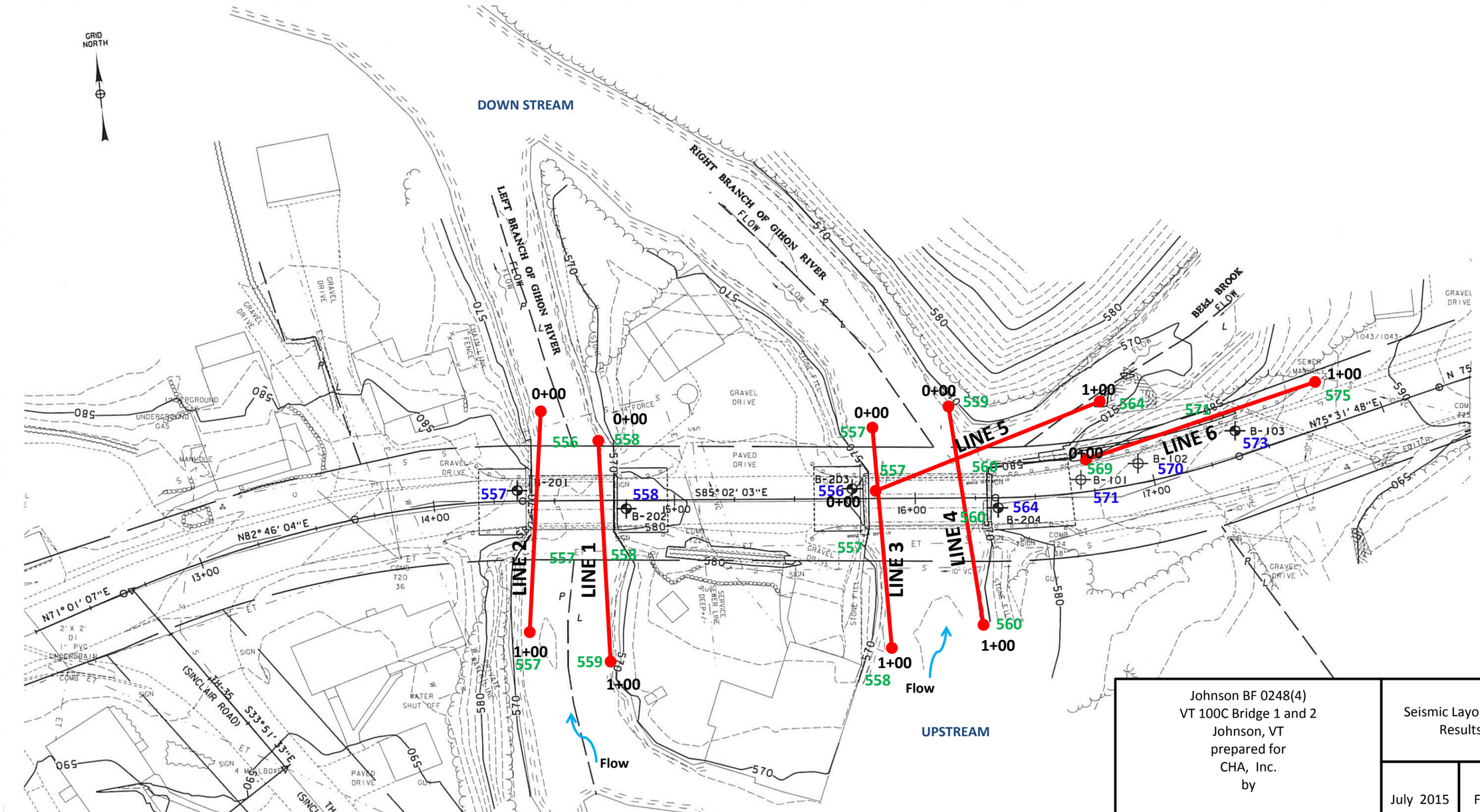




Johnson BF 0248(4) VT 100C Bridge 1 and 2 Johnson, VT prepared for CHA, Inc. by NDT Corporation	Abutment Geometry and Reinforcing	
	July 2015	Figure 4

Johnson BF 0248(4) Bridge 1 and Bridge 2

VT 100C over Gihon River
Date of Survey: 6/17/15-6/18/15



Johnson BF 0248(4) VT 100C Bridge 1 and 2 Johnson, VT prepared for CHA, Inc. by NDT Corporation	Seismic Layout and Results July 2015 — Figure 5

40 0 40 FT
SCALE

LEGEND
B-101 APPROXIMATE BORING LOCATION
B-201 PROPOSED BORING LOCATION

557 Top of Rock Elevation (CHA Borings)

560 Top of Rock Elevation Seismic Refraction

CHA
III Winners Circle, PO Box 5269
Albany, NY 12205-0269
518.453.4500 • www.chacompanies.com

BORING LOCATION PLAN

JOHNSON BF 0248(4)
VT 100C BRIDGE 2 IMPROVEMENTS
JOHNSON, VT

FIGURE **2**

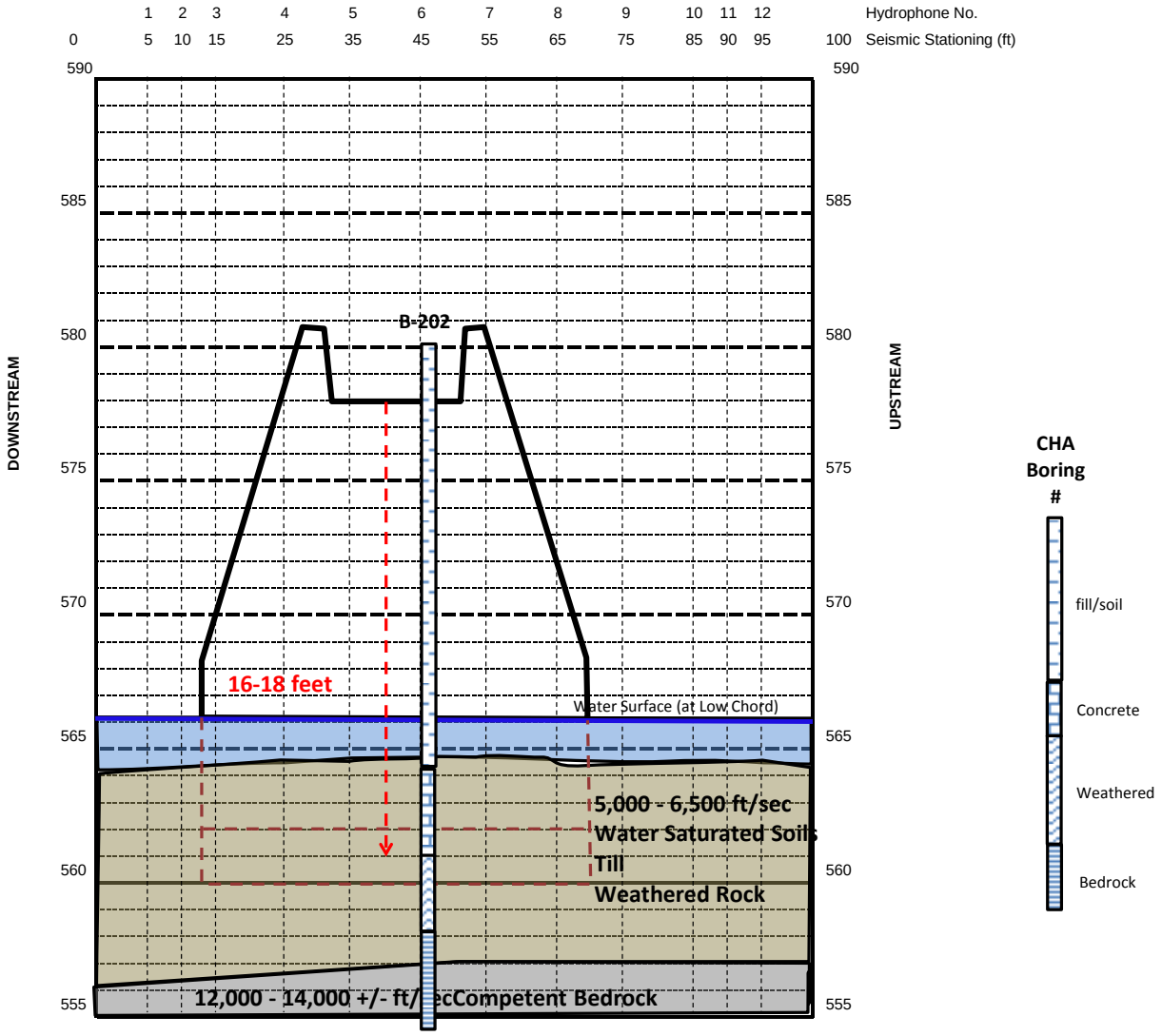
DATE: 06/2015

Johnson BF 0248(4) Bridge 1 Begin (West) Abutment

Line 2 - Approximately 5 feet from Begin (West) Abutment

VT 100C over Gihon River
Date of Survey: 6/17/15-6/18/15

LOW CHORD: 10' 6" **-----**MEASURED: center of Bridge 1 abutment bottom of beam to water surface
Field Notes: cobbles - "armored streambed", strong current



16-18 feet **Pulse Echo Results - Abutment Height**

Bottom of Abutment from Pulse Echo Results

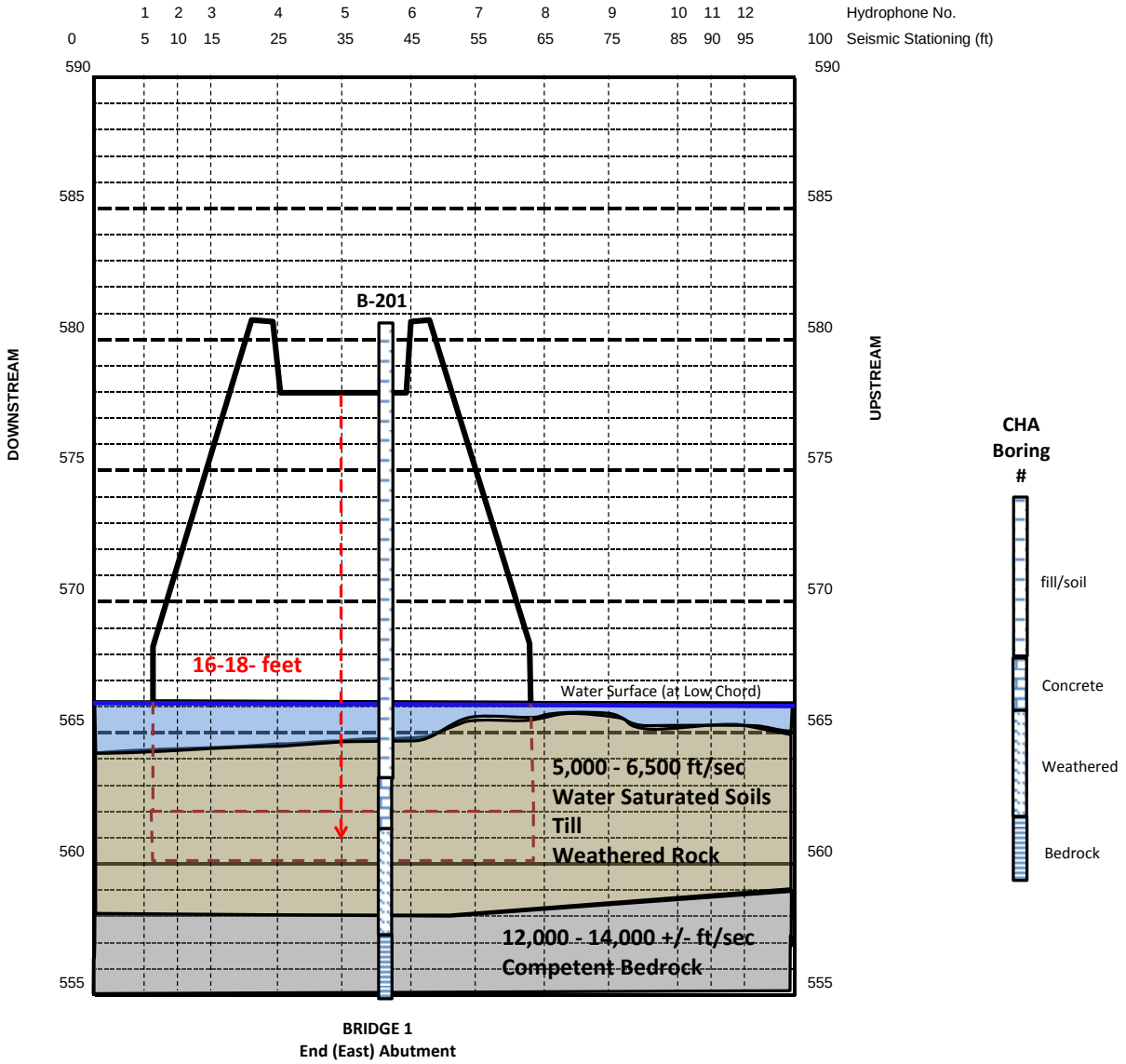
Johnson BF 0248(4) VT 100C Bridge 1 and 2 Johnson, VT prepared for CHA, Inc. by NDT Corporation	Seismic Profile Line 2	
	July, 2015	Figure 6

Johnson BF 0248(4) Bridge 1 End (East) Abutment

Line 1 - Approximately 9 feet from End (East) Abutment

VT 100C over Gihon River
Date of Survey: 6/17/15-6/18/15

LOW CHORD: 10' 6" -----MEASURED: center of Bridge 1 abutment bottom of beam to water surface
Field Notes: cobbles - "armored streambed"



16-18 feet **Pulse Echo Results - Abutment Height**

Bottom of Abutment from Pulse Echo Results

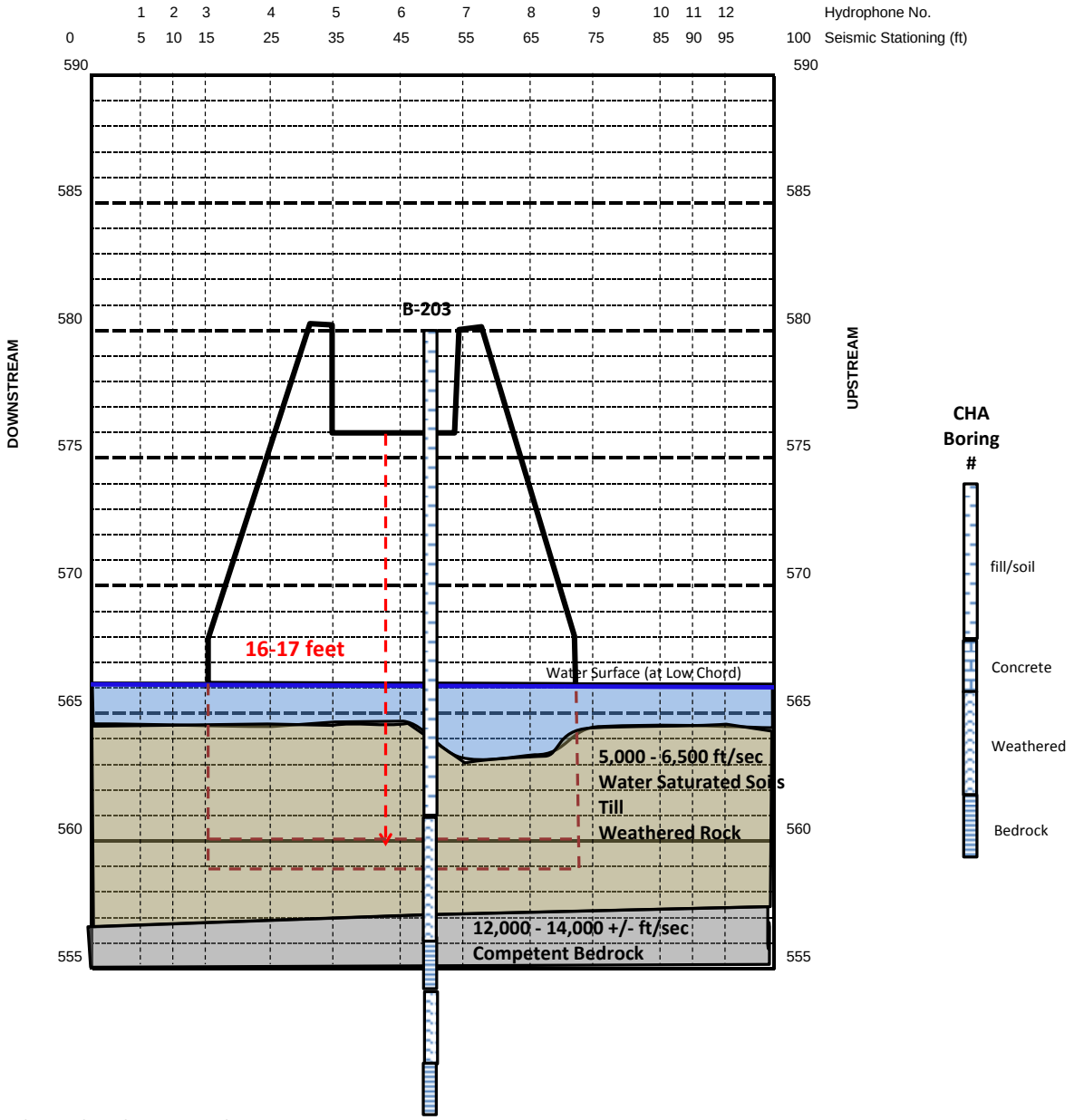
Johnson BF 0248(4) VT 100C Bridge 1 and 2 Johnson, VT prepared for CHA, Inc. by NDT Corporation	Seismic Profile Line 1	
	July, 2015	Figure 7

Johnson BF 0248(4) Bridge 2 Begin (West) Abutment

Line 3 - Approximately 5 feet from Begin (West) Abutment

VT 100C over Gihon River
Date of Survey: 6/17/15-6/18/15

LOW CHORD: 8' 6" -----MEASURED: center of Bridge 2 abutment bottom of beam to water surface
Field Notes: cobbles - "armored streambed", strong current



16-17 feet **Pulse Echo Results - Abutment Height**

Bottom of Abutment from Pulse Echo Results

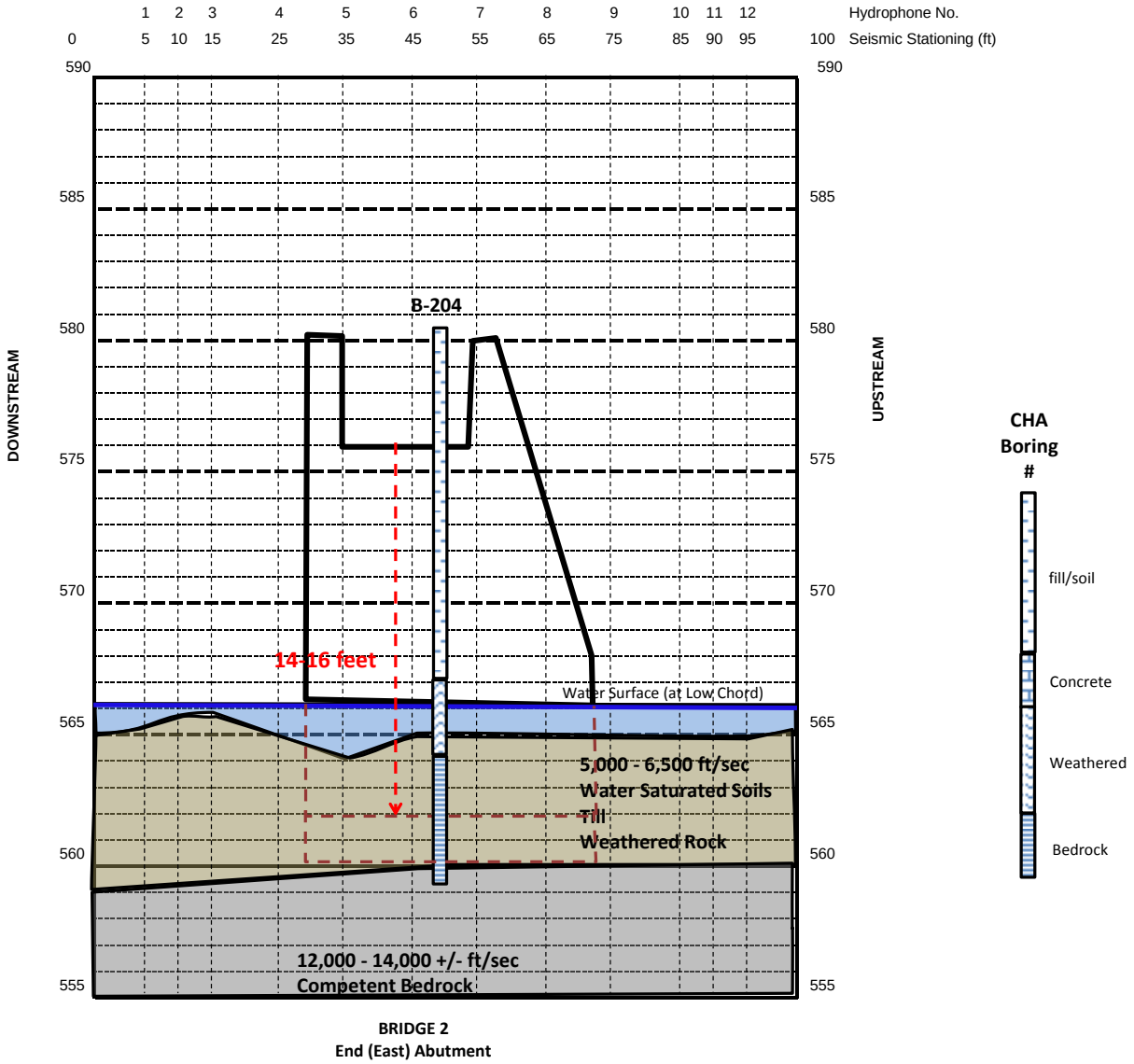
Johnson BF 0248(4) VT 100C Bridge 1 and 2 Johnson, VT prepared for CHA, Inc. by NDT Corporation	Seismic Profile Line 3	
	July, 2015	Figure 8

Johnson BF 0248(4) Bridge 2 End (East) Abutment

Line 4 - Approximately 10 feet from End (East) Abutment

VT 100C over Gihon River
Date of Survey: 6/17/15-6/18/15

LOW CHORD: 8' 6" -----MEASURED: center of Bridge 2 abutment bottom of beam to water surface
Field Notes: cobbles - "armored streambed", down stream sandy bottom



14-16 feet **Pulse Echo Results - Abutment Height**

Bottom of Abutment from Pulse Echo Results

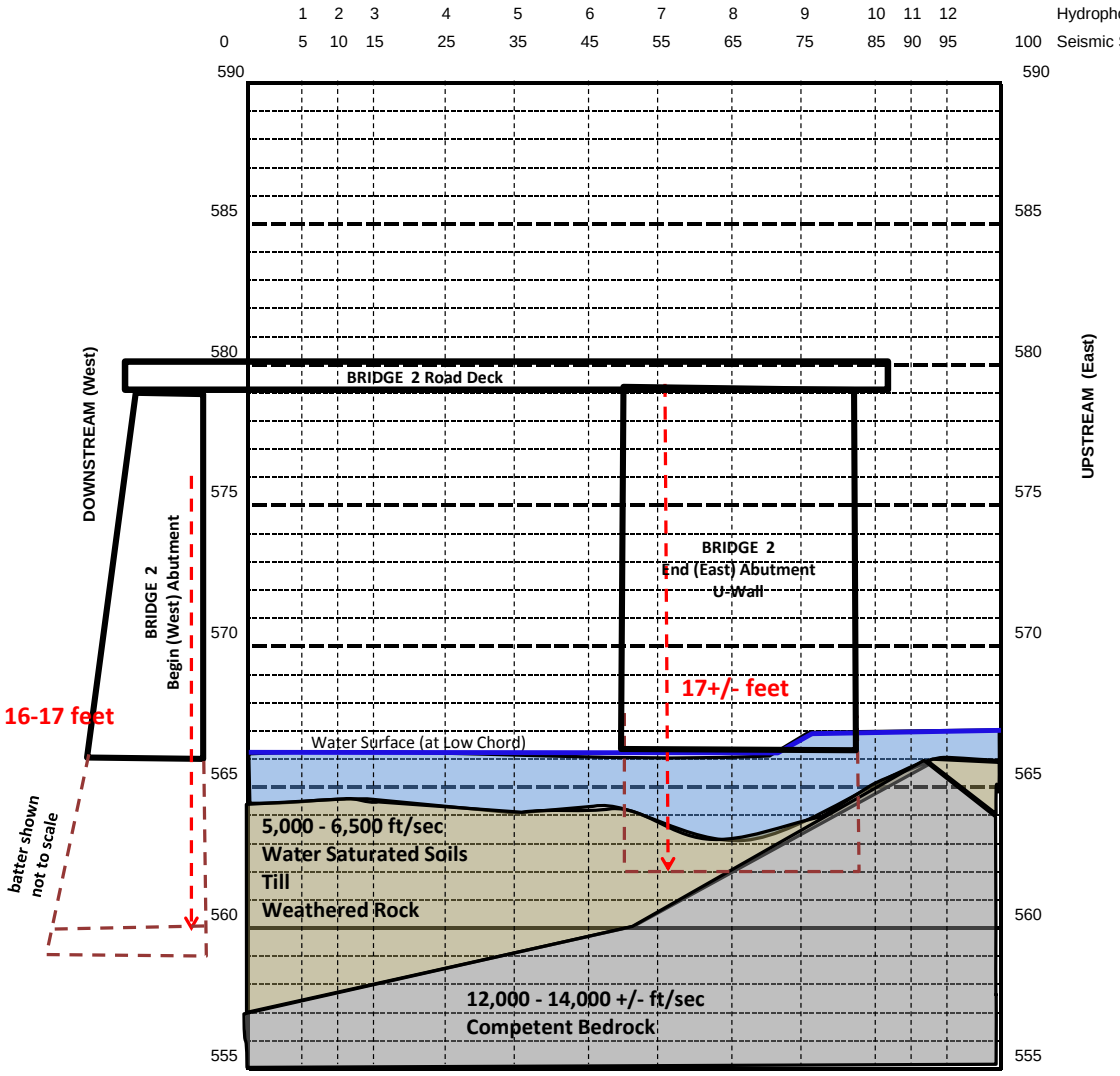
Johnson BF 0248(4) VT 100C Bridge 1 and 2 Johnson, VT prepared for CHA, Inc. by NDT Corporation	Seismic Profile Line 9	
	July, 2015	Figure 9

Johnson BF 0248(4) Bridge 2 Down-Stream

Line 5 - DS Begin Abutment across the stream parallel to U-Wall of Bridge 2

VT 100C over Gihon River
Date of Survey: 6/17/15-6/18/15

LOW CHORD: 8' 6" -----MEASURED: center of Bridge 2 abutment bottom of beam to water surface
Field Notes: cobbles, sand, outcrop upstream in tributary at small falls



17+/- feet **Pulse Echo Results - Abutment Height**

Bottom of Abutment from Pulse Echo Results

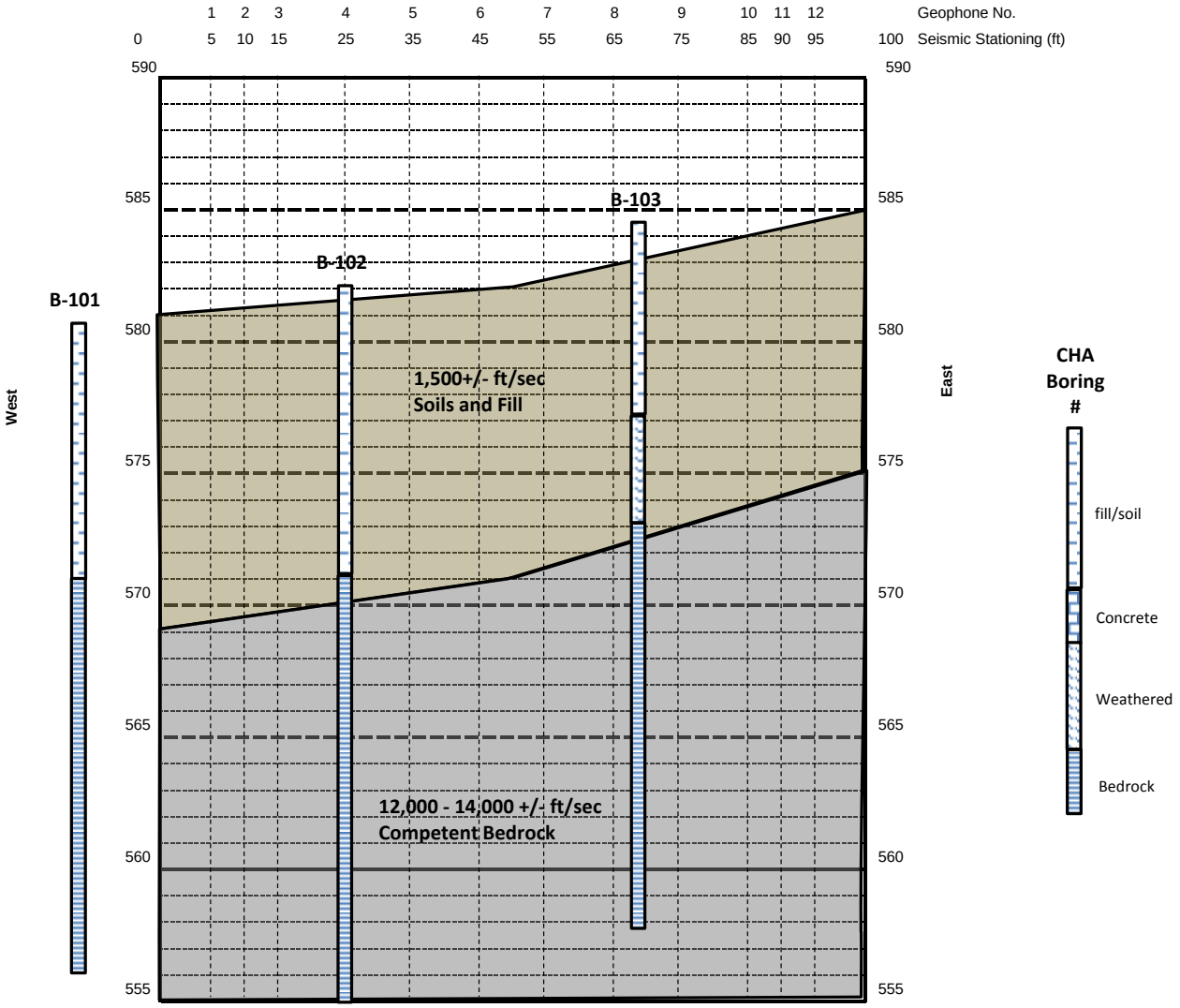
Johnson BF 0248(4) VT 100C Bridge 1 and 2 Johnson, VT prepared for CHA, Inc. by NDT Corporation	Seismic Profile Line 5	
	July, 2015	Figure 10

Johnson BF 0248(4) Bridge 2 North Curb of East Approach

Line 6 - Sta:16+75 to Sta: 17+25 North Curb

VT 100C over Gihon River
Date of Survey: 6/17/15-6/18/15

Begin 10 eastt of U-wall and end approximately 5 feet west of Sewer Manhole Cover
Field Notes: Soft soils



Johnson BF 0248(4) VT 100C Bridge 1 and 2 Johnson, VT prepared for CHA, Inc. by NDT Corporation	Seismic Profile Line 6	
	July, 2015	Figure 11

APPENDIX 1

SONIC/ULTRASONIC NONDESTRUCTIVE TESTING OF

APPENDIX
SONIC/ULTRASONIC NONDESTRUCTIVE TESTING OF CONCRETE

The sonic/ultrasonic measurements made to determine the characteristics of concrete (or rock) are generated by a relatively low energy source as a single discrete wide band impulse with a pulsed transducer, projectile, mechanical hammer, etc. Practical problems and the condition of the concrete surface largely determine the source(s) to be used. A rough concrete surface that has deposits of organic materials or mineral deposits generally requires a more powerful energy source whereas a relatively new or wet concrete may be inspected by the use of a pulsed transducer or other higher frequency source. In general high frequency sources that may work well in the laboratory may be unusable for field conditions. High frequency sources have the advantage of high resolution but the disadvantage of low penetration. While metals can be tested in the megahertz range, such signals in concrete will not have measurable signals for more than an inch in thickness. The energy source should be sufficient to maximize the resolution, have sufficient penetration to examine the concrete being tested and enough energy to excite the fundamental frequencies being sought.

The transmitted energy is in the form of three principal wave types: compressional (contraction/expansion, spring-like particle motion), shear (traction-sliding motion), and surface waves (combination of motions). Each boundary that has density and or velocity contrast will reflect and/or refract these waves. Compressional and shear wave velocity values are determined by the Young's, shear and bulk moduli values as well as the density and Poisson's Ratio. In turn the velocity can be used to determine the moduli values and Poisson's Ratio given that the density is known or can be assumed. The moduli values measured are the dynamic moduli values at low strain. In general, the difference between the dynamic values and the static values is almost entirely controlled by the crack densities of the concrete. Using the modulus values, a reasonable estimate of the unconfined compressive strength can be determined. The strength is largely dependent on the crack density of the concrete and for static tests, the orientation of the cracks. For static testing, cracks perpendicular to the axis of the core and perpendicular to the directed stress will produce a strength (static) that is not greatly different from uncracked concrete. The applied stress closes or compresses the cracks. Cracks that are near 45º to the direction of stress will result in lowest static strength. The approximate orientation of the cracks can be determined with dynamic measurements of the velocity values in different directions.

NDT Corporation makes several determinations from one energy impact. The velocity is measured directly from the energy point of impact to a linear array(s) of sensors on the surface. The array length is usually in excess of the thickness of the concrete being tested. In addition to the velocity measurements, reflections are measured individually or determined from a frequency analysis of the time domain recordings. Each reflecting surface (change of density and/or velocity) produces a multi-path reflection in the layer it bounds. A generated wave will travel to a delamination surface and reflect back to the surface of the concrete where it is reflected back to the delamination, resulting in multi-reflections that are apparent in the frequency domain. These reverberations (echoes) are particularly diagnostic of delaminations and thickness of the concrete. They will readily distinguish the presence of local delaminations, cracked or decomposed inclusions by the particular frequency band generated at the mechanical discontinuity. If a delamination is

severe or large in area, the reflected signals are strong resulting in a low frequency, high amplitude, and long duration "ringing" signal or a drum head effect that is usually quite distinguishable. This is the basis of the 'chain drag" using the human ear as the sensor to recognize frequency differences. The ear however is limited in its perception and will only distinguish within the hearing range.

DIRECT AND REFRACTED ENERGY

One of the advantages of the sonic/ultrasonic method is its ability to "look through" overlying materials coatings, particularly decomposed "softer layers" when the array(s) is configured properly. This is done using refracted waves associated with the different layer velocities or by careful examination of the resonant frequencies associated with such layering.

The diagram below shows the wave path for refracted energy generated for a softer (1) lower velocity layer over a harder (2) higher velocity layer. For example asphalt (1) over good concrete (2). The wave is bent (similar to the appearance of a stick in water) and travels along the boundary between the lower velocity layer and the higher velocity layer and radiates back to the surface. The higher velocity of the good concrete allows the refracted wave to overtake the direct wave in layer 1 at some distance designated as $D_{1|2}$. To the left of this point the lower velocity of layer 1 will be measured and beyond it the velocity of the deeper layer 2 is measured.

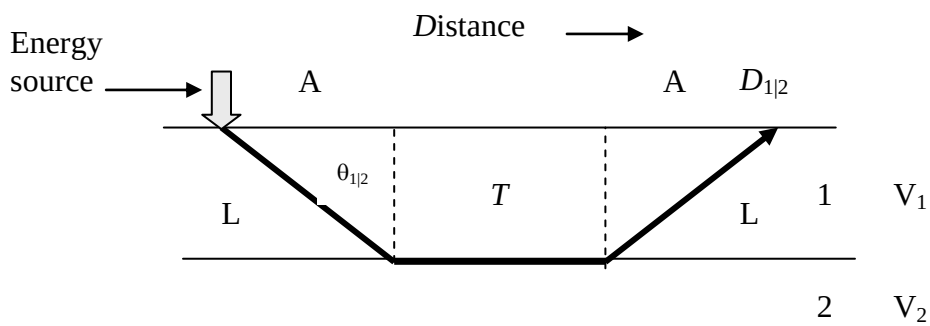


Figure A1

The time for the direct path is D/V_1; the refracted path time is $2L/V_1+(D_{1|2}-2A)/V_2$.
The array of sensors is placed in the distance direction and the time elapsed (travel time) from the time of energy impact to the sensor distance is measured. The velocity is determined from this time-distance measurement(s). The angle θ is the angle between the perpendicular to the layer and the incident wave that is critically refracted. The sine of this angle is the velocity of the first layer divided by that of the second layer (Snell's Law). The distance shown $D_{1|2}$ is the point on the surface where the refracted time arrival equals that of the direct wave (the refracted wave travels at a higher velocity than the direct wave).

$$\frac{D}{T} = \frac{D - 2\tan\Theta}{V_2} + \frac{2T}{V_1 \cos\Theta}$$

The thickness is expressed as:

$$T = \frac{D_{1|2}}{2}\sqrt{\frac{V_2 - V_1}{V_2 + V_1}}$$

D is the distance and *T* is the thickness. Since the times as well as the distances are measured, then V_1 and V_2 are determined. If a plot of distance versus time is made then the resulting graph will look like Figure A2.

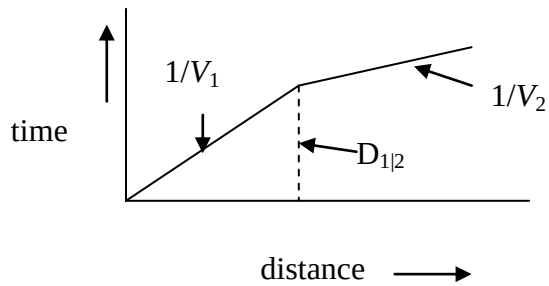


Figure A2

If the concrete has no overlay then the concrete velocity is simply *D*/*T*.

The resonant frequencies are determined by the thickness and velocity of the material. Since the velocity is measured as above, then the thickness can be determined directly.

The resonance of a simple beam is given by:

$$f = \frac{nV}{2L} \qquad (\textit{fixed} - \textit{fixed}, \quad \textit{free} - \textit{free}) \quad \textit{where } n = 1,2,3 - - -$$

$$f = \frac{nV}{4L} \qquad (\textit{open} - \textit{fixed}), \quad \textit{where} \quad n = 1,3,5,7 - - -$$

Since the frequency and velocity are measured, the thickness is determined. This thickness can be the thickness of the concrete floor, deck slabs, or column being measured or it can be the thickness of concrete overlying a delamination.

While the refracted wave is dependent only on a contrast in velocity, a reflection can take place where there is a change in velocity or density or both. The impedance (RF reflection coefficient) which causes a wave to be reflected is given by:

$$RF = \frac{\rho_2 V_2 - \rho_1 V_1}{\rho_2 V_2 + \rho V_{11}}$$

Where ρ is the density and V is the velocity of the material. The impedence determines the strength of the reflection. The contrast between an air filled void at the back of or within the concrete is significant; the velocity in air is 1,000ft/sec. and the velocity of good concrete is 13,000ft/sec. The density differences are of course very large between the concrete and air. A similar difference exists for a water filled void where the velocity in water is 5,000ft/sec. and concrete is nearly a factor of 2.5 denser. Voiding behind a liner or under a slab is usually well distinguished by a distinct " ringing" resonant frequency, reffered to above as a drum head effect..

MODULI VALUES AND STRENGTH

The moduli values as stated above are determined from the velocity values using an assumed or measured density. The density is usually the best known or best estimated value for the concrete, its variance generally does not affect the calculations significantly.

The relationships for Young's modulus versus the compressional velocity are shown in Figure A4; shear modulus versus the shear velocity Figure A5; Poisson's Ratio versus the compressional and shear wave velocities Figure A6; and finally a relationship between the velocity values (compressional and shear) and the unconfined compressiive strength of concrete, Figure A7.

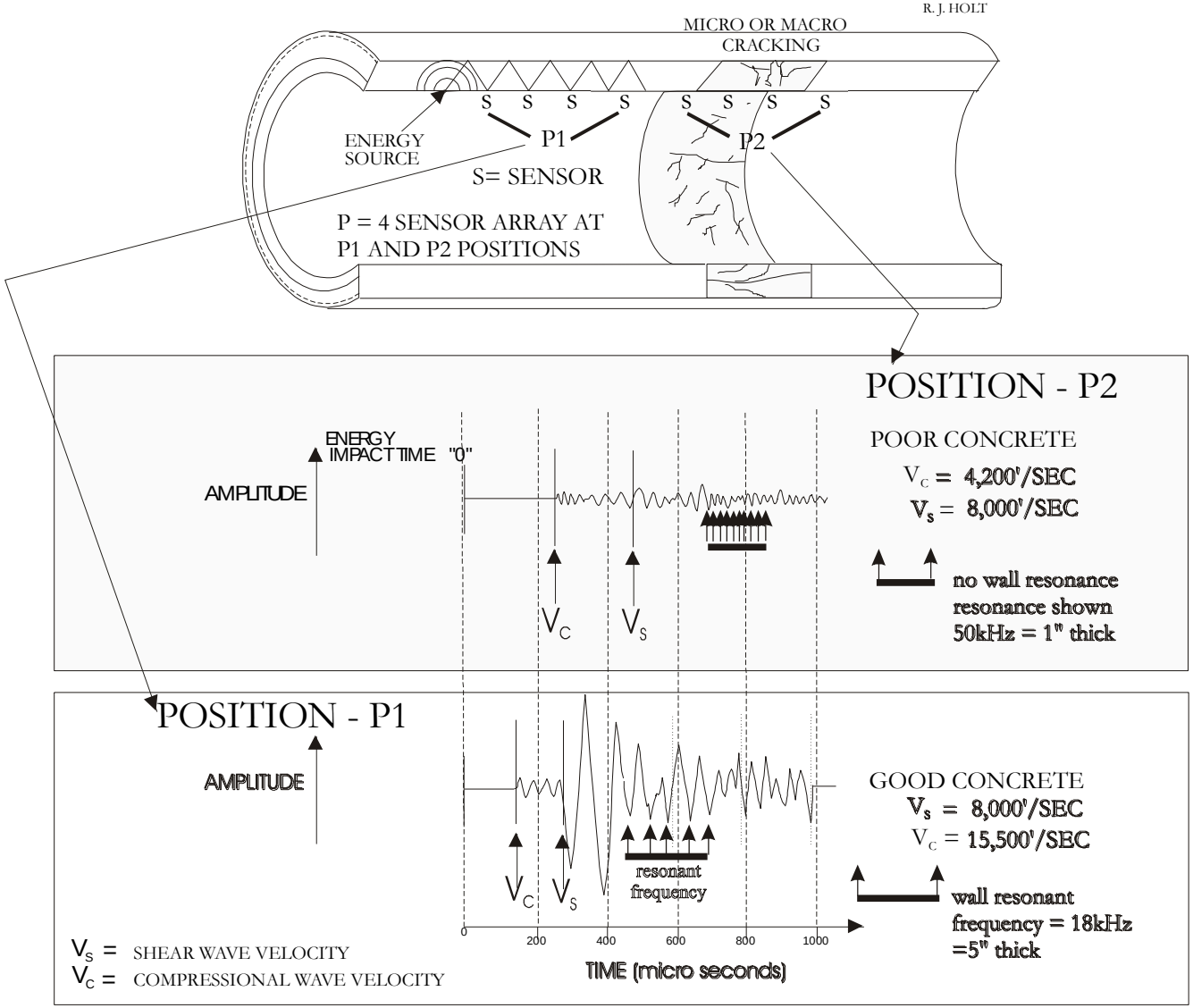
Figure A3 is illustrative of a tunnel liner or pipe investigation where there has been circumferential damage, perhaps at a construction joint or an outside zone of weakness (rock shear or fault, soil washout etc.) that has affected the integrity of the liner. The damage need not be visible; there can be a 20% reduction in the strength of the concrete from micro-cracking that is not visible to the naked eye. The process of deteroration of most concrete starts at the micro level and with continued stress the micro cracks coalesce into macro cracks and finally to spalling. The ability to measure at the micro level well in advance of future needed repairs provides a management tool for establishing proirities for repair, projected budgets, and asset valuation

SONIC/ULTRASONIC TUNNEL & PIPE LINE TESTING

Circumferential (face to back) Cracking

NDT ENGINEERING, INC.

R. J. HOLT



SONIC/ULTRASONIC TIME AMPLITUDE RECORDS
FOR FOURTH SENSOR - POOR AND GOOD CONCRETE

FIGURE A3

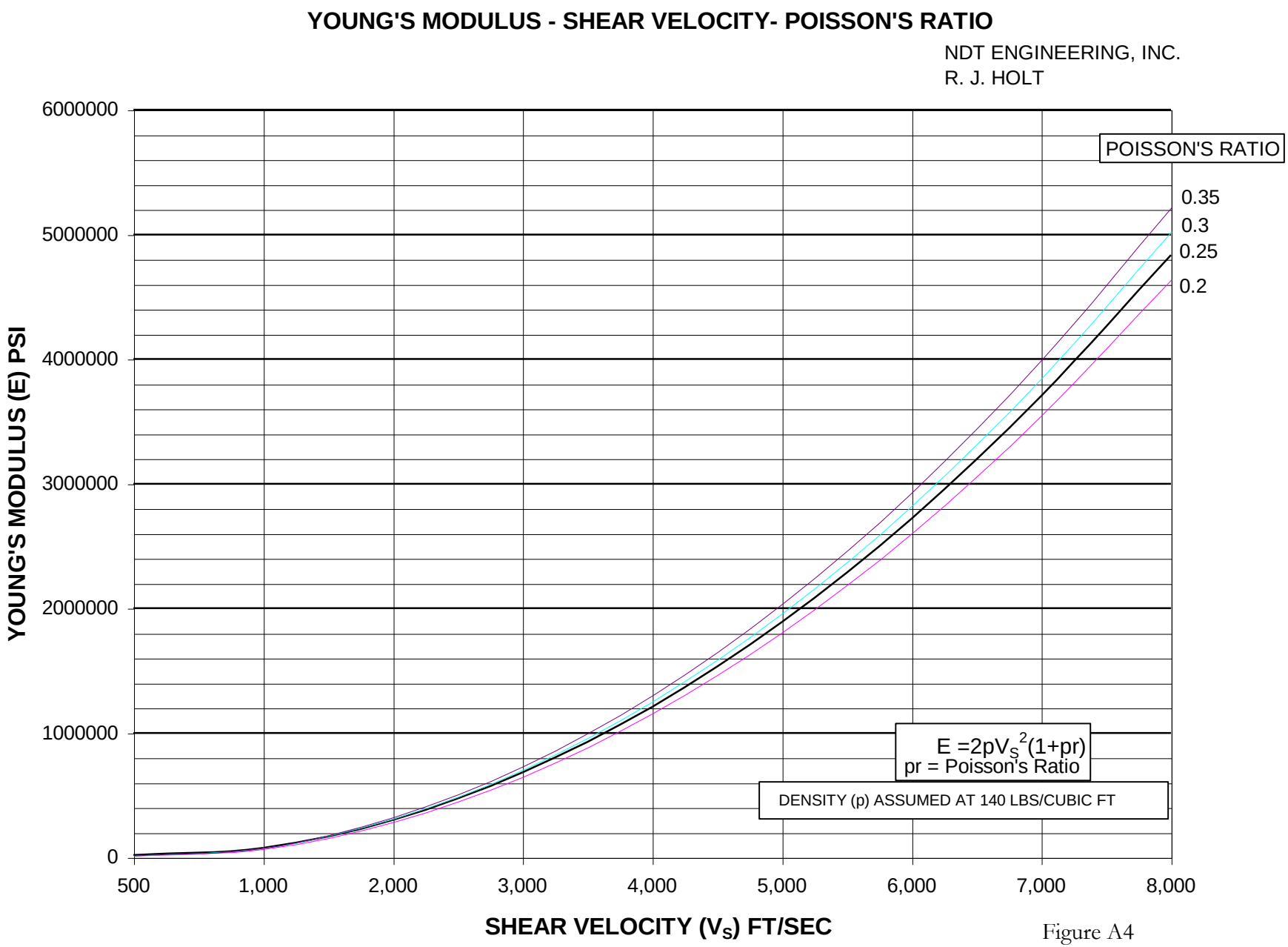

YOUNG'S MODULUS - SHEAR VELOCITY- POISSON'S RATIO
NDT ENGINEERING, INC.
R. J. HOLT
POISSON'S RATIO
0.35
0.3
0.25
0.2
E =2pV$_S^2$(1+pr)
pr = Poisson's Ratio
DENSITY (p) ASSUMED AT 140 LBS/CUBIC FT
YOUNG'S MODULUS (E) PSI
6000000
5000000
4000000
3000000
2000000
1000000
0
500
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
SHEAR VELOCITY (V$_S$) FT/SEC

Figure A4

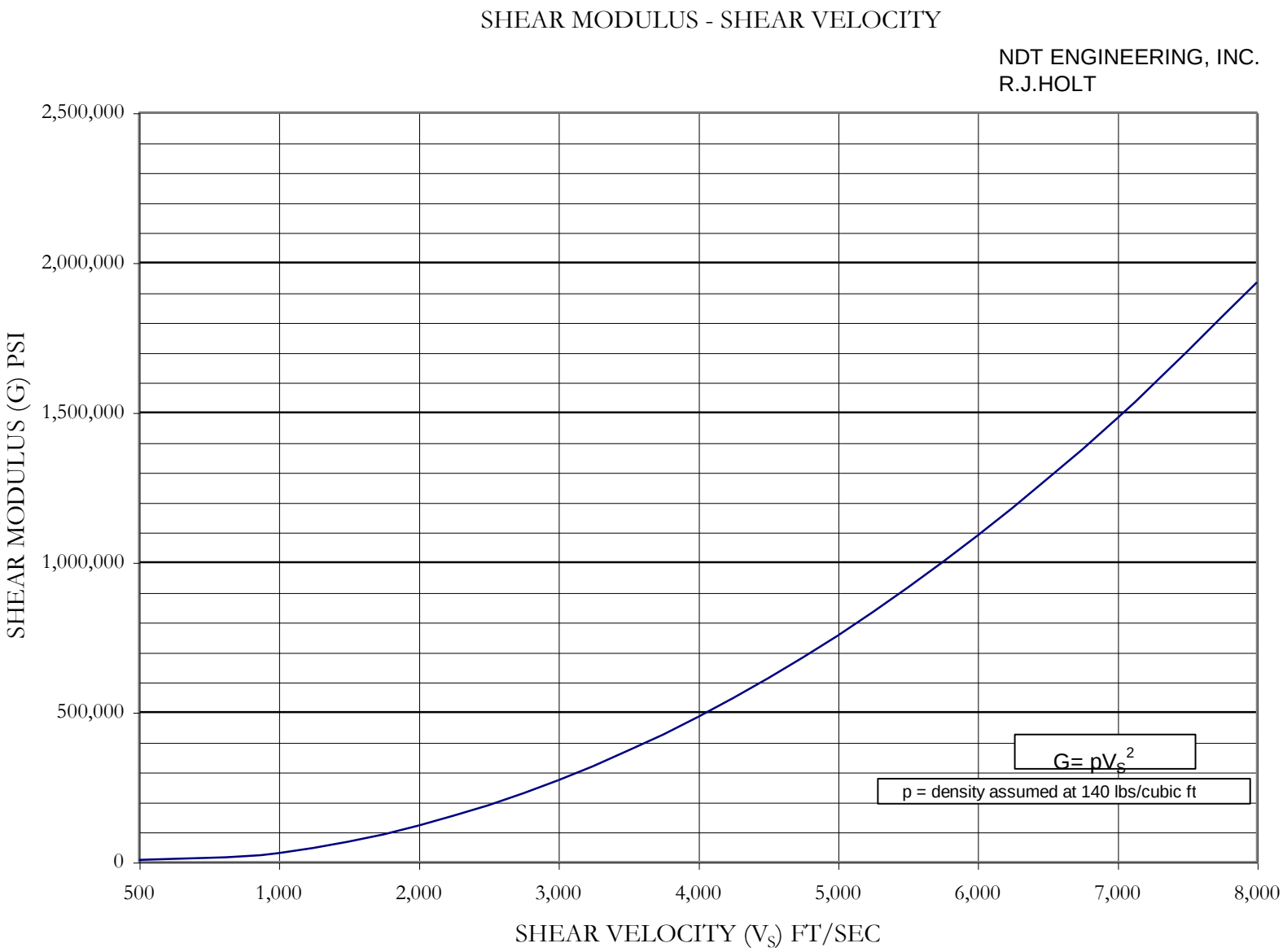

SHEAR MODULUS - SHEAR VELOCITY
NDT ENGINEERING, INC.
R.J.HOLT
2,500,000
2,000,000
1,500,000
1,000,000
500,000
0
SHEAR MODULUS (G) PSI
500
1,000
2,000
3,000
4,000
5,000
6,000
7,000
8,000
SHEAR VELOCITY (V_S) FT/SEC
$G = pV_S^2$
p = density assumed at 140 lbs/cubic ft

Figure A5

COMPRESSIONAL VELOCITY - SHEAR VELOCITY - POISSON'S RATIO

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R J HOLT

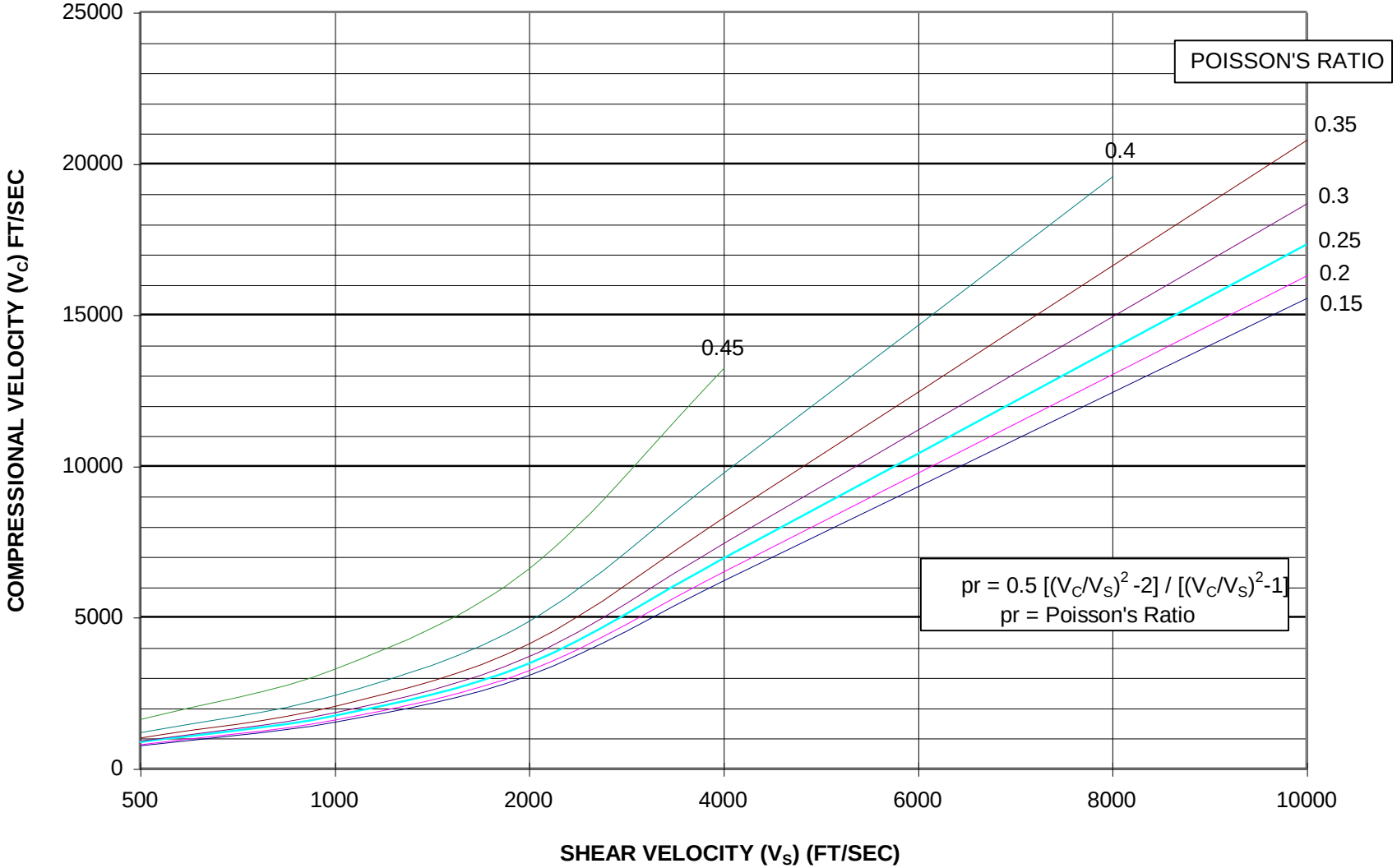
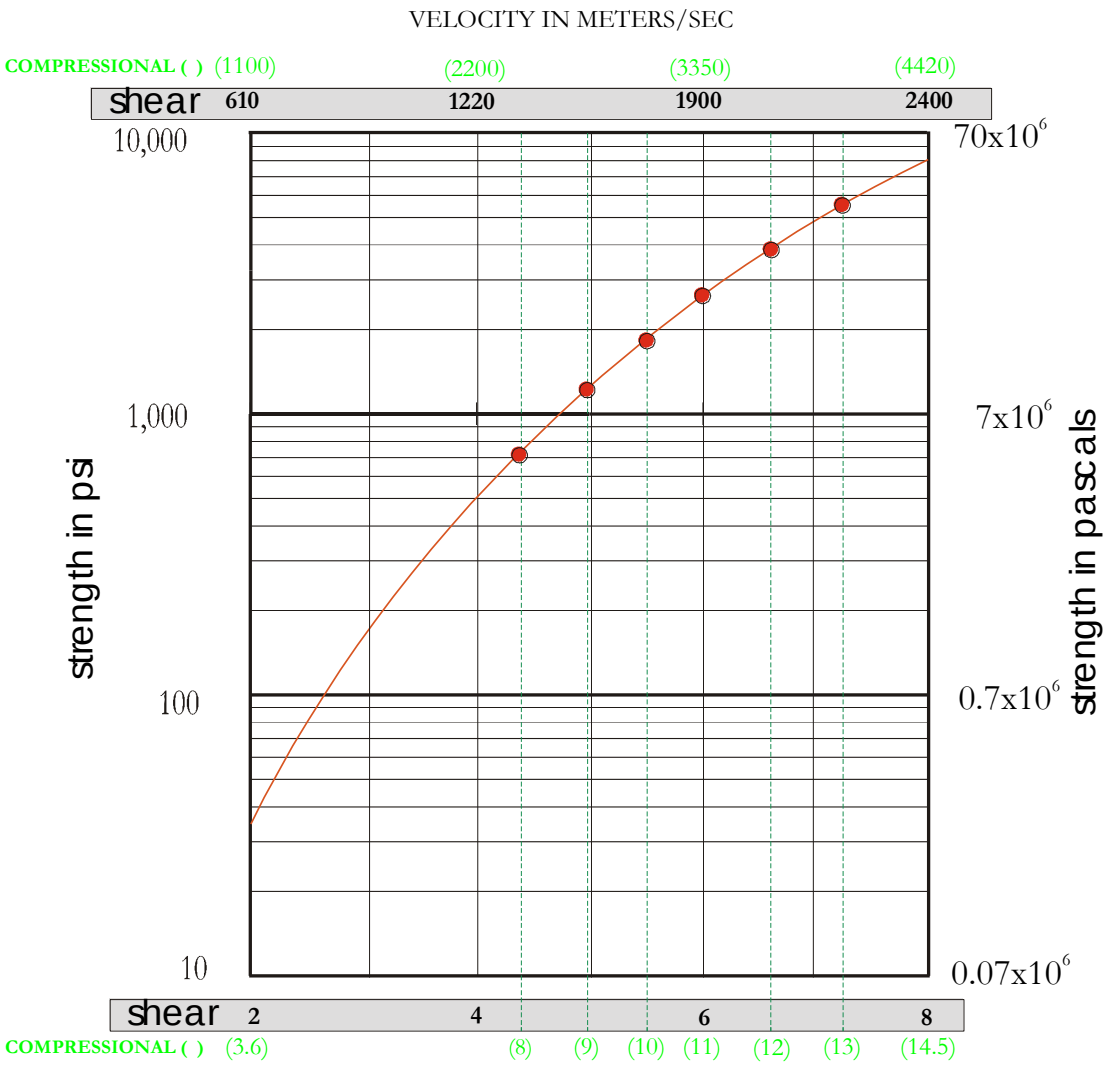
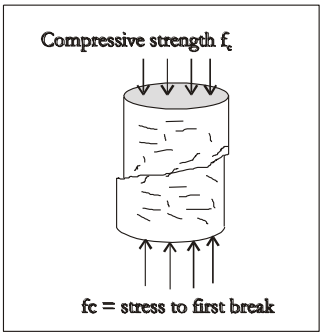


Figure A6

strength of concrete versus velocity

r. j. holt





VELOCITY IN FEET/SEC (X 1,000)

CURVE FOR RATIO: $V_{SHEAR} / V_{COMPRESSIONAL} = 0.55$
EQUALS POISSON'S RATIO OF 0.28

FIGURE A7

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APPENDIX 2

GROUND PENETRATING RADAR

APPENDIX: GROUND PENETRATING RADAR

Ground Penetrating Radar (GPR) is an electrical geophysical method for evaluating subsurface conditions by transmitting high frequency electromagnetic waves into the ground and detecting the energy reflected back to the surface. Electromagnetic signals are transmitted from the antenna (transmitter and receiver) at ground surface and reflected back to the antenna from interfaces with differing electrical (dielectric constant and conductivity) properties. The greater the contrast in the electrical properties between two materials, the more energy that is reflected to the surface and the more defined results are.

GPR reflections typically occur at subsurface discontinuities such as:

- Buried metal objects (utilities, tanks, reinforcing)
- Open and water filled voids
- Water table
- Soil stratification
- Seepage paths
- Bedrock fractures

The depth of penetration of GPR is site specific, limited by the attenuation of the electromagnetic energy. Signal attenuation is controlled by four different mechanisms:

- Scattering: energy losses due to scattering occur when signals are dispersed in random directions, away from the receiving antenna, by closely spaced rebar or large irregular shaped objects, such as boulders or tree stumps.
- High conductivity layers: the greater the conductivity values of materials at a site, the more signal attenuation or less penetration. (Mineral content, high moisture content, water table, metal plates, etc.)
- Water/Moisture Content: water molecules polarize in the presence of the applied electromagnetic field. Electromagnetic energy is lost to the radar system when it is converted to kinetic and thermal energy.
- Clays, (Ion content): ions along clay surfaces polarize in the presence of the applied electromagnetic field. Electromagnetic energy is lost to the radar when it is converted to kinetic and thermal energy.

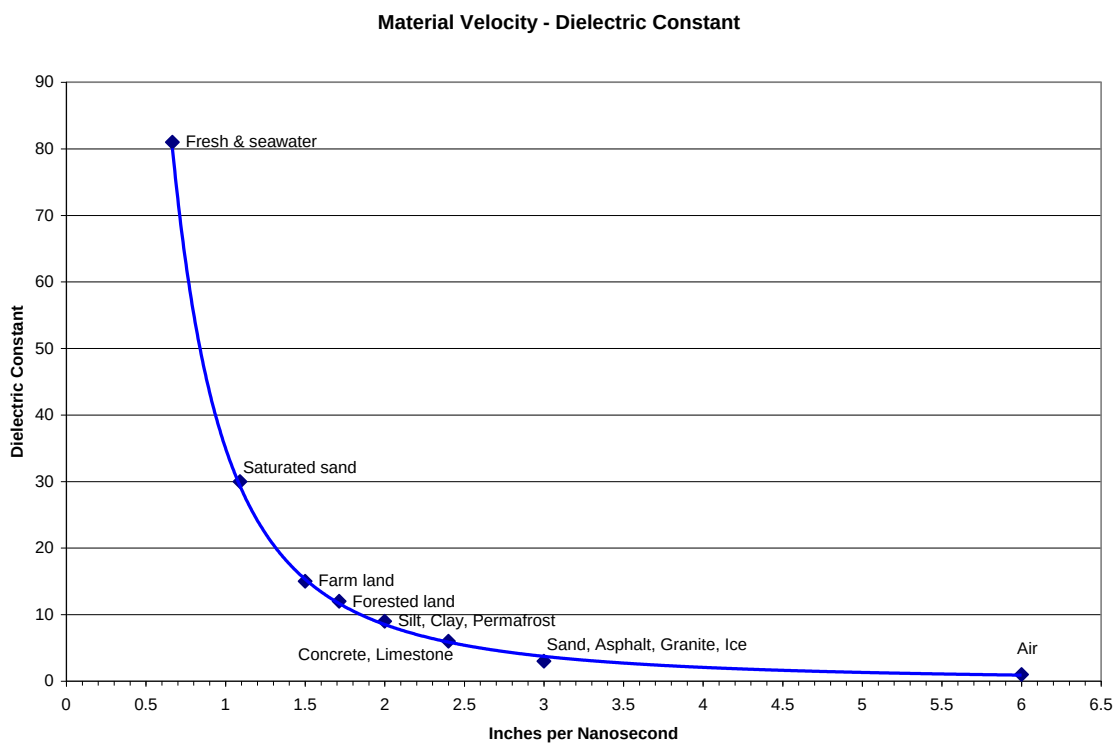
An onsite calibration should be conducted so that the velocity for the materials and the depth of penetration can be determined. Sites can be electrically variable so it may be necessary to conduct multiple onsite calibrations.

Signal penetration is also dependent on the frequency of the antenna. High frequency antennas have shallow penetration and high resolution. A 1500 MHz high frequency antenna has an approximate depth of penetration of 1.5 feet and is able to identify wire mesh. Low frequency antennas have lower resolution and deeper depth of investigation. A 400 MHz antenna is capable of penetrating 10 to 15 feet in dry soils.

Ground Penetrating Radar (GPR) can be used to locate underground pipes, buried drums, foundations, voids in rock and concrete, soil settlement, determine stratigraphy, depth to

water table, buried artifacts, filled excavations, and locate voids/settlement behind walls and under floor slabs, etc. GPR is also a good tool for evaluating concrete structures such as bridges, walls, beams, ceilings, etc where the GPR can locate rebar and conduits, quantify rebar spacing, cover variability over reinforcing, and concrete thickness.

Laterally GPR can cover large areas relatively quickly. Using a grid pattern of survey lines it is very effective for mapping the lateral extents of subsurface features as well as calculating the depth to the features of interest. Depth of investigation can be estimated using material dielectric constants and the diagram shown below. Accurate depth calculations require an onsite calibration, to determine the electrical properties (speed of the signal) of the materials at the site. Depth calibrations typically consist of collecting GPR data over a metal target with a known depth. Known utilities, and buried metal plates are good targets for calibrations. GPR surveys can be very effective when coupled with other geophysical surveys and/or ground truth methods to verify, correlate and extrapolate GPR results. GPR surveys are a fast and cost effective method to collect data over large or obstructed sites, and isolate anomalies and areas where borings or other methods can be focused for the best interest of a project.



GPR systems consist of: Control unit (pulse transmitter, digital recorder, data storage, monitor); and an antenna(s).

The GPR control unit is a computer which controls data acquisition parameters, such as sampling rate, range, gain control, filtering, etc. The Control Unit also visually displays the data, digitally archives the data, and allows for play back of the data.

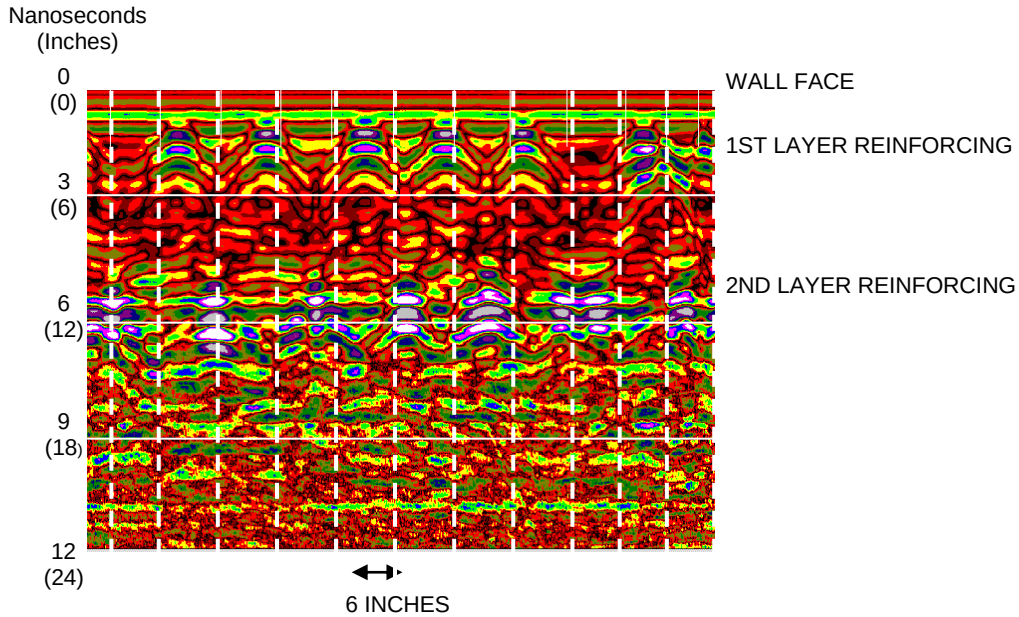
Coaxial cable connects the control unit to the antenna. The antenna(s) are sealed and shielded in fiberglass housing for the transmitter and receiver. Selection of the antenna is dictated by the requirements of the survey. For high resolution, near-surface data, a high frequency antenna is used; for deeper penetration investigation, a lower frequency antenna is used. Typically the 80 to 300 MHz antennas are used for geologic surveys; 300 to 900MHz are used for utility, near surface voiding settlement, foundation, etc surveys while the 900 to 1500 MHz is used for concrete assessment.

ACQUISITION AND INTERPRETATION:

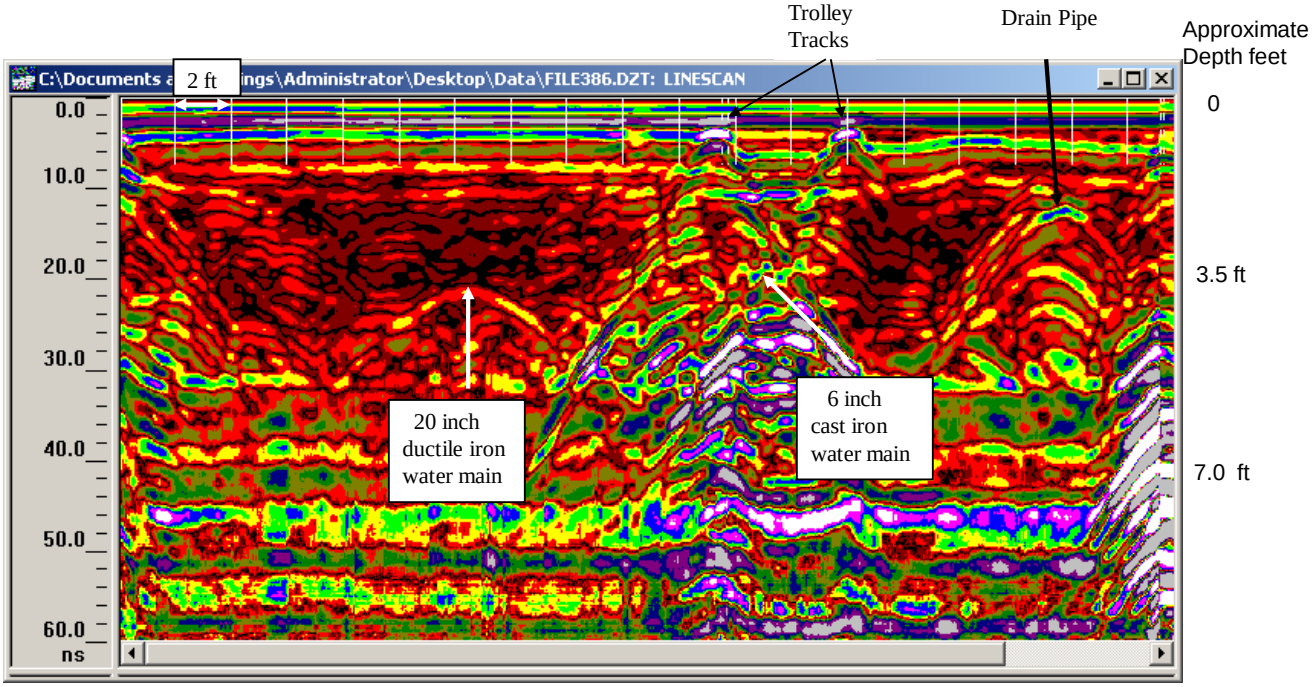
Radar data are typically acquired at a slow walking speed. Data is displayed on LCD screen for field verification and quality control of results and digitally saved. Calibrated measuring wheels are used to automatically added footage/station markers to the digital data. The saved data can be print or post processed.

Interpretation of GPR data is subjective. GPR results should be verified with borings or test pits. The strength of a reflected signal and/or the continuity of the reflector across the record may be indicative of a stratigraphic contact. Point targets, such as reinforcing, buried utilities, boulders, create a distinctive parabolic feature on GPR records. Annotated GPR records of reinforcing and buried metal utilities are shown below. Positive identification of the source of a point targets is subjective, as the GPR signature of a pipe is similar to that of a large boulder. Computer processing is available though it is somewhat costly and in most cases is not necessary, except for presentation purposes.

GPR RECORD
12" THICK WALL WITH REINFORCING



Buried Utility Detection and Mapping



APPENDIX 3

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.

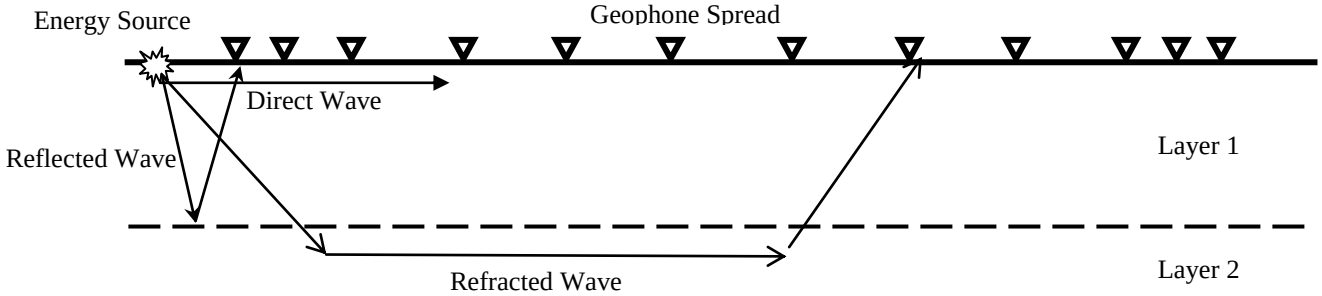


FIGURE A1:
SEISMIC WAVE PATHS FOR DIRECT WAVE, REFLECTED WAVE AND REFRACTED WAVE ILLUSTRATING EFFECTS OF A BOUNDARY BETWEEN MATERIALS WITH DIFFERENT ELASTIC PROPERTIES

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.

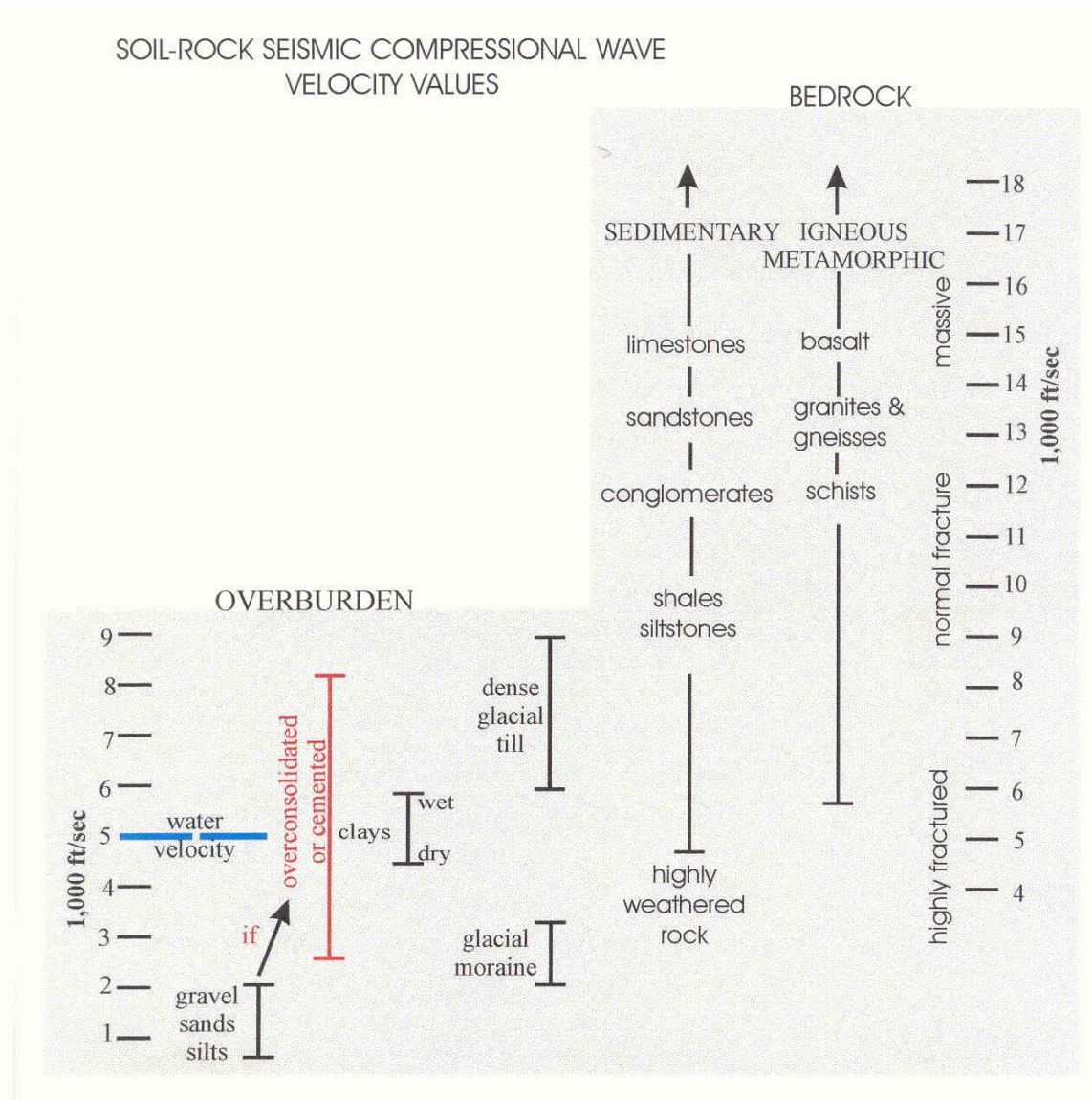


FIGURE A2:
GUIDE TO MATERIAL IDENTIFICATION BY P-WAVE VELOCITY

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.

EQUIPMENT:

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

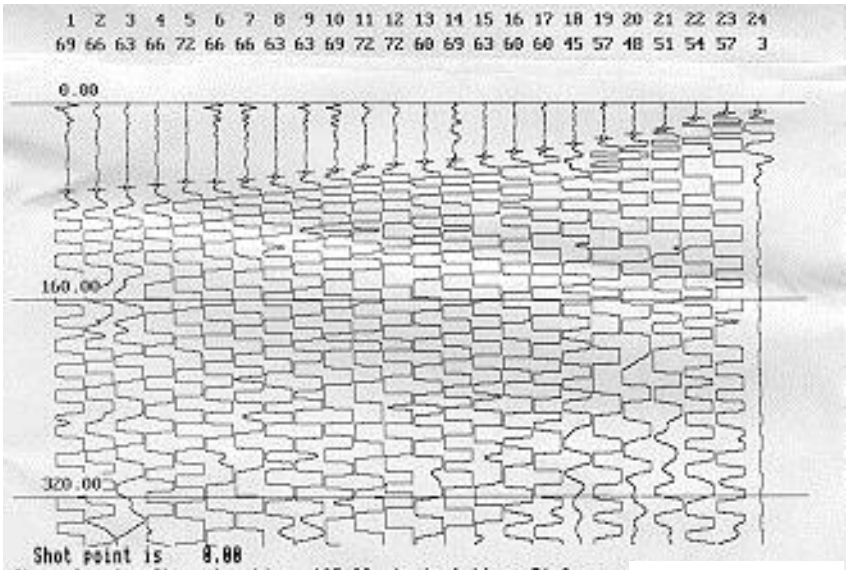
building where vibration-sensitive manufacturing is conducted, data acquisition may be constrained to particular time intervals and appropriate energy sources must be used. Over head and buried utilities must be located an avoided, for both safety and induced electrical noise concerns. Since the seismic 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.

DATA PRESENTATION AND INTERPETATION:

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.



Shot point is 0.00

Channel	1	Phone location	195.00	Arrival time	71.3	msec
Channel	2	Phone location	190.00	Arrival time	70.3	msec
Channel	3	Phone location	185.00	Arrival time	68.5	msec
Channel	4	Phone location	175.00	Arrival time	66.3	msec
Channel	5	Phone location	165.00	Arrival time	63.0	msec
Channel	6	Phone location	155.00	Arrival time	63.0	msec
Channel	7	Phone location	145.00	Arrival time	62.0	msec
Channel	8	Phone location	135.00	Arrival time	59.0	msec
Channel	9	Phone location	125.00	Arrival time	56.0	msec
Channel	10	Phone location	115.00	Arrival time	54.0	msec
Channel	11	Phone location	110.00	Arrival time	51.5	msec
Channel	12	Phone location	105.00	Arrival time	49.3	msec
Channel	13	Phone location	95.00	Arrival time	47.3	msec
Channel	14	Phone location	90.00	Arrival time	45.0	msec
Channel	15	Phone location	85.00	Arrival time	43.0	msec
Channel	16	Phone location	75.00	Arrival time	39.0	msec
Channel	17	Phone location	65.00	Arrival time	37.0	msec
Channel	18	Phone location	55.00	Arrival time	33.0	msec
Channel	19	Phone location	45.00	Arrival time	28.0	msec
Channel	20	Phone location	35.00	Arrival time	25.0	msec
Channel	21	Phone location	25.00	Arrival time	20.0	msec
Channel	22	Phone location	15.00	Arrival time	12.0	msec
Channel	23	Phone location	10.00	Arrival time	8.0	msec
Channel	24	Phone location	5.00	Arrival time	7.0	msec

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

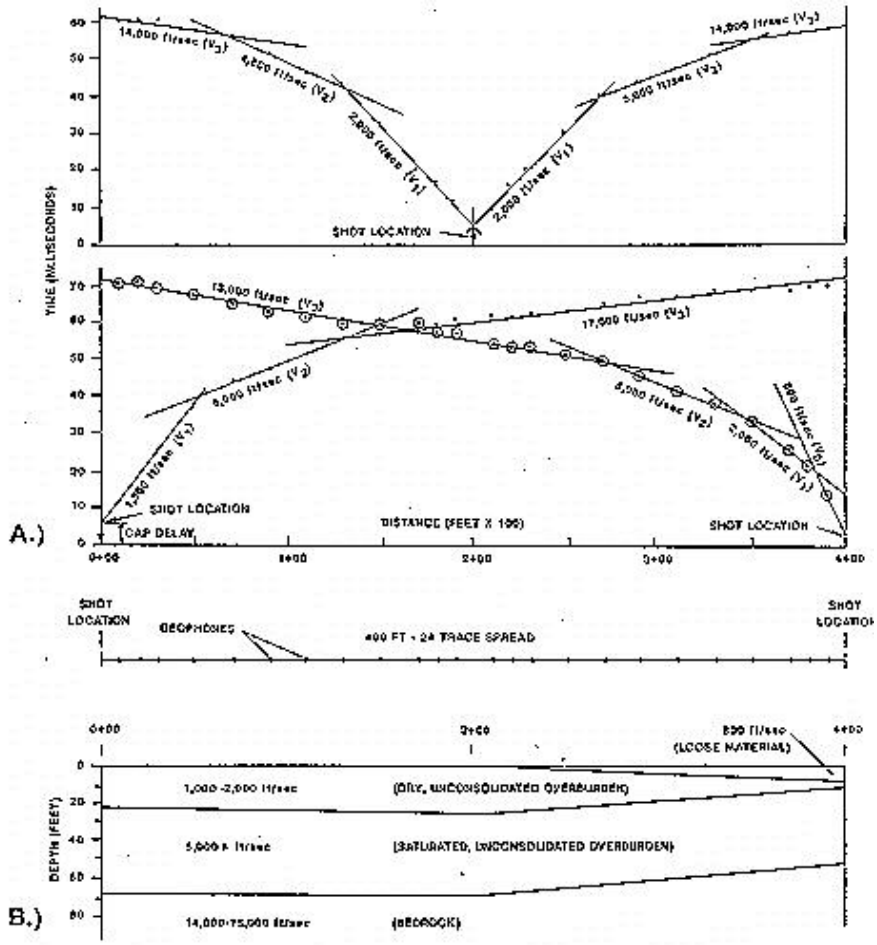


FIGURE A4:
A: TRAVEL-TIME PLOTS; UPPER PLOT IS A CENTER SHOT, LOWER PLOT IS TWO END SHOTS
B: RESULTING PROFILE OF SUBSURFACE MATERIALS SHOWING INTERFACE BETWEEN DIFFERENT SEISMIC VELOCITY LAYERS

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.