Geotechnical Report
For
Retail Store at Dover Village
Dover Plains, New York

File No. 2247

Prepared For:

Southern Reality Development

Prepared By:

Daniel G Loucks, PE
NYSPE 068389

13 January 2011

INTRODUCTION:

The subsurface investigation for the proposed Retail Building at Dover Village, Dover Plains, New York has been completed. Soil & Material Testing Inc. of Castleton, New York has completed six (6) soil borings at the site. In addition to the soil borings three (3) cone penetration (CPT) tests and one (1) shear wave velocity test have been performed at the site by Conetec Inc. of West Berlin, NJ to provide data for the seismic recommendations. Ten (10) test pits were excavated at the site. The logs of these borings, test pits and the cone penetration test results, along with a location diagram, have been included in the appendix of this report.

It is my understanding that the proposed construction will include a single-story building located approximately as indicated on the boring location diagram. The building will have a block bearing wall and steel frame design.

The maximum column loadings will range from 75 to 125 kips. Bearing wall loads will range from 2 to 4 kips per foot of wall. The settlement tolerances are normal. Settlement tolerances are considered to include up to 1 inch of total settlement and 3/4 inch of differential settlement between column locations.

The first floor slab will be established at approximately 1 foot above the existing ground surface.

The purpose of this report is to describe the investigation conducted and the results obtained; to analyze and interpret the data obtained; and to make recommendations for the design and construction of the feasible foundation types and earthworks for the project.

The scope of my services has been limited to coordinating the boring and laboratory investigation, analyzing the soils information, and providing a geotechnical report with foundation recommendations, seismic site classifications as per NYS Building Code. Environmental aspects of the project as well as grading, pavements and site design should be performed by qualified others.

FIELD INVESTIGATION PROCEDURES:

The borings were extended by means of 3.25 inch ID, hollow-stem, augers and by continuous sampling with a split-spoon sampler.

Representative samples were obtained from the boring holes by means of the split-spoon sampling procedure performed in accordance with ASTM D 1586. The standard penetration values obtained from this procedure have been indicated on the soil boring logs.

Soil samples obtained from these procedures were examined in the field, sealed in containers, and shipped to the laboratory for further examination, classification and testing, as applicable.

During the investigation, water level readings were obtained at various times where water accumulated in the boring hole. The water level readings, along with an indication of the time of the reading relative to the boring procedure, have been indicated on the soil boring logs.

The cone penetration tests (CPT) were performed with a 60 degree cone with a base area of 15 cm^2 and a 225 cm^2 friction sleeve. The CPT was performed in accordance with ASTM Specification D3441. Shear wave velocity testing was also performed at 1 meter intervals during the cone tests. The results of the cone tests and shear wave velocity tests can be found in the appendix of this report.

In addition to the field boring investigation, the soil engineer visited the site to observe the surface conditions and the excavation of the test pits at the site. Test pits were excavated with a track mounted excavator.

LABORATORY INVESTIGATION:

All samples were examined in the laboratory by the soil engineer and classified according to the Unified Soil Classification System. In this system, the soils are visually classified according to texture and plasticity. The appropriate group symbol is indicated on the soil boring logs.

Samples exhibiting significant percentages of fine-grained soils or organic materials were subjected to moisture content testing. This testing was performed in accordance with ASTM D 2216-71. The results of these tests have been included in the appendix of the report.

Samples exhibiting significant cohesion were tested with a calibrated, spring-loaded, penetrometer. This test is used to estimate the unconfined compressive strength of the soil sample by measuring the soil's resistance to the penetration of the penetrometer needle. The results of these tests are listed on the boring logs.

SITE CONDITIONS:

At the time of my site visit the ground surface in the proposed building area sloped very gently down to the north. To the east and to the west of the proposed building, the ground surface sloped more steeply upward to a railroad line to the east and Route 22 to the west.

I understand that to the north of the proposed building there was an old municipal dump that has been filled. I understand that the dump was filled in the 1970s and that the garbage was burned before burying it. The ground surface in this area is slightly lower, 1 to 3 feet, than in the building area and adjacent areas to the east and west. This likely indicates settlements that have occurred since the dump was filled. Borings and test pits in the dump area also encountered peat under the garbage fill. The consolidation of the peat likely also contributed to the settlements observed.

SUBSURFACE CONDITIONS:

The specific subsurface conditions encountered at each boring and test pit location are indicated on the individual soil boring and test pit logs. However, to aid in the evaluation of this data, I have prepared a generalized description of the soil conditions based on the boring and test pit data.

Borings and test pits were originally performed at the site to investigate the subsurface conditions for a larger building footprint. Since that investigation, the building size has been reduced and shifted somewhat to the south. The cone penetration work was performed in this area to provide additional subsurface information and more accurate seismic parameters. Test pits 1, 2, 13, and 14 were logged by others.

Some of borings and test pits in the proposed store location encountered an upper layer of topsoil that extended to between approximately 0.5 and 1.0 feet. Boring 4 encountered an upper layer of uncontrolled fill/topsoil that extended to approximately 2.5 feet below the existing ground surface. This uncontrolled fill is comprised of a mixture of sand and clayey silt with a trace of gravel and coal.

Beneath the topsoil, and at the other test pit locations a layer of gravel and sand with varying amounts silt was encountered.

This gravel and sand is loose to medium dense and extended to between approximately 1.2 and 8.0 feet. It extended to the bottom of test pit 11 at 6.5 feet.

Underlying the gravel and sand soils and the uncontrolled fill soils in boring 4, is generally layered silt/clayey silt with sand soils This layered soils are loose and extend to between approximately 1.2 and 9.0 feet and the bottom of test pits 5, 9, and 12.

Below the silt/clayey silt with sand is a layer of clayey silt with occasional thin clay layers. Thin sand layers were also observed in boring 5. This layered clayey silt is loose and extended to the bottom of the borings at 12.0 feet and the bottom of test pit 10 at 9.0 feet.

Borings 1, 2, and 6, and test pits 3, 4, 6, 7 and 8 were performed in the old building footprint which is now shown to be a parking lot area.

Test pit 8 was the only location, in this area, that didn't encounter any uncontrolled fill. In general test pit 8 encountered similar soil conditions to the borings and test pits in the proposed building area, sand and gravel soils over clayey silt with thin sand and clay layers.

The other test pits and borings, in this area, encountered an upper layer of uncontrolled fill that extended to between approximately 2.0 and 7.5 feet. This upper layer of uncontrolled fill consists of a mixture of sand and gravel with a trace to some silt/clayey silt and a trace of coal, cinders, ash, brick and roots. This layer is loose to medium dense.

Below the upper layer of uncontrolled fill in borings 2 and 6, and test pits 3, 4 and 6 is a layer of debris fill that includes glass and metal with lesser amounts of wood, sand and gravel. This layer is loose and extends to between approximately 3.5 and 10.0 feet.

A layer of peat with a trace to a trace to some wood was encountered below the debris fill. This peat is loose and extended to the bottom of test pit 3 at 11.0 feet, 8.0 feet in test pit 4 and 10.0 feet in test pit 6. In boring 2 the peat extended to approximately 15.0 feet, with a layer of slightly organic silt below it that extended to approximately 18 feet.

Underlying the uncontrolled fill and organic soils, in the borings and test pit 6, is clayey silt with occasional clay and fine sand layers. This layered clayey silt extended to the bottom of test pit 6 at 11.0 feet and the bottom of the borings at between 12.0 and 53.0 feet. This clayey silt is loose. In test pits 4 and 7 a layer of sand and gravel with varying amounts of clayey silt was encountered below the uncontrolled fill and organics. This sand and gravel extended to the bottom of the test pits at between 7.0 and 8.5 feet.

The three cone penetration tests that were performed at the site confirm that the upper soils are similar to those encountered in the boring and test pit investigation performed in the new building footprint.

The cone testing indicated that the layered silt/clayey silt with occasional sand layers extend to approximately 15 to 25 feet. Below the silt/clayey silt are layers of sandy silt with layers of sand. This layered material extended to approximately 60 feet. Gravelly sand and sand layers were generally encountered below 60 feet and extended to the end of the cone probe at a depth of 95.5 feet where refusal was encountered.

Seismic cone penetration testing was also performed at the site to determine the shear wave velocity of the soils in the upper 95.5 feet. The shear wave velocities were used to determine the seismic site classification and in the liquefaction analysis. The results of this testing can be found in the appendix of this report.

GROUNDWATER CONDITIONS:

Accurate groundwater levels are difficult to determine in clayey silt soils with only short term readings or observations. Clayey silt soils typically do not allow an adequate amount of water to flow through the soil to produce a water level reading during the drilling operation and the test pit investigation. I have indicated where water was observed on the boring and test pit logs.

Based on the groundwater levels observed during the boring and test pit investigation, the moisture condition of the samples recovered from the boring holes and coloration of the soil samples, I judge that the groundwater level was located below depth of 2.5 feet.

Perched groundwater tables may occur at higher elevations in the soil profile due to groundwater being retained by layers or lenses of silt or clay soils. Perched or seasonal groundwater levels are sometimes indicated by mottled brown/gray soils. These soil conditions were observed as shallow as 1.5 feet below the existing ground surface.

Some fluctuation in hydrostatic groundwater levels and perched water conditions should be anticipated with variations in the seasonal rainfall and surface runoff.

It should be noted that the groundwater levels were obtained during the drilling procedure and test pit excavations. Actual water levels may vary at the time of construction. Some groundwater could be encountered in soil layers labeled moist to wet on the boring logs.

ANALYSIS AND RECOMMENDATIONS:

Site Work:

The proposed building and sidewalk areas should be cleared and grubbed and all organic topsoil and vegetation along with any uncontrolled fill and debris should be stripped from the site. The soil engineer should observe the stripped subgrade to determine if all the uncontrolled fill has been removed. The subgrade should be proof-rolled with a 10-ton roller. This proof rolling will compact the subgrade and reveal the presence of soft spots. If saturated subgrade conditions exist, I recommend that the subgrade be observed and probed by the soil engineer in place of proof rolling. Any soft spots should be excavated and backfilled with controlled fill material.

The removal of any uncontrolled fill should extend to a minimum horizontal distance past the edge of the footings equal to the depth that the fill extends under the footing. This is equal to a 1:1 slope down from the outer edge of the footing to the virgin soil. All uncontrolled fill within the proposed building area should also be removed.

A way to stabilize a spongy, but suitable, virgin, subgrade would be to spread a reinforcement or separation type of geotextile on the subgrade and follow with a lift of clean, granular fill or stone. The thickness of the controlled fill can range from 1.0 to 2.5 feet, as necessary, to achieve a working mat upon which to construct the remainder of the controlled fill or to place footings. If open graded stone is used as controlled fill a layer of geotextile should be placed between the stone and any sand/gravel controlled fill or virgin soil.

A third method for stabilizing spongy areas of the subgrade would be to improve the drainage by use of properly designed drain tiles or by using properly designed sump pit and pump dewatering systems. Using these methods, the local groundwater table maybe able to be lowered sufficiently to aid in stabilizing the subgrade surface. If large quantities of water are encountered vacuum well point dewatering maybe required.

Controlled Fill:

Before any controlled fill is placed the site should be inspected to verify that the site has been prepared according to the recommendations contained in this report as required by the NYS Building Code Section 1704.7.1.

Controlled, relatively clean, granular fill can be spread in lifts not exceeding 12 inches in loose thickness. These materials should be compacted to a minimum of 95 percent of the maximum ASTM Specification D 1557-91 density, modified proctor.

Some on-site material may be difficult to compact during wet weather or poor drying conditions. Given good drying conditions, the on-site soils with more than 10 percent silt/clayey silt could be compacted using disc harrows and sheepsfoot rollers or rubber-tired rollers, as applicable. These types of soils are sensitive to moisture content and weather conditions. During freezing or wet weather conditions these materials may not be able to be adequately compacted for use as structural fill.

If crushed stone is used as controlled fill it should have a layer of geotextile with a minimum tensile strength of 200 lbs should be placed between the stone and existing soils. The stone should be placed in lifts not exceeding 12 inches in thickness and should be compacted with a minimum of 5 passes of a vibratory roller rated at 5 tons or larger.

Free Draining Controlled Fill Material: Naturally or artificially graded mixture of sand, natural or crushed stone or gravel conforming to NYS DOT Item 304-2.03, Type 4 or 2 as follows:

U.S. Sieve No.	Percent Passing by Weight
2 inch	100
1/4 inch	35-60
No. 40	5-40
No. 200	0-10

NYS DOT Table 703-4, Size 2 crushed stone, clean, durable, angular, and of uniform quality throughout:

U.S. Sieve No.	Percent Passing by Weight
1 ½ inch	100
1 inch	90-100
1/2 inch	0-15

All controlled fill should be free of organic and/or frozen material.

Free-draining controlled fill should have less than 10 percent fines passing the #200 sieve.

I recommend performing one field density test for every 2,000 square feet of controlled fill placed, within the overlaying building footprint, but in no case fewer than three tests per lift.

I recommend that for foundation wall and footing backfill that in each compacted backfill layer have at least one field in place density test for each 50 feet or less of wall or footing length, but not fewer than two tests along a wall face or footing be performed per lift.

Building Foundations:

I recommend that the proposed structure be supported by spread footing foundations resting on virgin, inorganic, soils or on controlled fill which, in turn, rests on these virgin materials. Footings can be designed for a maximum, net, allowable soil bearing pressure of 2500 psf.

The soil engineer should observe the footing subgrade at the beginning of the project or if soil conditions change to verify the allowable bearing pressure of the soil encountered.

Loads from adjacent footings or structures should be assumed to distribute based on the elastic theory. Typical Boussinesq charts can be used to approximate loads at various depths and locations due to adjacent structures.

A minimum footing width of 2.0 feet is recommended for load bearing strip footings. Isolated footings should be at least 3.0 feet wide.

Exterior footings or footings in unheated areas should have a minimum of 4.0 feet of embedment for protection from frost action. Interior footings should have a minimum embedment of 2.0 feet below finished grade to develop the bearing value of the soils.

All walls that retain soil on only one side should have a drain tile placed along the base of the wall. The drain tile should be a minimum of 4 inches in diameter, surrounded by a minimum of 6 inches of properly graded washed sand or crushed stone wrapped with a non woven filter fabric with a maximum apparent opening size of 70 and a minimum trapezoid tearing strength of 100 lbs. The drain tile should drain to a stormwater sewer, daylight, or a sump equipped with a pump.

The wall should then be backfilled with a controlled, well graded, free-draining granular material. The material should extend away from the wall a horizontal distance of two-thirds the height of the fill being placed. The upper 1 foot of material should be a fairly impermeable material to shed surface water.

If these procedures are used, a static lateral soil pressure of 40 psf per foot of retained soil can be used for design of the wall. This static, active lateral soil pressure is based on a moist unit weight of 125 pcf and an angle of internal friction of 32 degrees. A wall soil friction angle of 18 degrees and a coefficient of base sliding of 0.35 can also be used for design.

If the retaining wall is braced or if the deflection is limited prior to backfilling so the active soil pressure is not achieved, a static, at-rest lateral soil pressure of 63 psf per foot of retained soil can be used for design.

To resist overturning and sliding a static lateral passive pressure of 250 psf per foot of embedment can be used, provided foundations are backfilled with controlled fill. This static, passive pressure resistance value has been reduced from the calculated full passive pressure because of stress/strain characteristics of the soil. To develop the full, calculated resistance a certain amount of movement or deflection in the structure is required. The amount of movement required to generate this resistance generally greater then is acceptable for structures. I therefore recommend that the full passive pressure not be used.

The passive resistance of the exterior upper two feet of soil should be ignored due to surface effects of frost and moisture.

Any surcharge loading of existing adjacent building foundations or other adjacent structures/utilities should be addressed by the structural engineer using Boussinesq charts.

Floor Slabs:

Concrete floor slabs can be designed to rest on controlled fills resting on virgin materials. A 6-inch layer of well-graded, free-draining, granular material should be placed beneath the floor slab to provide drainage, act as a capillary break, and to provide better and more uniform support.

If vehicle loadings are to be applied to the floor slab, the proposed slab and supporting soils should be analyzed as a pavement structure. I recommend that a minimum of 12 inches of free draining controlled granular fill be placed below any concrete pavements.

A modulus of subgrade reaction of 150 psi per inch can be used to design concrete slabs resting on a minimum of 6 inches of free draining controlled fill that in turn rests on virgin soils. A modulus of subgrade reaction of 175 psi per inch can be used to design concrete slabs resting on a minimum of 12 inches of free draining controlled fill that in turn rests on virgin soils. A modulus of subgrade reaction of 125 psi per inch can be used to design exterior slabs or pavements resting on a minimum of 12 inches of free draining controlled fill. This reduced value is recommended due to seasonal variations that occur due to frost in the soils.

Exterior concrete pavements may experience some frost heave movements during the winter and spring. If these movements are not acceptable then a minimum of 4.0 feet of approved subbase material and properly designed drains would be required below the concrete pavements or sidewalks. The use of properly designed footing drains can also be used to reduce possible frost heave movements adjacent to the proposed structure.

If the moisture levels of floor slab areas are critical additional drainage materials and vapor barriers will be required beneath the floor slab. Also the moisture content of the subbase soils should be carefully monitored to prevent excess water from saturating these subbase soils before the floor slab is poured. This aspect of the design should be performed by qualified others.

Seismic Conditions:

The potential seismic conditions at the proposed site have been investigated using the information provided The NYS Building Code Section 1613 and 18 and the seismic cone penetration testing information obtained during my investigation. The seismic testing was performed to directly measure the shear wave velocity of the existing soils. In my opinion this is a more accurate measure of the soil seismic parameters during a seismic event than the standard penetration values.

Based on the shear wave velocity data obtained during the CPT testing it is my opinion that the Site Classification (Table 1615.1.1) could be assumed to be D. Using figures 1615 (1 and 2), and the data from the USGS Hazards Mapping and the USGS Open File Report 01-437, I estimate that the MCE spectral acceleration (S_{MS}) at short periods is 42.1 and the MCE spectral acceleration (S_{M1}) at 1 s period is 15.9.

The probabilistic ground motion values are expressed in %g for rock site class B. Peak ground accelerations in the upper soil profile may vary. If it is determined by the structural engineer that the Seismic Design Category is D, E or F additional geotechnical recommendations can be provided.

A copy of the USGS Seismic Hazard Mapping has been included in the appendix of this report to provide additional information if required.

The soil samples, cone penetration seismic test results and my analysis do not indicate any significant potential seismic hazards such as liquefaction, sensitive clays, weakly cemented soil or surface rupture. The seismic settlement analysis indicates that up to approximately 0.5 inches of seismically induced settlement could be experienced if the design 6.0 Mw earthquake was to occur

CONSTRUCTION PROCEDURES AND PROBLEMS:

The NYS Building Code Section 17 requires special inspections and follow up reports. These inspections should be performed to verify compliance with the recommendations contained in this report.

All excavations of more than a few feet should be sheeted and braced or laid back to prevent sloughing in of the sides.

Excavations should not extend below adjacent footings or structures unless properly designed sheeting and bracing or underpinning is installed.

Footing and floor slab subgrades should be tamped to compact any soil disturbed during the excavation process. A flat plate should be placed on the end of the excavator or backhoe bucket to reduce disturbance of the footing subgrade. If over excavation of subgrades are required to remove cobbles or possibly boulders, then the over excavated areas should be filled with controlled granular fill or lean concrete.

A layer of geotextile (min. tensile strength of 200 lbs) and 4 to 8 inches of crushed stone may be required in footing excavations to prevent disturbance of the virgin subgrade during wet weather. The stone and fabric should be placed as described in the *Controlled Fill* section of this report.

Sump-pit and sump-pump-type dewatering may be required in excavations or low areas during wet weather or if groundwater is encountered. If large quantities of groundwater are encountered vacuum wells maybe required to stabilize the subgrade soils. All dewatering programs should be designed to prevent bottom heave. Any dewatering program should be performed with properly designed filtration protection on all pumps to prevent loss of ground.

Subgrades should be kept from freezing during construction.

Water, snow, and ice should not be allowed to collect and stand in excavations or low areas of the subgrade.

Some obstacles, including possibly cobbles, may be encountered in excavations.

Design and construction procedures should include measures to limit the potential for slab curl and vapor transmission. The shrinkage properties of the concrete should be controlled and the curing of the concrete controlled. Differential shrinkage between the top and bottom of the slabs could otherwise result in curling of the slabs. The control of vapor transmission through the slab should also be addressed. These phenomena may be only indirectly related to soil conditions. The architect/engineer should address this aspect of the design.

Current American Concrete Institute recommendations for the design and construction of floor slabs and the control of shrinkage, slab curl and vapor transmission can be referred to.

Dover Village Retail Building
Dover Plains, New York
File No. 2247

CONTENTS OF APPENDIX:

GENERAL NOTES

DRILLING & SAMPLING SYMBOLS

SS : Split-Spoon — 1¾ " I.D., 2" O.D., except where noted
S : Shelby Tube — 2" O.D., except where noted
PA : Power Auger Sample
DB : Diamond Bit — NX: BX: AX:
CB : Carboloy Bit — NX: BX: AX:
OS : Osterberg Sampler — 3" Shelby Tube
HS : Housel Sampler
WS : Wash Sample
FT : Fish Tail
RB : Rock Bit
WO : Wash Out

Standard "N" Penetration: Blows per foot of a 140 pound hammer falling 30 inches on a 2 inch OD split spoon, except where noted

WATER LEVEL MEASUREMENT SYMBOLS

WL : Water Level
WCI : Wet Cave In
DCI : Dry Cave In
WS : While Sampling
WD : While Drilling
BCR : Before Casing Removal
ACR : After Casing Removal
AB : After Boring

Water levels indicated on the boring logs are the levels measured in the boring at the times indicated. In pervious soils, the indicated elevations are considered reliable ground water levels. In impervious soils the accurate determination of ground water elevations is not possible in even several day's observation, and additional evidence on ground water elevations must be sought.

CLASSIFICATION

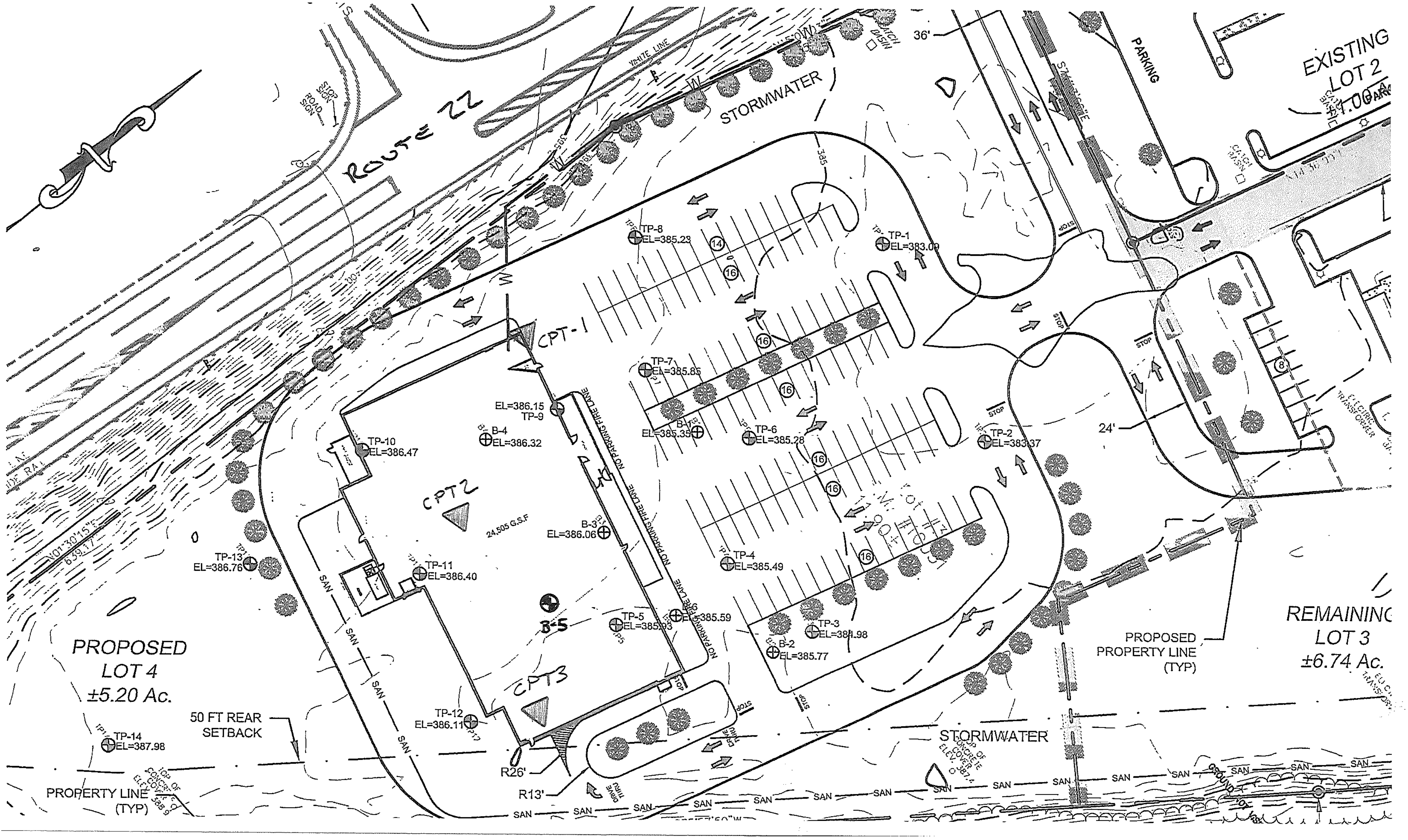
COHESIONLESS SOILS

"Trace"	: 1% to 10%	
"Trace to some"	: 10% to 20%	
"Some"	: 20% to 35%	
"And"	: 35% to 50%	
Loose	: 0 to 9 Blows	or equivalent
Medium Dense	: 10 to 29 Blows	or equivalent
Dense	: 30 to 59 Blows	or equivalent
Very Dense	: ≥60 Blows	or equivalent

COHESIVE SOILS

If clay content is sufficient so that clay dominates soil properties, then clay becomes the principle noun with the other major soil constituent as modifiers: i.e., silty clay. Other minor soil constituents may be added according to classification breakdown for cohesionless soils; i.e., silty clay, trace to some sand, trace gravel.

Soft	: 0.00 —0.59 tons/ft²
Medium	: 0.60 — 0.99 tons/ft²
Stiff	: 1.00 — 1.99 tons/ft²
Very Stiff	: 2.00 — 3.99 tons/ft²
Hard	: ≥ 4.00 tons/ft²


EXISTING
LOT 2
PARKING
CATCH BASIN
ELECTRIC TRANSFORMER
36'
24'
STORMWATER
ROUTE 22
STOP SIGN
ROAD SIGN
WHITE LINE
TP-1 EL=383.09
TP-2 EL=383.37
TP-3 EL=384.98
TP-4 EL=385.49
TP-5 EL=385.93
TP-6 EL=385.28
TP-7 EL=385.85
TP-8 EL=385.23
EL=386.15 TP-9
TP-10 EL=386.47
TP-11 EL=386.40
TP-12 EL=386.11
TP-13 EL=386.76
TP-14 EL=387.98
B-2 EL=385.77
B-3 EL=386.06
B-4 EL=386.32
B-5
B-6 EL=385.59
EL=385.35
CPT-1
CPT2
CPT3
24,505 G.S.F
NO PARKING FIRE LANE
STOP
DRIVE THRU
R26'
R13'
SAN
PROPOSED
LOT 4
±5.20 Ac.
50 FT REAR SETBACK
PROPERTY LINE (TYP)
PROPOSED PROPERTY LINE (TYP)
REMAINING
LOT 3
±6.74 Ac.
TOP OF CONCRETE COVER ELEV. 387.4
TOP OF CONCRETE ELEV. 388.9

PROJECT NAME: Dover Village Store

FILE NUMBER: 2247

LOCATION: Dover Plains, New York

OFFSET: None

DATE STARTED/COMPLETED: December 2010

SURFACE ELEV.: 385.4 ft

ENGINEER/ARCHITECT: Rennia Engineering

DRILL CONTRACTOR: Soil & Material Testing Inc

DRILLING METHOD: Hollow Steam Auger

DRILL RIG TYPE: Truck Mount

HAMMER WEIGHT: 140 Lbs

DROP: 30 Inches

CASING DIAMETER: OD/ID: 3.25 inchID

WATER LEVEL DEPTH: 5.5 ft **TIME:** ACR

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DEPTH	Sample Number	Sample Type	BLOW COUNTS per 6 inches	"N" Value	Recovery
1–2	1	SS	2-3-6-3	9	
3–4	2	SS	1-1-1-2	2	
5–6	3	SS	3-4-3	7	
7–8	4	SS	4-6-12-6	18	
9–10		PA	4-6-12-6		
11–12	5	SS	4-5-5-6	10	
13–15		PA			
16–17	6	SS	4-6-6-5	12	
18–20		PA			
21–22	7	SS	WRH-2-2	2	
23–25		PA			
26–27	8	SS	WRH-2-2	2	

DESCRIPTION

Topsoil

Fine to Coarse Sand, some Gravel, trace Silt, Brown, Moist, Loose (SM) FILL

Fine to Coarse Sand, trace to some Gravel, Silt, trace Coal, Brick, Brown, Moist, Loose (SM) FILL

Clayey Silt and Sand, trace to some Gravel, trace Roots, Brown/Gray, Moist to Wet, Loose (ML-SM) FILL

Medium to Coarse Sand and Gravel, trace Silt, Roots, Brown, Wet, Medium Dense (SM-SP)(GM-GP) Possible Fill

Clayey Silt and Sand, Gray, Wet, Medium Dense (ML)(SM) Layered

Clayey Silt, trace to some Clay, Gray, Wet, Loose/Soft (ML)(CL) Occasional Thin Clay Layers

BORING NO: 1

PROJECT NAME: Dover Village Store

LOCATION: Dover Plains, New York

DATE STARTED/COMPLETED: December 2010

ENGINEER/ARCHITECT: Rennia Engineering

DRILLING METHOD: Hollow Steam Auger

DRILL RIG TYPE: Truck Mount

HAMMER WEIGHT: 140 Lbs

DROP: 30 Inches

CASING DIAMETER: OD/ID: 3.25 inchID

WATER LEVEL DEPTH: 5.5 ft **TIME:** ACR

FILE NUMBER: 2247

OFFSET: None

SURFACE ELEV.: 385.4 ft

DRILL CONTRACTOR: Soil & Material Testing Inc

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DEPTH	Sample Number	Sample Type	BLOW COUNTS per 6 inches	"N" Value	Recovery	DESCRIPTION
28–30		PA				Clayey Silt, trace to some Clay, Gray, Wet, Loose/Soft (ML)(CL) Occasional Thin Clay Layers
30–32	9	SS	WRH-1-2-2	3		
32–35		PA				
35–37	10	SS	WRH-1-1-2	2		
37–40		PA				
40–42	11	SS	WRH-2-2-2	4		
42–45		PA				
45–47	12	SS	WRH-1-2-2	3		
47–48		PA				
48–50	13	SS	WRH-2-3-2	5		
50–54						End of Boring at 50.0 Feet

BORING NO: 2

SHEET 1 of 1

PROJECT NAME: Dover Village Store

LOCATION: Dover Plains, New York

DATE STARTED/COMPLETED: December 2010

ENGINEER/ARCHITECT: Rennia Engineering

DRILLING METHOD: Hollow Steam Auger

DRILL RIG TYPE: Truck Mount

HAMMER WEIGHT: 140 Lbs

DROP: 30 Inches

CASING DIAMETER: OD/ID: 3.25 inchID

WATER LEVEL DEPTH: 4.0 ft **TIME:** ACR

FILE NUMBER: 2247

OFFSET: None

SURFACE ELEV.: 385.8 ft

DRILL CONTRACTOR: Soil & Material Testing Inc

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DEPTH	Sample Number	Sample Type	BLOW COUNTS per 6 inches	"N" Value	Recovery	DESCRIPTION
0-2	1	SS	3-5-8-9	13		Topsoil Fine to Coarse Sand, some Gravel, trace to some Silt, Brown, Moist, Medium Dense (SM) FILL
2-4	2	SS	6-6-13-12	19		Fine to Coarse Sand and Gravel, trace to some Silt, Gray, Moist, Medium Dense (SM-GM) FILL
4-6	3	SS	4-4-1-0	5		Fine to Coarse Sand, some Gravel, Clayey Silt, Gray, Moist to Wet, Loose (SM) FILL
6-8	4	SS	1-0-1-1	1		Fine to Coarse Sand, some Gravel, trace to some Glass, Metal, Wood, Gray, Wet, Loose (SM-SP) FILL
8-10	5	SS	1-1-1-1	2		
10-12	6	SS	2-3-7-3	10		Peat, trace to some Wood, Dark Brown, Wet, Medium Dense (Pt)
12-15		PA				
15-17	7	SS	1-0-1-1	1		Organic Silt, Gray, Wet, Loose (OL)
17-20		PA				Silt, trace to some Clay, Gray, Wet, Loose/Soft (ML)(CL) Occasional Thin Clay Layers
20-22	8	SS	1-2-2-3	4		
22-27						End of Boring at 22.0 Feet

BORING LOG

BORING NO: 3

SHEET 1 of 1

PROJECT NAME: Dover Village Store

LOCATION: Dover Plains, New York

DATE STARTED/COMPLETED: December 2010

ENGINEER/ARCHITECT: Rennia Engineering

DRILLING METHOD: Hollow Steam Auger

DRILL RIG TYPE: Truck Mount

HAMMER WEIGHT: 140 Lbs

DROP: 30 Inches

CASING DIAMETER: OD/ID: 3.25 inchID

WATER LEVEL DEPTH: 4.5 ft **TIME:** ACR

FILE NUMBER: 2247

OFFSET: None

SURFACE ELEV.: 386.1 ft

DRILL CONTRACTOR: Soil & Material Testing Inc

Daniel G Loucks PE
PO Box 163
Ballston Spa, New York 12020
Phone: 518-371-7622
Fax: 518-383-2069

DEPTH	Sample Number	Sample Type	BLOW COUNTS per 6 inches	"N" Value	Recovery	DESCRIPTION
0-2	1	SS	2-13-9-18	22		Topsoil; Fine to Coarse Sand, some Gravel, trace Silt, Brown, Moist, Medium Dense (SM-SP)
2-4	2	SS	4-4-3-3	7		Medium to Coarse Sand, trace to some Gravel, trace Silt, Brown, Moist, Loose (SM-SP)
4-6	3	SS	5-6-11-11	17		No Recovery
6-8	4	SS	9-6-6-11	12		Fine to Coarse Sand, some Silt, trace to some Gravel, Brown, Wet, Medium Dense (SM) Driller Notes Cobbles
8-10	5	SS	6-6-6-7	12		Clayey Silt, trace to some Clay, Gray, Wet, Loose to Medium Dense/Soft (ML)(CL) Occasional Clay Layers
10-12	6	SS	3-4-4-5	8		
12						End of Boring at 12.0 Feet

BORING NO: 4

PROJECT NAME: Dover Village Store

LOCATION: Dover Plains, New York

DATE STARTED/COMPLETED: December 2010

ENGINEER/ARCHITECT: Rennia Engineering

DRILLING METHOD: Hollow Steam Auger

DRILL RIG TYPE: Truck Mount

HAMMER WEIGHT: 140 Lbs

DROP: 30 Inches

CASING DIAMETER: OD/ID: 3.25 inchID

WATER LEVEL DEPTH: 10.0 ft **TIME:** WS

FILE NUMBER: 2247

OFFSET: None

SURFACE ELEV.: 386.3 ft

DRILL CONTRACTOR: Soil & Material Testing Inc

Daniel G Loucks PE
PO Box 163
Ballston Spa, New York 12020
Phone: 518-371-7622
Fax: 518-383-2069

DEPTH	Sample Number	Sample Type	BLOW COUNTS per 6 inches	"N" Value	Recovery	DESCRIPTION
1–2	1	SS	3-4-5-5	9		Fine to Medium Sand and Clayey Silt, trace Gravel, Coal, Dark Brown, Moist, Loose (SM-ML) Topsoil FILL
3–4	2	SS	4-6-6-7	12		Fine Sand and Clayey Silt, trace Gravel, Brown, Moist, Medium Dense (SM-ML)
5–6	3	SS	3-3-4-5	7		Clayey Silt, trace Clay, Gray, Wet, Loose (ML)(CL) Occasional Thin Clay Layers
7–8	4	SS	5-4-4-4	8		
9–10		PA				
11–12	5	SS	2-2-2-2	4		
12						End of Boring at 12.0 Feet
13–27						

BORING LOG

BORING NO: 5

SHEET 1 of 1

PROJECT NAME: Dover Village Store

LOCATION: Dover Plains, New York

DATE STARTED/COMPLETED: December 2010

ENGINEER/ARCHITECT: Rennia Engineering

DRILLING METHOD: Hollow Steam Auger

DRILL RIG TYPE: Truck Mount

HAMMER WEIGHT: 140 Lbs

DROP: 30 Inches

CASING DIAMETER: OD/ID: 3.25 inchID

WATER LEVEL DEPTH: 4.0 ft **TIME:** WS

FILE NUMBER: 2247

OFFSET: None

SURFACE ELEV.: 386.1 ft

DRILL CONTRACTOR: Soil & Material Testing Inc

Daniel G Loucks PE
PO Box 163
Ballston Spa, New York 12020
Phone: 518-371-7622
Fax: 518-383-2069

DEPTH	Sample Number	Sample Type	BLOW COUNTS per 6 inches	"N" Value	Recovery	DESCRIPTION
1–2	1	SS	3-4-6-7	10		Topsoil Fine to Coarse Sand and Gravel, trace Silt, Moist, Loose to Medium Dense (SM-SP)(GM-GP)
3–4	2	SS	4-4-3-3	7		
5–6	3	SS	1-1-4-7	5		Fine to Coarse Sand, some Gravel, trace to some Silt, Brown, Wet, Loose (SM-SP)
7–8	4	SS	12-7-4-4	11		Silt, trace to some Fine Sand, Gray, Moist to Wet, Medium Dense (ML)(SM) Fine Sand Layers
9–10		PA				Clayey Silt, trace to some Clay, trace Sand, Gray, Wet, Loose/Soft (ML)(CL)(SM) Occasional Thin Clay and Sand Layers
11–12	5	SS	2-3-2-2	5		
13–27						End of Boring at 12.0 Feet

BORING LOG

BORING NO: 6

SHEET 1 of 1

PROJECT NAME: Dover Village Store

LOCATION: Dover Plains, New York

DATE STARTED/COMPLETED: December 2010

ENGINEER/ARCHITECT: Rennia Engineering

DRILLING METHOD: Hollow Steam Auger

DRILL RIG TYPE: Truck Mount

HAMMER WEIGHT: 140 Lbs

DROP: 30 Inches

CASING DIAMETER: OD/ID: 3.25 inchID

WATER LEVEL DEPTH: 5.0 ft **TIME:** ACR

FILE NUMBER: 2247

OFFSET: None

SURFACE ELEV.: 385.6 ft

DRILL CONTRACTOR: Soil & Material Testing Inc

Daniel G Loucks PE
PO Box 163
Ballston Spa, New York 12020
Phone: 518-371-7622
Fax: 518-383-2069

DEPTH	Sample Number	Sample Type	BLOW COUNTS per 6 inches	"N" Value	Recovery	DESCRIPTION
1	1	SS	2-2-3-4	5		Topsoil; Fine to Medium Sand, trace to some Gravel, Clayey Silt, Ash, Dark Brown, Moist, Loose (SM) FILL
2						
3	2	SS	7-12	19		Driller Notes Concrete, Glass, Cobbles FILL
4		PA				Clayey Silt, trace to some Sand, Wood, trace Gravel, Gray, Moist to Wet, Loose (ML) Possible Fill
5	3	SS	3-2-2-3	4		Clayey Silt and Sand, trace Gravel, Brown, Wet, Loose (ML-SM)
6						
7	4	SS	2-1-2-2	3		
8						
9	5	SS	1-0-1-0	1		Medium to Coarse Sand, trace to some Gravel, trace Silt, Brown, Wet, Loose (SM-SP)
10						
11	6	SS	1-1-2-2	3		Clayey Silt, trace Sand, Brown/Gray, Loose (ML)
12						End of Boring at 12.0 Feet
13						
14						
15						
16						
17						
18						
19						
20						
21						
22						
23						
24						
25						
26						
27						

Test Pit Logs

Retail Building Dover Village

Dover Plains, New York

2 December 2010

Test Pit # 3
0.0 – 4.5 ft Brown/Gray Sand and Gravel, trace to some Silt (SM-GM) FILL
4.5 - 7.0 ft Metal, Wood, Glass FILL
7.0 – 11.0 ft Dark Brown Peat (Pt)
Water Observed at 4.5 Feet

Test Pit # 4
0.0 – 3.5 ft Brown/Gray Sand and Gravel, trace to some Silt (SM-GM) FILL
3.5 – 5.5 ft Metal, Glass, trace to some Wood FILL
5.5 – 8.0 ft Dark Brown Peat (Pt)
8.0 – 8.5 ft Gray Gravel, some Clayey Silt, trace to some Sand (GM)
Water Observed at 4.5 Feet

Test Pit # 5
0.0 – 3.7 ft Brown Gravel and Sand, trace to some Silt, trace Cobbles (GM-SM)
3.7 – 4.2 ft Brown Clayey Silt, Occasional Thin Sand Layers (ML)(SM)
Water Observed at 4.0 Feet

Test Pit # 6
0.0 – 2.0 ft Brown Sand and Gravel, trace to some Silt (SM-GM) FILL
2.0 – 3.0 ft Gray Sand, some Ash, Cinders, trace to some Silt (SM) FILL
3.0 – 7.2 ft Glass, Metal, trace Wood FILL
7.2 – 10.0 ft Dark Brown Peat (Pt)
10.0 – 11.0 ft Gray Silt and Clay Layered (ML)(CL)
Water Observed at 4.0 Feet

Test Pit # 7
0.0 – 2.0 ft Dark Brown Sand, trace to some Gravel, Ash, trace Glass (SM) FILL
2.0 – 2.6 ft Dark Brown Sand, trace to some Silt, Organics (SM) Topsoil
2.6 – 3.0 ft Brown Silt, trace to some Sand (ML)
3.0 – 7.0 ft Brown Gravel and Sand, trace to some Silt (GM-SM)
Water Observed at 6.0 Feet

Test Pit # 8

0.0 – 0.5 ft Dark Brown Sand, some Gravel, trace to some Silt (SM) Topsoil
0.5 – 6.3 ft Brown Sand and Gravel, trace to some Silt (SM-GM)
6.3 – 7.3 ft Brown Clayey Silt, Occasional Thin Sand Layers (ML)(SM)
7.3 – 9.0 ft Gray Clayey Silt, Occasional Clay, Sand Layers (ML)(CL)(SM)

Water Observed at 4.5 Feet

Test Pit # 9

0.0 – 1.2 ft Dark Brown Sand, some Gravel, trace to some Silt (SM) Topsoil
1.2 – 3.5 ft Brown Sand and Gravel, trace to some Clayey Silt (SM-GM)
3.5 – 4.0 ft Brown Clayey Silt, Occasional Thin Sand Layers (ML)(SM)

Water Observed at 3.5 Feet

Test Pit # 10

0.0 – 0.4 ft Dark Brown Sand, some Gravel, trace to some Silt (SM) Topsoil
0.4 – 2.0 ft Brown Sand and Gravel, trace to some Clayey Silt (SM-GM)
2.0 – 4.3 ft Brown Clayey Silt, Occasional Thin Sand Layers (ML)(SM)
4.3 – 9.0 ft Gray Clayey Silt, Occasional Clay, Sand Layers (ML)(CL)(SM)

No Water Observed

Test Pit # 11

0.0 – 6.5 ft Brown Gravel, some Sand, trace Silt, Cobbles (GM-GP)

Water Observed at 4.5 Feet

Test Pit # 12

0.0 – 1.2 ft Brown Gravel and Sand, trace to some Silt (GM-SM)
1.2 – 6.5 ft Brown Clayey Silt, Occasional Thin Sand Layers (ML)(SM)

Water Observed at 1.2 Feet

CONETEC

Job No	11-701
Client	Southern Realty Development
Project Title	Dover Store
Hole	CPT-02
Site	Dover Plains, New York
Date	1/10/2011

Seismic Source:	Beam	
Source Offset:	1.97	(ft)
Source Depth:	0.00	(ft)
Geophone Offset:	0.66	(ft)

SEISMIC TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Depth Interval (ft)	Time Interval (ms)	Mid-layer Depth (ft)	Vs Interval Velocity (ft/s)
2.79	2.13	2.90				
6.07	5.41	5.76	2.86	5.80	3.77	493
9.35	8.69	8.91	3.15	5.23	7.05	603
12.63	11.97	12.13	3.22	4.95	10.33	651
15.91	15.25	15.38	3.25	4.95	13.61	656
19.19	18.53	18.64	3.26	4.67	16.89	698
22.47	21.81	21.90	3.26	4.50	20.17	725
25.75	25.09	25.17	3.27	4.29	23.45	762
29.04	28.38	28.45	3.28	4.29	26.74	765
32.32	31.66	31.73	3.27	4.03	30.02	812
35.60	34.94	35.00	3.27	4.03	33.30	812
38.88	38.22	38.27	3.28	3.65	36.58	897
42.16	41.50	41.55	3.28	2.31	39.86	1418
45.44	44.78	44.83	3.28	2.07	43.14	1583
48.72	48.06	48.10	3.28	1.89	46.42	1734
52.00	51.34	51.38	3.28	2.17	49.70	1510
55.28	54.62	54.66	3.28	2.19	52.98	1497
58.56	57.90	57.94	3.28	2.50	56.26	1311
61.84	61.18	61.22	3.28	2.85	59.54	1150
65.12	64.46	64.49	3.28	2.68	62.82	1223
68.41	67.75	67.78	3.29	2.73	66.11	1205
71.69	71.03	71.06	3.28	2.66	69.39	1233
74.97	74.31	74.34	3.28	2.78	72.67	1179
78.25	77.59	77.62	3.28	2.69	75.95	1219
81.53	80.87	80.90	3.28	2.61	79.23	1256
84.81	84.15	84.18	3.28	2.70	82.51	1214
88.09	87.43	87.46	3.28	2.42	85.79	1355
91.37	90.71	90.74	3.28	2.67	89.07	1228
94.65	93.99	94.01	3.28	2.52	92.35	1301
95.47	94.81	94.83	0.82	0.45	94.40	1822

Retail Building Dover Plains
CPT-2
Liquefaction Analysis

Vs No.	Depth (ft)	Vs (ft/sec)	Total Stress (psf)	Effective Stress (psf)	Vs1 (ft/sec)	Fines Content (%)	V*s1 (ft/sec)	N1,60	Ksigma	Alpha	Kalpha	Ka1	CSR	CRR	Safety Factor	Probability of Liquefaction (%)
1	3.8	493	418	336.88	690.2	60	656.16	33.9	1	0	1	1	.083	NL	---	---
2	7.1	603	781	493.96	844.2	60	656.16	81.1	1	0	1	1	.111	NL	---	---
3	10.3	651	1133	646.27	872.84	90	656.16	93.7	1	0	1	1	.122	NL	---	---
4	13.6	656	1496	803.36	832.98	90	656.16	76.5	1	0	1	1	.13	NL	---	---
5	16.9	698	1859	960.44	847.61	90	656.16	82.5	1	0	1	1	.135	NL	---	---
6	20.2	725	2222	1117.51	847.68	80	656.16	82.6	1	0	1	1	.137	NL	---	---
7	23.5	762	2585	1274.6	862.13	60	656.16	88.8	1	0	1	1	.139	NL	---	---
8	26.7	765	2937	1426.92	841.43	60	656.16	80	1	0	1	1	.139	NL	---	---
9	30	812	3300	1584	870.11	70	656.16	92.4	1	0	1	1	.139	NL	---	---
10	33.3	812	3663	1741.08	849.79	90	656.16	83.5	1	0	1	1	.137	NL	---	---
11	36.6	897	4025.99	1898.15	918.69	90	656.16	117	1	0	1	1	.134	NL	---	---
12	39.9	1418	4389	2055.23	1423.71	60	656.16	779.6	1	0	1	1	.131	NL	---	---
13	43.1	1583	4741	2207.56	1561.22	15	688.96	1162	1	0	1	1	.128	NL	---	---
14	46.4	1734	5111	2371.63	1679.76	15	688.96	1595.2	1	0	1	1	.123	NL	---	---
15	49.7	1510	5490.5	2545.22	1437.16	15	688.96	812	1	0	1	1	.118	NL	---	---
16	53	1497	5870	2718.79	1401.48	20	680.76	728.2	1	0	1	1	.113	NL	---	---
17	56.3	1311	6249.5	2892.38	1208.51	15	688.96	383.5	1	0	1	1	.107	NL	---	---
18	59.5	1150	6617.5	3060.7	1045.21	15	688.96	204.5	1	0	1	1	.103	NL	---	---
19	62.8	1223	6997	3234.28	1096.33	15	688.96	251.5	1	0	1	1	.099	NL	---	---
20	66.1	1205	7376.5	3407.86	1066.17	10	697.17	222.9	1	0	1	1	.096	NL	---	---
21	69.4	1233	7756	3581.44	1077.48	10	697.17	233.3	1	0	1	1	.093	NL	---	---
22	72.7	1179	8121.99	3741.51	1019.09	15	688.96	183.3	1	0	1	1	.09	NL	---	---
23	76	1219	8485	3898.6	1042.88	10	697.17	202.6	1	0	1	1	.088	NL	---	---
24	79.2	1256	8837	4050.92	1064.29	10	697.17	221.2	1	0	1	1	.086	NL	---	---
25	82.5	1214	9200	4208	1018.96	10	697.17	183.2	1	0	1	1	.085	NL	---	---
26	85.8	1355	9563	4365.07	1126.94	15	688.96	283.4	1	0	1	1	.083	NL	---	---
27	89.1	1228	9926	4522.16	1012.33	15	688.96	178.1	1	0	1	1	.082	NL	---	---
28	92.4	1301	10289	4679.23	1063.39	15	688.96	220.4	1	0	1	1	.082	NL	---	---
29	94.4	1822	10509	4774.44	1481.76	15	688.96	926.8	1	0	1	1	.081	NL	---	---

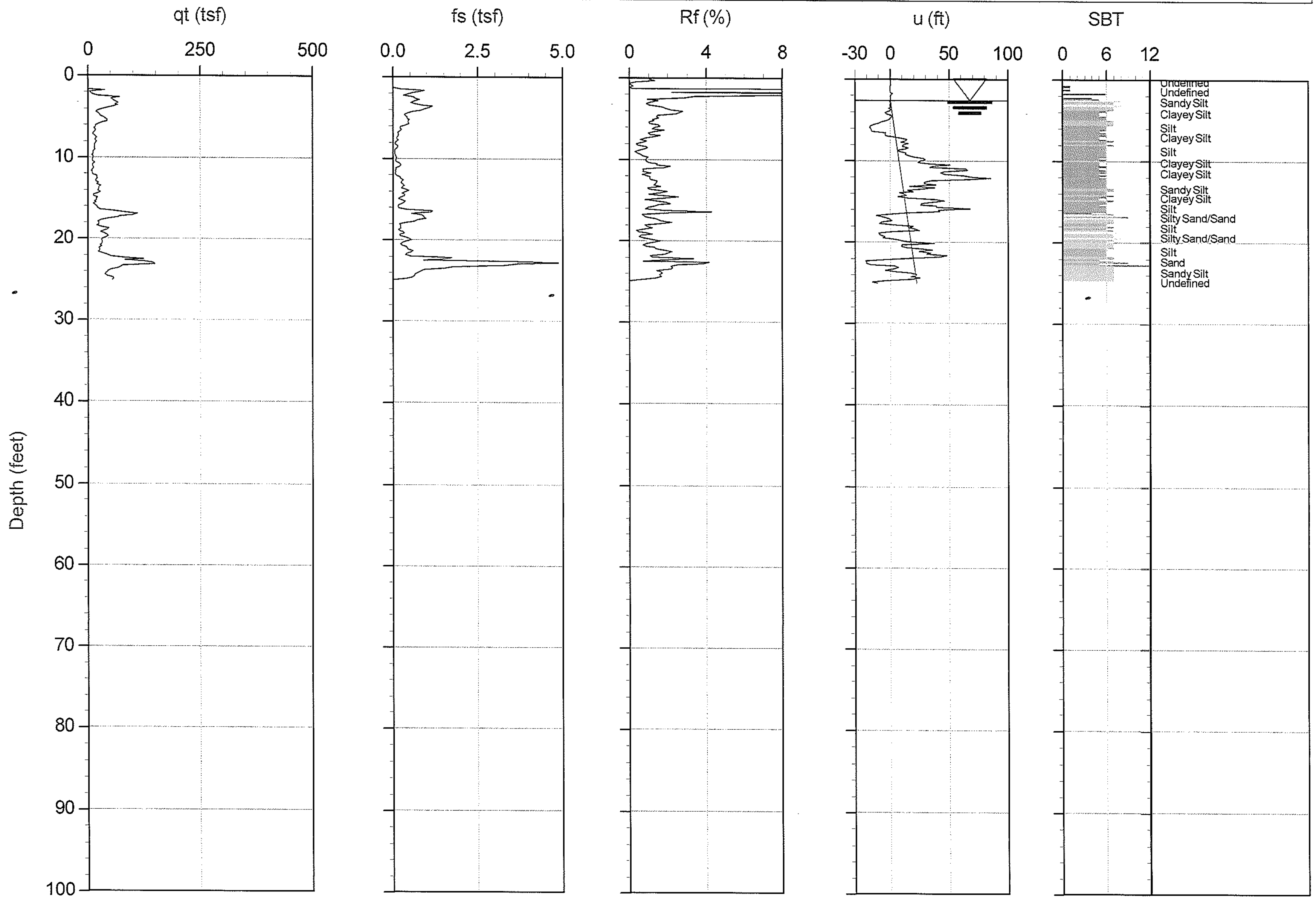
Notes:
CSR analysis using Seed & Idriss (1971)
CSR File: C:\SHAKE2000\2247CPT2.CSR
CRR using Vs Data and Andrus & Stokoe Approach.
CRR File:
Earthquake used in CSR Analysis:
Earthquake Magnitude for CRR Analysis: 6
Peak Ground Acceleration for CSR Analysis (g, from User): .112
Magnitude Scaling Factor (MSF): 1.77
Depth to Water Table for CRR Analysis (ft): 2.5
Depth to Water Table for Cvs Calculation (ft): 2.5
Depth to Base Layer for CSR Analysis (ft): 97.5
MSF Option: I.M. Idriss (1997)
Ksigma Option: Cetin et al. (2001) - Hynes & Olsen (1999)
CRR Equation from Andrus & Stokoe (2004)
Age of Soil Deposit: <10,000 years
*Value modified by user

CONETEC

Southern Realty Development

Job No: 11-701
Date: 01:10:11 12:38
Site: Dover Plains, NY

Sounding: CPT-01a
Cone: 301:T1500F15U500



Max Depth: 7.650 m / 25.10 ft
Depth Inc: 0.050 m / 0.164 ft

File: 701CP01A.COR

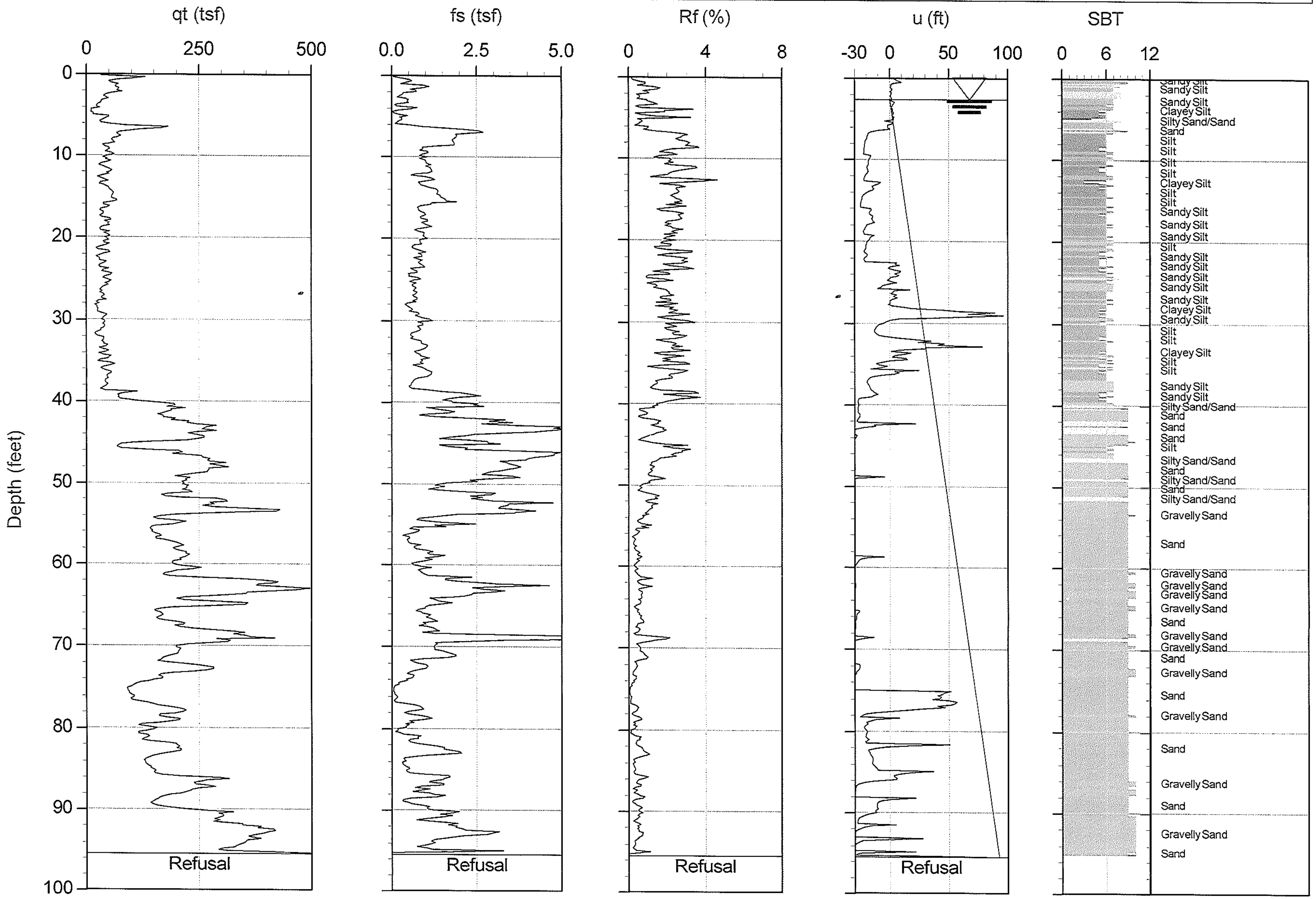
SBT: Lunne, Robertson and Powell, 1997
Coords: N: 4621078.471 E: 618050.741

CONETEC

Southern Realty Development

Job No: 11-701
Date: 01:10:11 12:59
Site: Dover Plains, NY

Sounding: CPT-02
Cone: 301:T1500F15U500



Max Depth: 29.100 m / 95.47 ft
Depth Inc: 0.050 m / 0.164 ft

File: 701CP02.COR

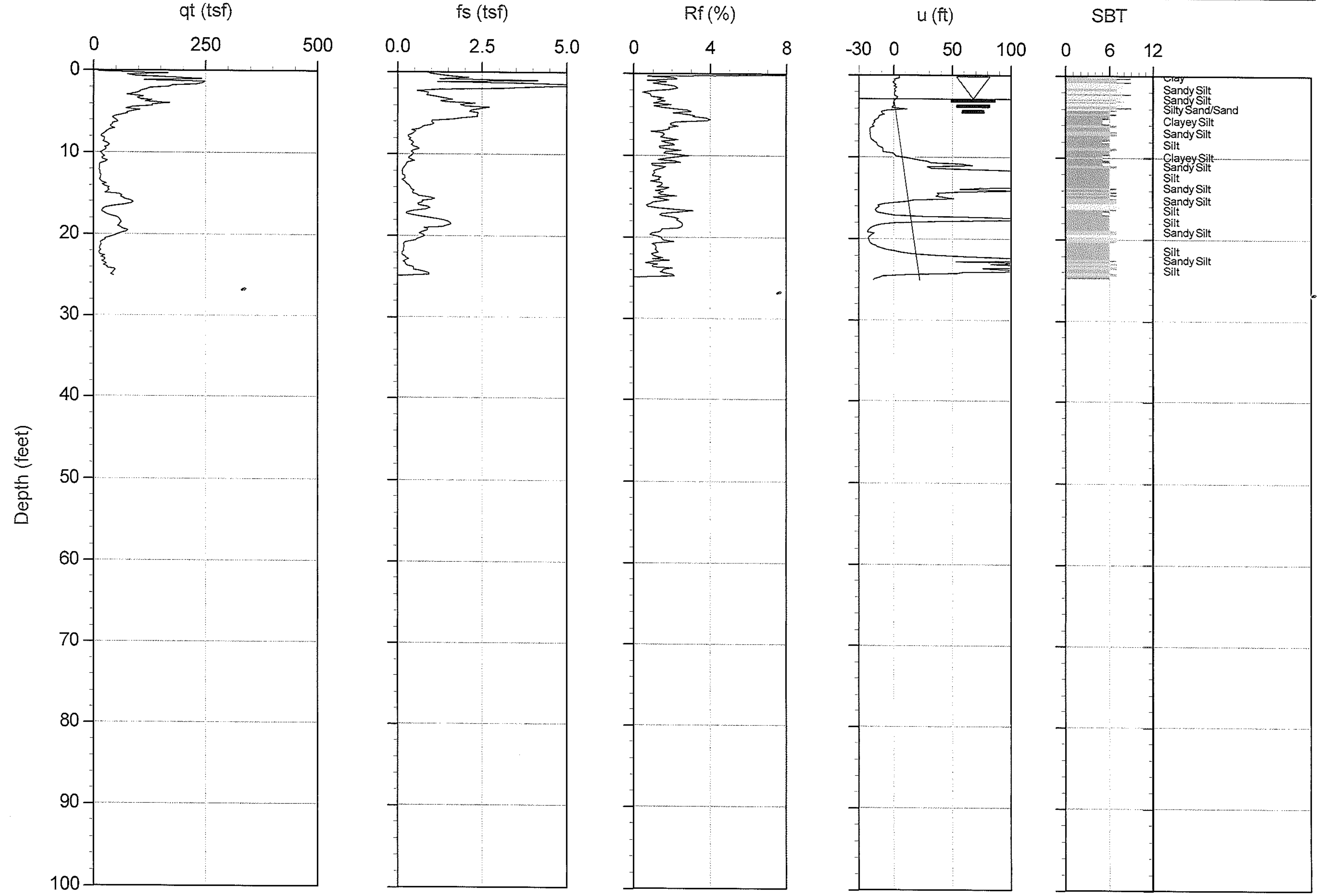
SBT: Lunne, Robertson and Powell, 1997
Coords: N: 4621034.769 E: 618094.443

CONETEC

Southern Realty Development

Job No: 11-701
Date: 01:10:11 14:21
Site: Dover Plains, NY

Sounding: CPT-03
Cone: 301:T1500F15U500



Max Depth: 7.650 m / 25.10 ft
Depth Inc: 0.050 m / 0.164 ft

File: 701CP03.COR

SBT: Lunne, Robertson and Powell, 1997
Coords: N: 4621027.916 E: 618127.831

CONSTRUCTION TECHNOLOGY

INSPECTION & TESTING DIVISION, P.D.& T.S., INC.
4 William Street, Ballston Lake, New York 12019
Phone: (518) 399-1848 Fax: (518) 399-1913

CLIENT:	**DANIEL LOUCKS, P.E.**	REPORT NUMBER:	1 : PAGE: 1
	POST OFFICE BOX 163	REPORT DATE:	12/14/10
	BALLSTON SPA, NEW YORK 12020	OUR FILE NUMBER:	750.001
		LAB CONTROL NUMBER:	12238

ATT'N: MR. DANIEL LOUCKS, P.E.

PROJECT: **DOVER VILLAGE**

A.S.T.M. D-2216: DETERMINATION OF WATER (MOISTURE) CONTENT IN SOILS

SAMPLE ID: **B-2, S-6, 10-12'**

WET WEIGHT (g)	DRY WEIGHT (g)	MOISTURE CONTENT
195.5	54.9	**256.1%**

SAMPLE ID: **B-2, S-6, 15-17'**

WET WEIGHT (g)	DRY WEIGHT (g)	MOISTURE CONTENT
171.6	119.9	**43.1%**

REPORT DISTRIBUTION
1: FILE
2:
3:
4:

RESPECTFULLY,
CONSTRUCTION TECHNOLOGY

TOM JOSLIN, S.E.T. (NICET)
MANAGER TECHNICAL SERVICES

Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Zip Code = 12522
Spectral Response Accelerations Ss and S1
Ss and S1 = Mapped Spectral Acceleration Values
Data are based on a 0.05000000074505806 deg grid spacing

Period (sec)	Centroid Sa (g)	
0.2	0.253	(Ss)
1.0	0.066	(S1)

Period (sec)	Maximum Sa (g)	
0.2	0.265	(Ss)
1.0	0.066	(S1)

Period (sec)	Minimum Sa (g)	
0.2	0.246	(Ss)
1.0	0.065	(S1)

Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Zip Code = 12522
Spectral Response Accelerations SMs and SM1
SMs = Fa x Ss and SM1 = Fv x S1
Site Class D

Period (sec)	Centroid Sa (g)	
0.2	0.405	(SMs, Fa = 1.597)
1.0	0.158	(SM1, Fv = 2.400)

Period (sec)	Maximum Sa (g)	
0.2	0.421	(SMs, Fa = 1.588)
1.0	0.159	(SM1, Fv = 2.400)

Period Minimum Sa
(sec) (g)

1.0	0.157	(SM1, Fv = 2.400)

Conterminous 48 States
2003 NEHRP Seismic Design Provisions
Zip Code = 12522
Spectral Response Accelerations SDs and SD1
SDs = 2/3 x SMs and SD1 = 2/3 x SM1
Site Class D

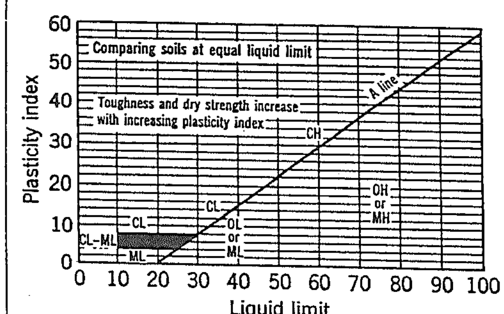
Period (sec)	Centroid Sa (g)	
0.2	0.270	(SDs)
1.0	0.105	(SD1)

Period (sec)	Maximum Sa (g)	
0.2	0.281	(SDs)
1.0	0.106	(SD1)

Period (sec)	Minimum Sa (g)	
0.2	0.263	(SDs)
1.0	0.105	(SD1)

Table 3.5 Unified Soil Classification

Major divisions			Field Identification Procedures (Excluding particles larger than 3 in. and basing fractions on estimated weights)			Group Symbols [a]	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria
Coarse-grained soils. More than half of material is *larger* than No. 200 sieve size[b] (The No. 200 sieve size is about the smallest particle visible to naked eye)	Gravels. More than half of coarse fraction is larger than No. 4 sieve size (For visual classification, the ¼ in. size may be used as equivalent to the No. 4 sieve size)	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes			*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name; indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics Example: *Silty sand*, gravelly; about 20% hard, angular gravel particles ½-in. maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)	Use grain size curve in identifying the fractions as given under field identification Determine percentages of gravel and sand from grain size curve Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse grained soils are classified as follows: Less than 5% *GW, GP, SW, SP* More than 12% *GM, GC, SM, SC* 5% to 12% *Borderline* cases requiring use of dual symbols $C_U = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing			*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for *GW*
		Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)			*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures		Atterberg limits below "A" line, or *PI* less than 4 — Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)			*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures		Atterberg limits above "A" line, with *PI* greater than 7
	Sands. More than half of coarse fraction is smaller than No. 4 sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes			*SW*	Well graded sands, gravelly sands, little or no fines		$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3
			Predominantly one size or a range of sizes with some intermediate sizes missing			*SP*	Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for *SW*
		Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)			*SM*	Silty sands, poorly graded sand-silt mixtures		Atterberg limits below "A" line or *PI* less than 5 — Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
			Plastic fines (for identification procedures, see *CL* below)			*SC*	Clayey sands, poorly graded sand-clay mixtures		Atterberg limits below "A" line with *PI* greater than 7
Fine-grained soils. More than half of material is *smaller* than No. 200 sieve size	Identification Procedures on Fraction Smaller than No. 40 Sieve Size		Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)				
	Silts and clays liquid limit less than 50		None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)	
			Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays		
			Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity		
	Silts and clays liquid limit greater than 50		Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts		
			High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays		
			Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity		
Highly Organic Soils			Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils		



Plasticity chart for laboratory classification of fine grained soils

From Wagner, 1957.

[a] *Boundary classifications*. Soils possessing characteristics of two groups are designated by combinations of group symbols. For example *GW–GC*, well graded gravel-sand mixture with clay binder.

[b] All sieve sizes on this chart are U.S. standard.

Field Identification Procedure for Fine Grained Soils or Fractions

These procedures are to be performed on the minus No. 40 sieve size particles, approximately 1/64 in. For field classification purposes, screening is not intended, simply remove by hand the coarse particles that interfere with the tests.

Dilatancy (Reaction to shaking):

After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.

Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil.

Very fine clean sands give the quickest and most distinct reaction whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

Dry Strength (Crushing characteristics):

After removing particles larger than No. 40 sieve size, mould a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun or air drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.

High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.

Toughness (Consistency near plastic limit):

After removing particles larger than the No. 40 sieve size, a specimen of soil about one-half inch cube in size, is moulded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about one-eight inch in diameter. The thread is then folded and re-rolled repeatedly. During this manipulation the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached.

After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles.

The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin-type clays and organic clays which occur below the A-line.

Highly organic clays have a very weak and spongy feel at the plastic limit.

Soil Characteristics Pertinent to Roads and Airfields

Major Divisions		Letter (1)		Name	Value as Subgrade When Not Subject to Frost Action	Value as Subbase When Not Subject to Frost Action	Value as Base When Not Subject to Frost Action	Potential Frost Action	Compressibility and Expansion	Drainage Characteristics	Compaction Equipment	Unit Dry Weight lb. per cu. ft.	Typical Design Values: CBR (2)	Typical Design Values: Subgrade Modulus k lb. per cu. in.
Coarse-Grained Soils	Gravel and Gravelly Soils	GW		Well-graded gravels or gravel-sand mixtures, little or no fines	Excellent	Excellent	Good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	125-140	40-80	300-500
		GP		Poorly graded gravels or gravel-sand mixtures, little or no fines	Good to excellent	Good	Fair to good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller, steel-wheeled roller	110-140	30-60	300-500
		GM	d	Silty gravels, gravel-sand-silt mixtures	Good to excellent	Good	Fair to good	Slight to medium	Very slight	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	125-145	40-60	300-500
			u		Good	Fair	Poor to not suitable	Slight to medium	Slight	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	115-135	20-30	200-500
		OC		Clayey gravels, gravel-sand-clay mixtures	Good	Fair	Poor to not suitable	Slight to medium	Slight	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	130-145	20-40	200-500
	Sand and Sandy Soils	SW		Well-graded sands or gravelly sands, little or no fines	Good	Fair to good	Poor	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller	110-130	20-40	200-400
		SP		Poorly graded sands or gravelly sands, little or no fines	Fair to good	Fair	Poor to not suitable	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired roller	105-135	10-40	150-400
		SM	d	Silty sands, sand-silt mixtures	Fair to good	Fair to good	Poor	Slight to high	Very slight	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	120-135	15-40	150-400
			u		Fair	Poor to fair	Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	100-130	10-20	100-300
		SC		Clayey sands, sand-clay mixtures	Poor to fair	Poor	Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired roller, sheepsfoot roller	100-135	5-20	100-300
Fine-Grained Soils	Silts and Clays LL is less than 50	ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity	Poor to fair	Not suitable	Not suitable	Medium to very high	Slight to medium	Fair to poor	Rubber-tired roller, sheepsfoot roller; close control of moisture	90-130	15 or less	100-200
		CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	Poor to fair	Not suitable	Not suitable	Medium to high	Medium	Practically impervious	Rubber-tired roller, sheepsfoot roller	90-130	15 or less	50-150
		OL		Organic silts and organic silt-clays of low plasticity	Poor	Not suitable	Not suitable	Medium to high	Medium to high	Poor	Rubber-tired roller, sheepsfoot roller	90-105	5 or less	50-100
	Silts and Clays LL is greater than 50	MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts	Poor	Not suitable	Not suitable	Medium to very high	High	Fair to poor	Sheepsfoot roller, rubber-tired roller	80-105	10 or less	50-100
		CH		Inorganic clays of medium to high plasticity, organic silts	Poor to fair	Not suitable	Not suitable	Medium	High	Practically impervious	Sheepsfoot roller, rubber-tired roller	90-115	15 or less	50-150
		OH		Organic clays of high plasticity, fat clays	Poor to very poor	Not suitable	Not suitable	Medium	High	Practically impervious	Sheepsfoot roller, rubber-tired roller	80-110	5 or less	25-100
Highly Organic Soils		Pt		Peat and other highly organic soils	Not suitable	Not suitable	Not suitable	Slight	Very high	Fair to poor	Compaction not practical	—	—	—

Note:

(1) Unit Dry Weights are for compacted soil at optimum moisture content for modified AASHO compaction effort. Division of GM and SM groups into subdivision of d and u are for roads and airfields only. Subdivision is basis of Atterberg limits; suffix d (e.g., GMd) will be used when the liquid limit (LL) is 25 or less and the plasticity index is 6 or less; the suffix u will be used otherwise.

(2) The maximum value that can be used in design of airfields is, in some cases, limited by gradation and plasticity requirements.

GENERAL QUALIFICATIONS

This report has been prepared in order to aid in the evaluation of this property and to assist the architect and/or engineer in the design of this project. The scope of the project and location described herein, and my description of the project represents my understanding of the significant aspects relevant to soil and foundation characteristics. In the event that any changes in the design or location of the proposed facilities, as outlined in this report, are planned, I should be informed so the changes can be reviewed and the conclusions of this report modified or approved in writing by myself.

It is recommended that all construction operations dealing with earthwork and foundations be inspected by an experienced soil engineer to assure that the design requirements are fulfilled in the actual construction. If you wish, I would welcome the opportunity to review the plans and specifications when they have been prepared so that I may have the opportunity of commenting on the effect of soil conditions on the design and specifications.

The analysis and recommendations submitted in this report are based upon the data obtained from the soil borings and/or test pits performed at the locations indicated on the location diagram and from any other information discussed in the report. This report does not reflect any variations which may occur between these boring and/or test pits. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is a well-known fact that variations in soil and rock conditions exist on most sites between boring locations and also such situations as groundwater conditions vary from time to time. The nature and extent of variations may may not become evident until the course of construction. If variations then appear evident, it will be necessary for a reevaluation of the recommendations of this report after performing on-site observations during the construction period and noting the characteristics of any variations.