

June 30, 1994



**LAW**

ENGINEERING AND ENVIRONMENTAL SERVICES

Mr. J. Michael Fry, P.E.  
Campco Engineering, Inc.  
Post Office Box 11326  
Rock Hill, South Carolina 29731-1326

**Subject:      Report of Geotechnical Exploration  
Replacement Bridge over Steele Creek  
Pleasant Road  
York County, North Carolina  
Law Engineering Job 222-07867-01**

Gentlemen:

As authorized by your acceptance of our Proposal No. 082G4 dated May 11, 1994, Law Engineering has completed a subsurface exploration for the subject project. The purpose of this exploration was to develop information about the site and subsurface conditions and to provide foundation recommendations for the proposed construction. This report describes the work performed and presents the results obtained, along with our geotechnical recommendations for foundation design and site preparation.

### PROJECT AND SITE INFORMATION

The existing Pleasant Road bridge over Steele Creek will be replaced with a new bridge. The new five-span bridge will be 240 feet long. The two end spans will be 30 feet long and the three interior spans will be 60 feet.


The above information was obtained from site meetings and telephone conversations with Mr. Mike Fry of Campco Engineering, a furnished drawing and site observations by our field personnel. South Carolina Department of Transportation guidelines were used in preparing this report.

### FIELD EXPLORATION

#### Soil Test Borings

Six soil test borings were made at the site at locations shown on the attached Boring Location Plan. The boring locations were selected by Law Engineering, and were located in the field by our drill crew from

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map-scaled distances, using a tape and estimated right angles. The elevations on the Test Boring Records were estimated using the ground surface profiles furnished to us by Campco Engineering. Some of the borings had to be moved from the originally proposed locations on bridge bents, due to soft, inaccessible flood plain conditions. Boring B-5 had to be drilled on the shoulder of the existing road, about 7 ft above the flood plain ground surface elevation.

The borings were made by mechanically twisting a continuous flight steel auger into the soil. Soil sampling and penetration testing were performed in general accordance with ASTM D 1586. At regular intervals, soil samples were obtained with a standard 1.4-inch I. D., 2-inch O. D., split-tube sampler. The sampler was first seated 6 inches to penetrate any loose cuttings, and then driven an additional 12 inches with blows of a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler the final 12 inches was recorded and is designated the "penetration resistance". The penetration resistance, when properly evaluated, is an index to the soil's strength and foundation supporting capability.

Representative portions of the soil samples, thus obtained, were placed in glass jars and transported to the laboratory. In the laboratory, the samples were examined by a geotechnical engineer to verify the driller's field classifications. Test Boring Records are attached, showing the soil descriptions and penetration resistances.

#### Undisturbed Sampling

Split-barrel samples are suitable for visual examination and classification tests but are not sufficiently intact for quantitative laboratory tests. Therefore, relatively undisturbed samples were obtained in selected borings by drilling to the desired depth and hydraulically forcing a section of 3-inch O.D., 16 gauge steel tubing into the soil. The sampling procedure is described by ASTM D 1587. Each tube, together with the encased soil, was carefully removed from the ground, made airtight and transported to the laboratory. The depths of undisturbed samples are shown on the appropriate Test Boring Records.

### Core Drilling

Refusal materials are materials that cannot be penetrated with the soil drilling methods employed. Refusal, thus indicated, may result from boulders, lenses, ledges or layers of relatively hard rock underlain by partially weathered rock or residual soil; refusal may also represent the surface of relatively continuous bedrock. Core drilling procedures are required to penetrate refusal materials and determine their character and continuity.

Prior to coring, casing was set in the drilled hole through the overburden soils, to keep the hole from caving. Refusal materials were then cored according to ASTM D 2113 using a diamond-studded bit fastened to the end of a hollow double-tube core barrel. This device was rotated at high speeds, and the cuttings were brought to the surface by circulating water. Core samples of the material penetrated were protected and retained in the swivel-mounted inner tube. Upon completion of each drill run, the core barrel was brought to the surface, the core recovered was measured, the samples were removed and the core placed in boxes for storage.

The core samples were returned to our laboratory where the refusal material was identified and the percent core recovery and rock quality designation (RQD) was determined by a geotechnical engineer or geologist. The percent core recovery is the ratio of the sample length obtained to the depth cored, expressed as a percent. The rock quality designation is obtained by summing only those pieces of recovered core which are 4 inches or longer and are at least moderately hard, and dividing by the total length cored. The percent core recovery and the RQD are related to soundness and continuity of the refusal material. Refusal material descriptions, recoveries and the bit size used are shown on the Test Boring Records.

#### HARDNESS:

Very Soft:	Pieces 1 inch or more in thickness can be broken by finger pressure; can be scratched readily by a fingernail.
Soft:	May be broken with fingers.

Medium:	May be cratched with a nail; corners and edges may be broken with fingers.
Moderately Hard:	Moderate blow of hammer required to break sample.
Hard:	Hard blow of hammer required to break sample.
Very Hard:	Several hard blows of hammer required to break sample.

## LABORATORY TESTING

### Natural Moisture Content

The natural moisture content of selected samples split-spoon was determined in accordance with ASTM D 2216. The moisture content of the soil is the ratio, expressed as a percentage, of the weight of water in a given mass of soil to the weight of the soil particles. The results are presented on the attached boring log B-6 and on the attached Grain Size Distribution Test Report sheets.

### Grain Size Distribution

Grain size tests were performed on representative soil samples to determine the particle size distribution of these materials. After initial drying, the samples were washed over a U. S. standard No. 200 sieve to remove the fines (particles finer than a No. 200 mesh sieve). The samples were then dried and sieved through a standard set of nested sieves. This test was performed in a manner similar to that described by ASTM D 422. The results are presented as percent finer by weight versus particle size curves on the attached Grain Size Distribution sheets.

### Soil Plasticity

Representative samples of the upper clayey soils were selected for Atterberg Limits testing to determine their soil plasticity characteristics. The soil's Plasticity Index (PI) is representative of this characteristic and is bracketed by the Liquid Limit (LL) and the Plastic Limit (PL). These characteristics are

determined in accordance with ASTM D 4318. The LL is the moisture content at which the soil will flow as a heavy viscous fluid. The PL is the moisture content at which the soil begins to lose its plasticity. The data obtained are presented on the attached boring log B-6 and on the attached Grain Size Distribution Test Report sheets.

## **AREA GEOLOGY**

The project site is located in the Piedmont Physiographic Province, an area underlain by ancient igneous and metamorphic rocks. The virgin soils encountered in this area are the residual product of in-place chemical weathering of rock which was similar to the rock presently underlying the site. In areas not altered by erosion or disturbed by the activities of man, the typical residual soil profile consists of clayey soils near the surface, where soil weathering is more advanced, underlain by sandy silts and silty sands. The boundary between soil and rock is not sharply defined. This transitional zone termed "partially weathered rock" is normally found overlying the parent bedrock. Partially weathered rock is defined, for engineering purposes, as residual material with standard penetration resistances in excess of 100 blows per foot. Weathering is facilitated by fractures, joints and by the presence of less resistant rock types. Consequently, the profile of the partially weathered rock and hard rock is quite irregular and erratic, even over short horizontal distances. Also, it is not unusual to find lenses and boulders of hard rock and zones of partially weathered rock within the soil mantle, well above the general bedrock level.

Often, the upper soils along drainage features and in flood plain areas are water-deposited (alluvial) materials that have been eroded and washed down from adjacent higher ground. These alluvial soils are usually soft and compressible, having never been consolidated by pressures in excess of their present overburden.

## **SUBSURFACE CONDITIONS**

Boring B-5, drilled in the shoulder of the existing road, encountered 7 ft of embankment fill. Beneath

the fill at B-5 and from the ground surface at borings B-4 and B-6, alluvial soils were encountered. The alluvium ranged from 8 to 13.5 ft thick and was composed of very soft to firm sandy silty clay (CH) and loose clayey silty sand. Alluvial soils were not present at borings B-1, B-2 and B-3.

Residual soils were encountered from the ground surface at borings B-1, B-2 and B-3 and beneath the alluvium elsewhere. The upper residual soils at borings B-1, B-2 and B-3 consisted of stiff to very stiff clayey silts or very firm clayey silty sands. The deeper residual soils at all the borings consisted of firm (some loose) to very dense silty sand. The residual soils are micaceous.

The borings encountered residual soil hard enough to be designated partially weathered rock at depths ranging from 7 to 17 ft below the flood plain surface. Partially weathered rock was not encountered in B-4. The partially weathered rock was sampled as silty sand.

Refusal to roller cone drill was encountered at 20 ft, 13.5, and 18.6 ft in borings B-2, B-3, B-6 and B-4, respectively. The refusal material was cored at borings B-3 and B-4 using rock coring techniques. The recovered rock consisted of hard to very gabbro. The recovery values were 78 and 98 percent and RQD values were 92 and 100 percent.

Groundwater was encountered within 1 to 3 feet of the ground surface at all the borings except B-5, which was drilled about 7 ft higher than all the other borings. Groundwater was at 5.5 in B-5, which is slightly above the adjacent flood plain elevation. Groundwater levels may fluctuate several feet with seasonal and rainfall variations and with changes in the water level in adjacent drainage features. Normally, the highest groundwater levels occur in late winter and spring and the lowest levels occur in late summer and fall.

The above descriptions provide a general summary of the subsurface conditions encountered. The attached Test Boring Records contain detailed information recorded at each boring location. These Test Boring Records represent our interpretation of the field logs based on engineering examination of the field samples. The lines designating the interfaces between various strata represent approximate boundaries and the transition between strata may be gradual.

## FOUNDATION EVALUATION AND RECOMMENDATIONS

### Foundations

Based on the boring data and our past experience with similar soils, the partially weathered rock and/or rock at the borings will provide adequate support for a system of deep foundations for the proposed bridge, subject to the criteria and site preparation recommendations that follow. Spread foundations will not be suitable due to the generally compressible low consistency upper soils and potential for scour.

Project designers are considering H-Piles and driller piers (caissons) to support the bridge. Both foundation systems are described in the following sections of this report.

### H-Piles

Steel H piles driven (HP sections) into partially weathered rock or refusing on rock will develop their working capacities primarily from end bearing with some contribution from skin friction in the deeper harder residual soils. Based on the completed borings, we recommend a maximum allowable working capacity of 60 tons for H-piles driven several feet into partially weathered rock or refusing on hard continuous rock. Pile lengths ranging from 15 to 20 ft below the existing flood plain elevations are expected based on the borings. The pile section selected by the designer should comply with the South Carolina Building Code regarding the pile cross-sectional area as required to keep the design stress within allowable limits.

The vertical support provide by the soil in contact with the pile cap should be omitted from the capacity calculations. We recommend a minimum center-to-center pile spacing of 3 pile diameters. This restriction is advisable to limit surface heave, to enhance the bearing efficiency of the individual piles, and to reduce the possibility of damaging previously installed piles. During installation of piles, it is recommended that the piling be driven from the center of each pile group to the perimeter to help minimize any heaving effects.

In our opinion, a load testing of the piles should not be required provided the pile design load is less than 60 tons. If the bridge designer desires to have a pile load test performed, the geotechnical engineer should be retained to consult on selection of the load test location, observe and document the foundation installation, analyze and report the load test results, and develop recommendations for production foundation depths and installation procedures. Any significant differences from accepted procedures or expected results should be brought to the attention of the foundation designers.

The pile driving hammer should be compatible with the pile type selected. The pile driving hammer utilized should be selected in relation to the weight of the piles. The ratio of the pile hammer ram weight to the weight of the piles are driven should not be less than 1/2 and preferably on the order of 3/4 to 1. A proper cushion system, located between the pile head and ram, should be selected in relation to the pile type and the hammer size. A pile driven with a minimum energy of 25,000 to 30,000 ft should be used to drive the piles.

Installation of the production piles should be governed by a suitable dynamic pile driving criteria. The use of either the Hiley Formula, the Janbu Formula, the Wave Equation, or any other formula that considers the weight of the pile is recommended. Piles should be installed in conformance with the applicable South Carolina State Building Code. Once the pile hammer and pile section are finalized, the wave equation analysis can be performed to evaluate driving stresses to be sure driving stresses are within tolerable limits to avoid overstressing the piles during driving.

An engineering technician working under the supervision of the geotechnical engineer should monitor the installation of all production piles. Pile driving records/documenting driving equipment, driven length, driving resistance at termination, and other pertinent information should be maintained by the technician and reviewed by the geotechnical engineer.

#### Caissons (Drilled Piles)

Caissons would be designed to penetrate through the compressible soils to bear in the relatively



*June 30, 1994*

incompressible underlying partially weathered rock and/or materials. The allowable end bearing pressure for the caissons would be dependent upon the quality of the bearing materials. Caissons bearing in continuous partially weathered rock (50=6") without soil seams below the bearing level may be designated utilizing an allowable total load end bearing pressure of 30 ksf. An end bearing pressure of 60 ksf could be utilized for caissons bearing in continuous partially weathered rock with penetration resistance of 50=2" or better. A design end bearing pressure of up to 100 ksf would be potentially available for caissons bearing on hard, continuous rock.

The borings indicate that the depth to the top of partially weathered rock ranges from 7 to 14 ft below existing site grade with stabilized ground water near the ground surface. Due to the shallow ground water, in our opinion, it would not be practical to install belled (under-reamed) caissons. In our opinion, the most practical caisson design would be to use straight shaft caissons designed for an end bearing pressure of 60 ksf. It will be important to properly seat the caisson liner into the bearing materials to "seal-off" ground water inflow.

For efficient drilling of caissons to the 60 ksf and bearing material, heavy, powerful caisson drilling machinery with at least 30,000 lbs of available total weight applied to the augers should be used. Even with this equipment, the use of rock augers will be required in some locations to drill through layers of partially weathered rock or rock interlayered with soil that must be penetrated to reach continuous hard partially weathered rock.

The borings indicate that the depth of caissons required to obtain a design end bearing pressure of 60 ksf would range from about 13.5 ft on the hard rock of B-3 and B-4 to as much as 22 to 25 ft at B-5. The caisson bearing depths must be below the scour depth determined by the bridge designer.

A temporary protective steel casing should be installed in the drilled hole. This will prevent side wall collapse and, if sealed on or into the refusal material, prevent excessive mud intrusion. Water inflow through the seamy, fractured material may require extending the casing deeper into the material. The steel casing should be installed deep enough to allow workers to safely excavate, clean, and inspect the drilled

pier. A minimum drilled pier diameter of 30 inches is recommended to provide reasonable entry space for cleaning, bottom penetration, and inspection.

The protective steel casing may be extracted as the concrete is placed. A sufficient head of concrete should be maintained above the bottom of the casing during withdrawal and the contractor should prevent concrete from "hanging-up" inside the shell which can cause soil and water intrusion below the casing.

Concrete slumps ranging from 4 to 7 inches are recommended for the drilled pier construction. Concrete with slumps in this range will usually fill irregularities along the sides and bottom of the hole, displace water as it is placed, and permit placement of any reinforcing cages into the fluid concrete.

The geotechnical engineer should be retained to observe the drilled pier construction. He should document the shaft diameter, depth, cleanliness, plumbness, and type of bearing material. Significant deviations from the specified or anticipated conditions should be reported to the owner's representative and to the foundation designer. The drilled pier excavation should be observed after the bottom of the hole is leveled, cleared of any mud or extraneous material, and dewatered.

We recommend that the drilled pier construction include at least one probe hole in the bottom of each drilled pier excavation. The probe holes should be about 1.5 inches in diameter and are usually drilled with a pneumatic percussion drill. These probe holes should be drilled to a depth of at least 1.5 times the pier bottom diameter or to 10 ft, whichever is less. Each hole should be checked by the geotechnical engineer with a steel feeler rod to assess the rock continuity. If this check indicates discontinuous rock or compressible seams, the drilled pier should be excavated deeper to competent material and verified by probe hole inspection.

#### Embankment Fills

Due to the shallow groundwater and soft surface soils the use of geogrids or geotextiles would be useful in constructing the embankment fills. We performed a slope stability analysis for a 2:1 (H:V) end slope

at the north abutment of the bridges using assumed soil strength parameters based on our previous experience with laboratory testing of similar soils on other sites. The compute program PC STABL, Version 6.0 was used for this analysis. The analyzed cross section is shown on Figure 2. A factor of safety of 1.53 was computed for this geometry. Based on the completed slope stability analyses end slopes of 2:1 (H:V) may be used. Roadway embankment approaches should be constructed prior to pile driving so the embankment can settle prior to the piles being driven.

### Engineered Fill

All fill used for raising site grade or for replacement of material that is undercut should be uniformly compacted in thin lifts to at least 95 percent of the standard Proctor maximum dry density (ASTM D 698). In addition, at least the upper 24 inches of subgrade fill beneath pavements should be compacted to 100 percent of the same specification.

Before filling operations begin, representative samples of each proposed fill material should be collected and tested to determine the compaction and classification characteristics. The maximum dry density and optimum moisture content should be determined. Once compaction begins, a sufficient number of density tests should be performed by an experienced engineering technician working under the direct supervision of the geotechnical engineer to measure the degree of compaction being obtained.

In site areas where several feet of structural fill will be placed to achieve proposed grades, we recommend that construction be delayed to allow time for the underlying soils and fill to "settle out" as they adjust to the overlying weight of materials. A period of several weeks may be required for this adjustment. Settlement plates installed at the base of the fill and monitored with a precision level would aid in determining when settlements are negligible and construction could begin.

The surface of compacted subgrade soils can deteriorate and lose its support capabilities when exposed to environmental changes and construction activity. Deterioration can occur in the form of freezing, formation of erosion gullies, extreme drying, exposure for a long period of time or rutting by construction

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traffic. We recommend that the surfaces of floor slab and pavement subgrades that have deteriorated or softened be proofrolled, scarified and recompacted (and additional fill placed, if necessary) immediately prior to construction of the floor slab or pavement. Additionally, any excavations through the subgrade soils (such as utility trenches) should be properly backfilled in compacted lifts. Recomposition of subgrade surfaces and compaction of backfill should be checked with a sufficient number of density tests to determine if adequate compaction is being achieved.


### QUALIFICATION OF REPORT


Our evaluation of foundation support conditions has been based on our understanding of the site and project information and the data obtained in our exploration. The general subsurface conditions utilized in our foundation evaluation have been based on interpolation of subsurface data between the borings. In evaluating the boring data, we have examined previous correlations between penetration resistances and foundation bearing pressures observed in soil conditions similar to those at your site. If the project information is incorrect or if the structure location (horizontal or vertical) and/or dimensions are changed, please contact us so that our recommendations can be reviewed. The discovery of any site or subsurface conditions during construction which deviate from the data outlined in this exploration should be reported to us for our evaluation. The assessment of site environmental conditions or the presence of pollutants in the soil, rock and ground water of the site was beyond the scope of this exploration.

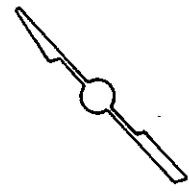
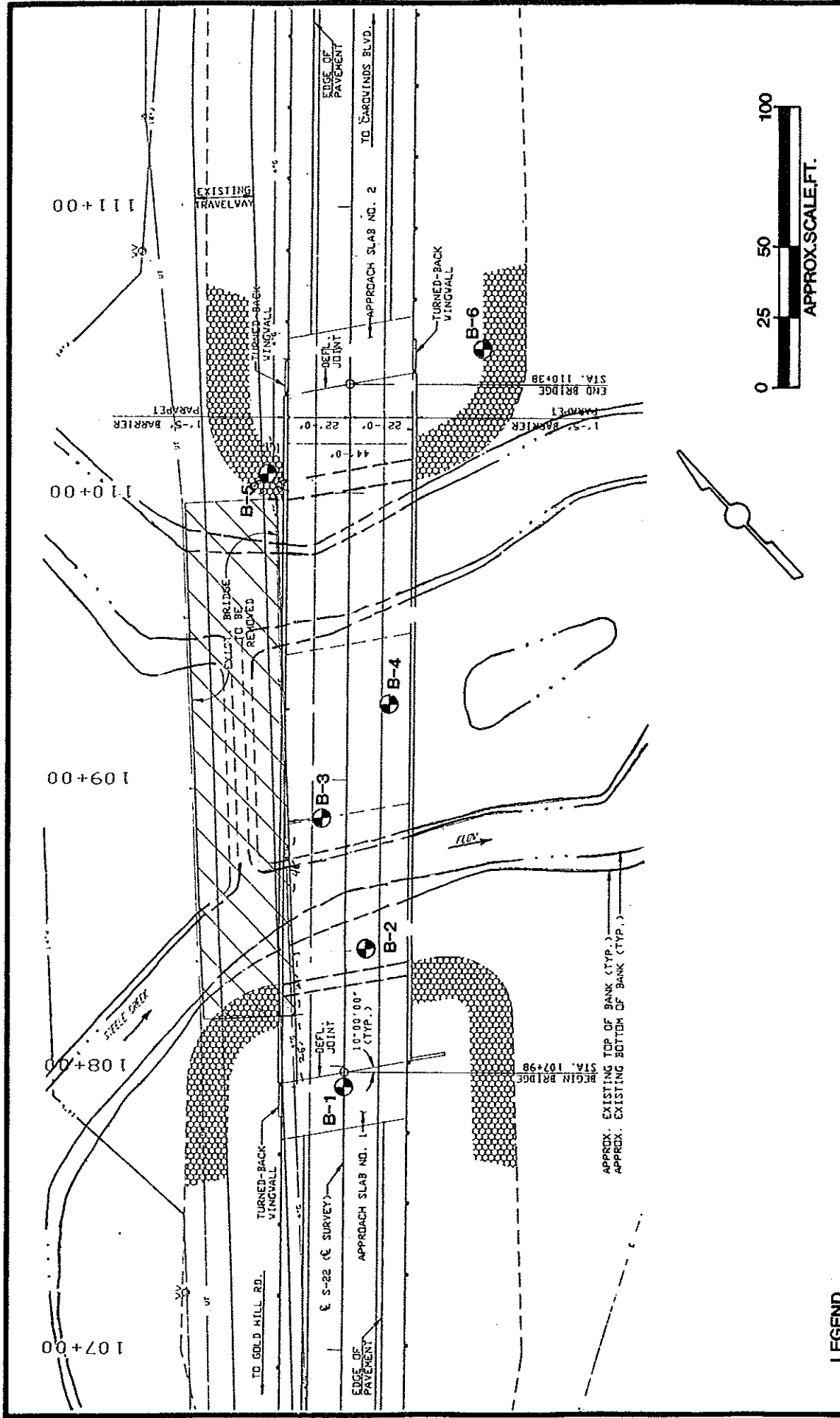
Thank you for the opportunity to provide our professional geotechnical services during this phase of your project. Please contact us when we can be of further service or if you have any questions concerning this report.

Very truly yours,

LAW ENGINEERING, INC.

  
Randall D. Lipshay, P.E.  
Senior Geotechnical Engineer  
Registered, S.C. 13412

  
Mel Y. Browning, P.E.  
Principal Geotechnical Engineer  
Registered, S.C. 8807



**LEGEND**

⊙ APPROX. LOCATION OF SOIL TEST BORING



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**CHARLOTTE, NORTH CAROLINA**

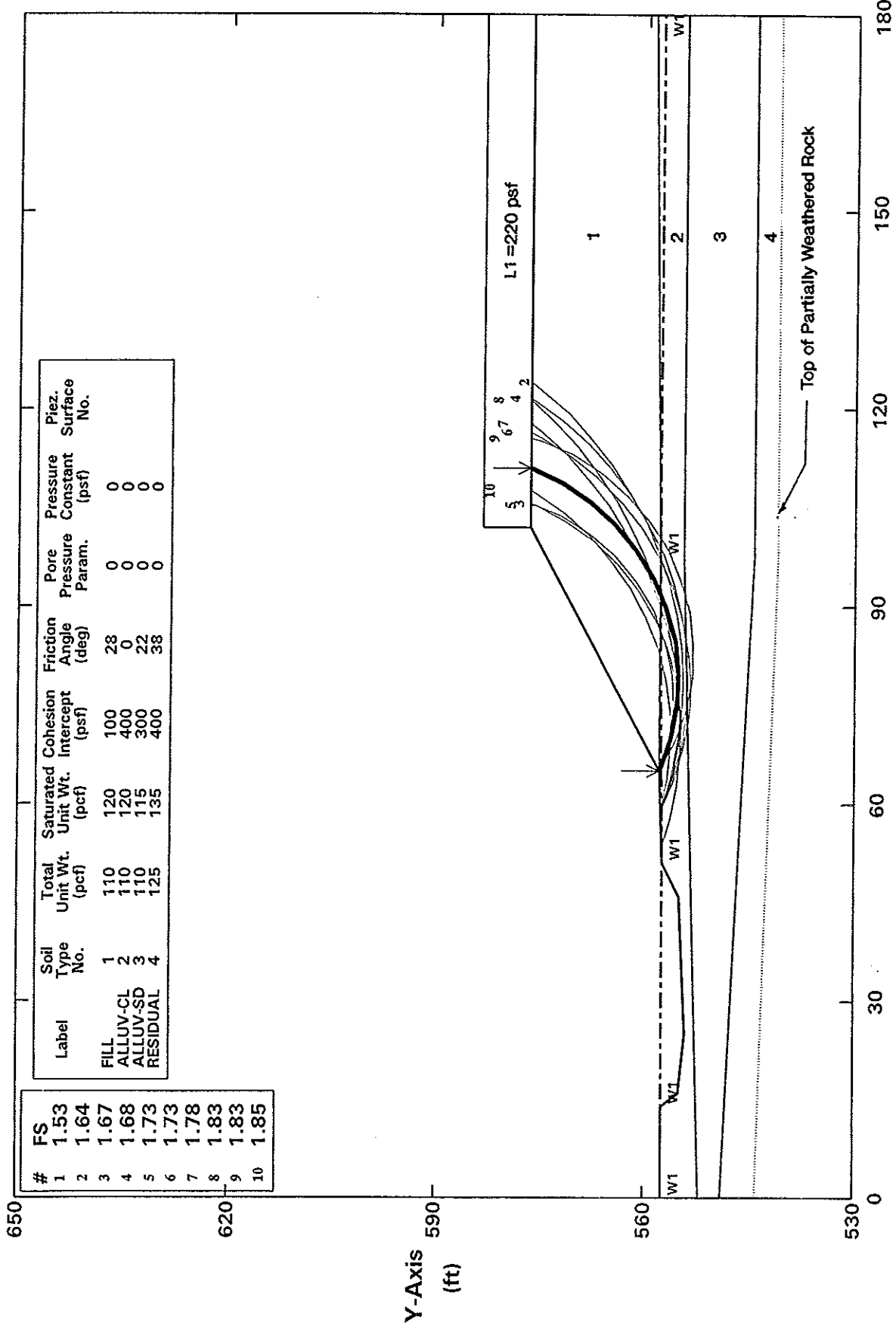
**BORING LOCATION PLAN**  
**REPLACE BRIDGE OVER STEELE CREEK**  
**PLEASANT ROAD IMPROVEMENTS**  
**YORK COUNTY, SOUTH CAROLINA**

REF: "PRELIMINARY PLAN & PROFILE", PREPARED BY CAMPCO ENGINEERING, INC. SHEET

NO: BR3, DATED MAY 18, 1994

JOB NO. 222-07867-01

FIGURE 1







PCSTABL FSmin = 1.53 X-Axis (ft)

## KEY TO CLASSIFICATIONS AND SYMBOLS

### CORRELATION OF PENETRATION RESISTANCE WITH RELATIVE DENSITY AND CONSISTENCY

	<u>No. of Blows, N</u>	<u>Relative Density*</u>
Sands	0 - 4	Very Loose
	5 - 10	Loose
	11 - 20	Firm
	21 - 30	Very Firm
	31 - 50	Dense
	51 +	Very Dense
		<u>Consistency*</u>
Silts and Clays	0 - 1	Very Soft
	2 - 4	Soft
	5 - 8	Firm
	9 - 15	Stiff
	16 - 30	Very Stiff
	31 +	Hard

## SYMBOLS

	- Undisturbed Sample (UD) Recovered
50=2"	- Number of Blows (50) to Drive the Spoon a Number of Inches (2)
BQ,NX,NQ,NW	- Core Barrel Sizes Which Obtain Cores 1-7/16, 2-1/8 Inches, 1-7/8 Inches, 2-1/6 Inches in Diameter, Respectively
65%	- Percentage (65) of Rock Core Recovered (Compared to Cored Length)
RQD	- Rock Quality Designation - Percentage of Recovered Cored Length Consisting of Moderately Hard or Better core Segments 4 or More Inches Long
	- Water Table Approximately 24 Hours or More After Drilling
	- Water Table Approximately at Time of Drilling (Within 1 Hour)
	- Loss of Drilling Fluid
C-	- Borehole Caved at Depth Indicated

\* Terminology may be altered if presence of gravel, cobbles or boulders interferes with accurate measurement of standard penetration resistances.

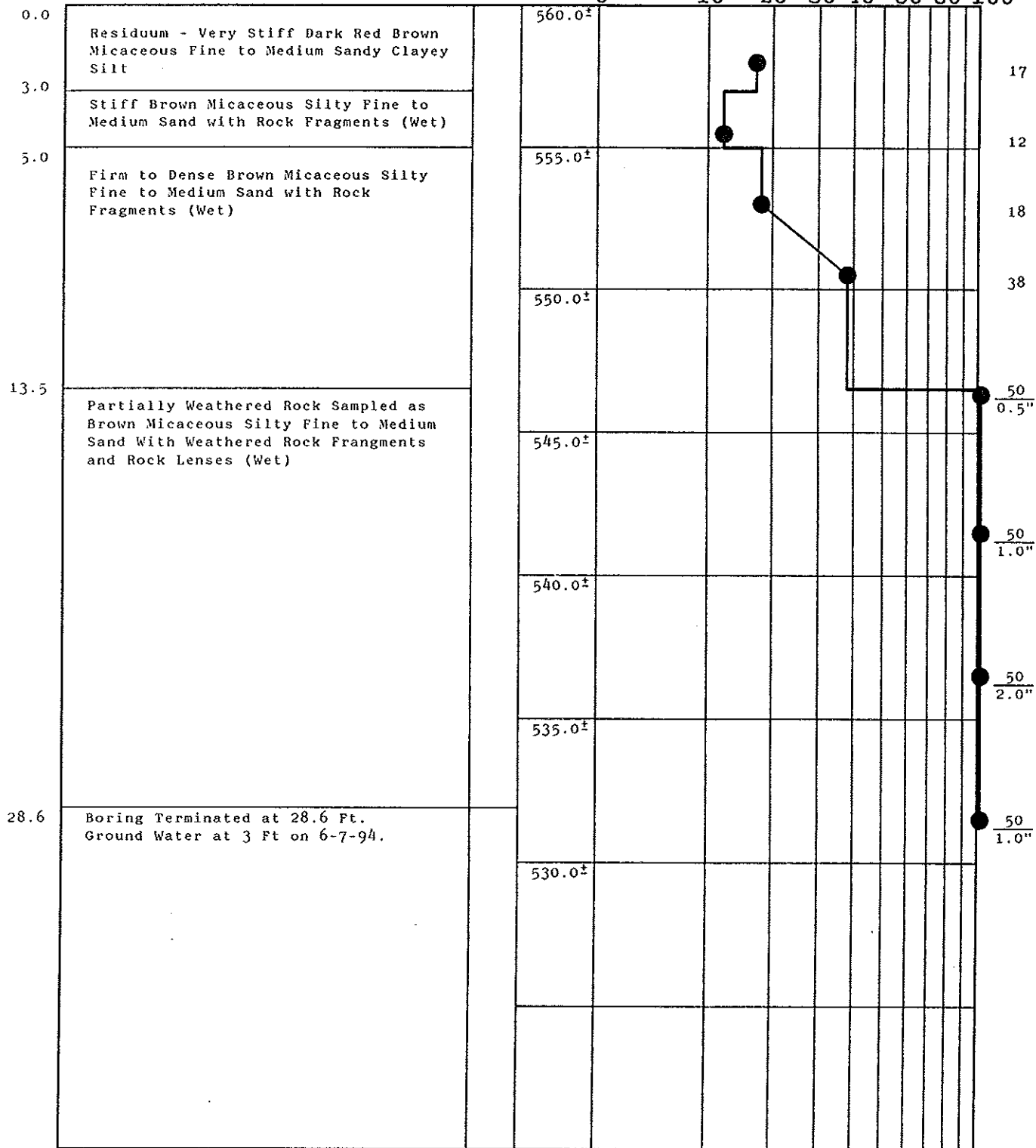
DEPTH  
(FT.)

DESCRIPTION

ELEVATION  
(FT.)

● PENETRATION - BLOWS/FOOT

0 10 20 30 40 60 80 100



TEST BORING RECORD

BORING NUMBER B-1  
 DATE DRILLED 5-31-94  
 PROJECT NUMBER 222-07867-01  
 PROJECT PLEASANT RD IMPROV.  
 PAGE 1 OF 1

SEE KEY SHEET FOR EXPLANATION OF  
SYMBOLS AND ABBREVIATIONS USED ABOVE

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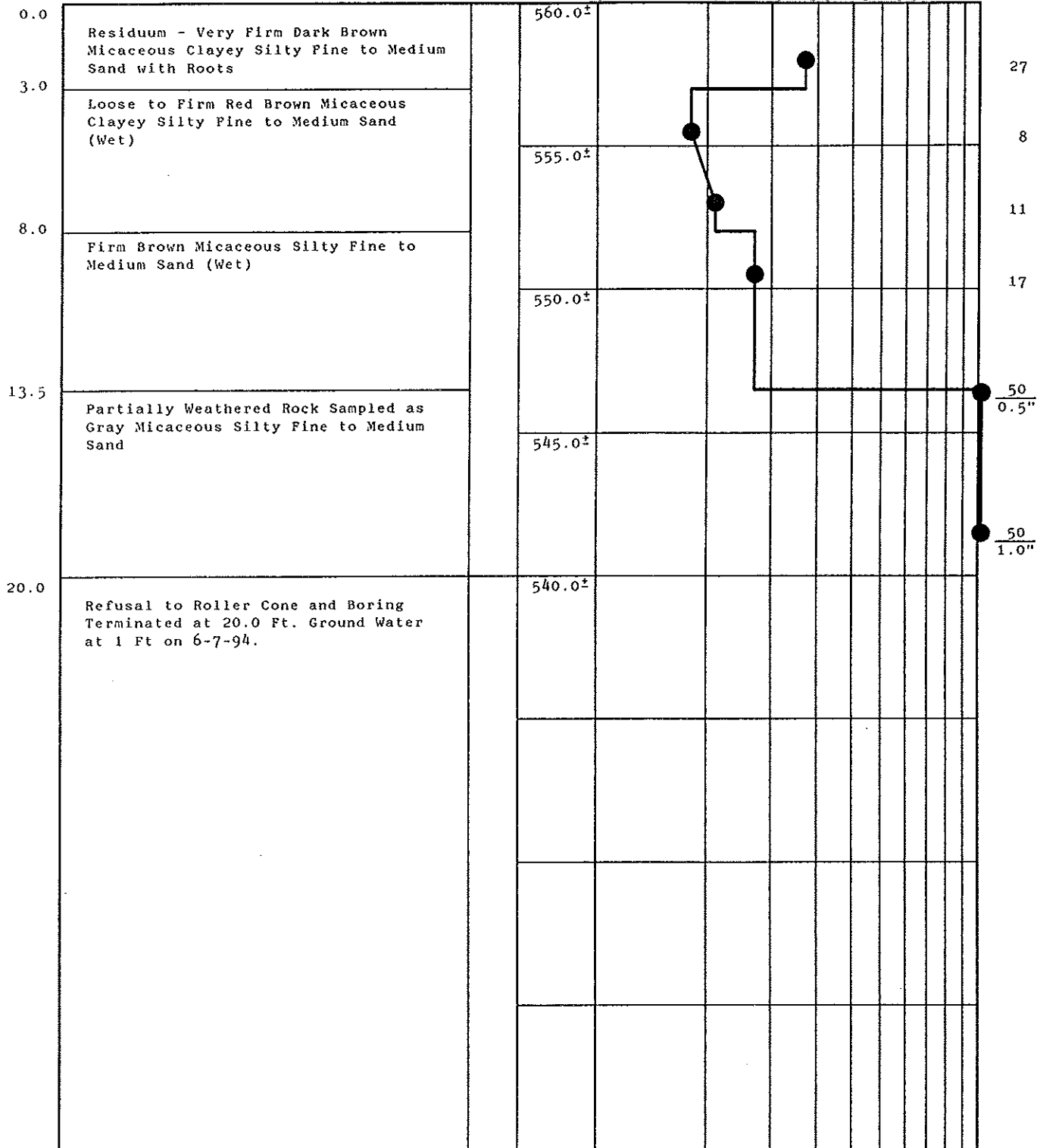
DEPTH  
(FT.)

DESCRIPTION

ELEVATION  
(FT.)

● PENETRATION - BLOWS/FOOT

0 10 20 30 40 60 80 100



TEST BORING RECORD

BORING NUMBER B-2  
 DATE DRILLED 5-31-94  
 PROJECT NUMBER 222-07867-01  
 PROJECT PLEASAND RD IMPROV.  
 PAGE 1 OF 1

SEE KEY SHEET FOR EXPLANATION OF  
 SYMBOLS AND ABBREVIATIONS USED ABOVE

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DEPTH  
(FT.)

DESCRIPTION

ELEVATION  
(FT.)

● PENETRATION - BLOWS/FOOT

0 10 20 30 40 60 80 100

0.0

Alluvium - Firm to Very Soft Gray  
Brown Micaceous Fine Sandy Silty Clay  
(Wet)

558.0±

5

8.0

Residuum - Dense Brown Micaceous  
Silty Fine to Medium Sand with Rock  
Fragments

553.0±

1

5

548.0±

39

Refusal to Roller Cone at 13.5 Ft

%RQD

%REC

13.5

Hard Gabbro

NQ

92

543.0±

78

18.5


Boring and Coring Terminated at 18.5  
Ft. Ground Water at 1 Ft on 6-7-94.

538.0±

### TEST BORING RECORD

BORING NUMBER B-4  
DATE DRILLED 6-2-94  
PROJECT NUMBER 222-07867-01  
PROJECT PLEASANT RD IMPROV.  
PAGE 1 OF 1

SEE KEY SHEET FOR EXPLANATION OF  
SYMBOLS AND ABBREVIATIONS USED ABOVE

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DEPTH (FT.)	DESCRIPTION	ELEVATION (FT.)	● PENETRATION - BLOWS/FOOT									
			0	10	20	30	40	60	80	100		
0.0	Roadway Fill	565.0±										
		560.0±										
7.0	Alluvium - Soft Brown Micaceous Fine Sandy Silty Clay (Wet)		●									4
10.0	Undisturbed Soil Sample	555.0±										
12.0	Soil											
13.0	Undisturbed Soil Sample											
15.0	Soil	550.0±										
20.5	Partially Weathered Rock Sampled as Brown Micaceous Silty Fine to Medium Sand	545.0±									● 50 1.0"	
											● 50 1.0"	
25.6	Refusal to Roller cone and Boring Terminated at 25.6 Ft. Ground Water at 5.5 Ft on 6-20-94.	540.0±										
	NOTE: Boring Drilled on Shoulder of Road, Due to Inaccessibility at Boring Location in Flood Plain.	535.0±										

### TEST BORING RECORD

BORING NUMBER B-5  
 DATE DRILLED 6-7-94  
 PROJECT NUMBER 222-07867-01  
 PROJECT PLEASANT RD IMPROV.  
 PAGE 1 OF 1

SEE KEY SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS USED ABOVE

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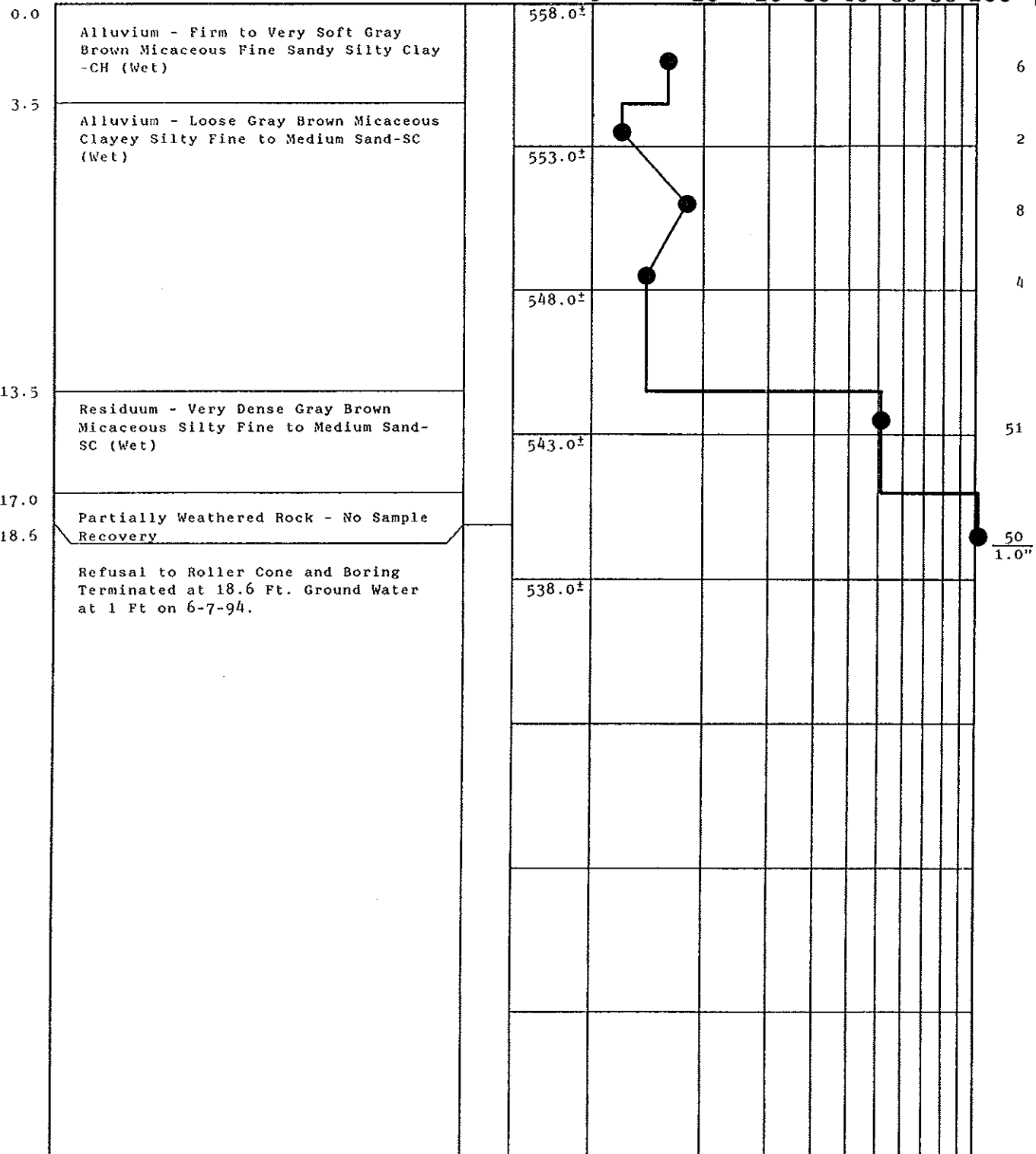
DEPTH  
(FT.)

DESCRIPTION

ELEVATION  
(FT.)

● PENETRATION - BLOWS/FOOT

0 10 20 30 40 60 80 100



### TEST BORING RECORD

BORING NUMBER B-6  
 DATE DRILLED 6-6-94  
 PROJECT NUMBER 222-07867-01  
 PROJECT PLEASANT RD IMPROV.  
 PAGE 1 OF 1

SEE KEY SHEET FOR EXPLANATION OF  
SYMBOLS AND ABBREVIATIONS USED ABOVE

 LAW ENGINEERING

# GRAIN SIZE DISTRIBUTION TEST REPORT



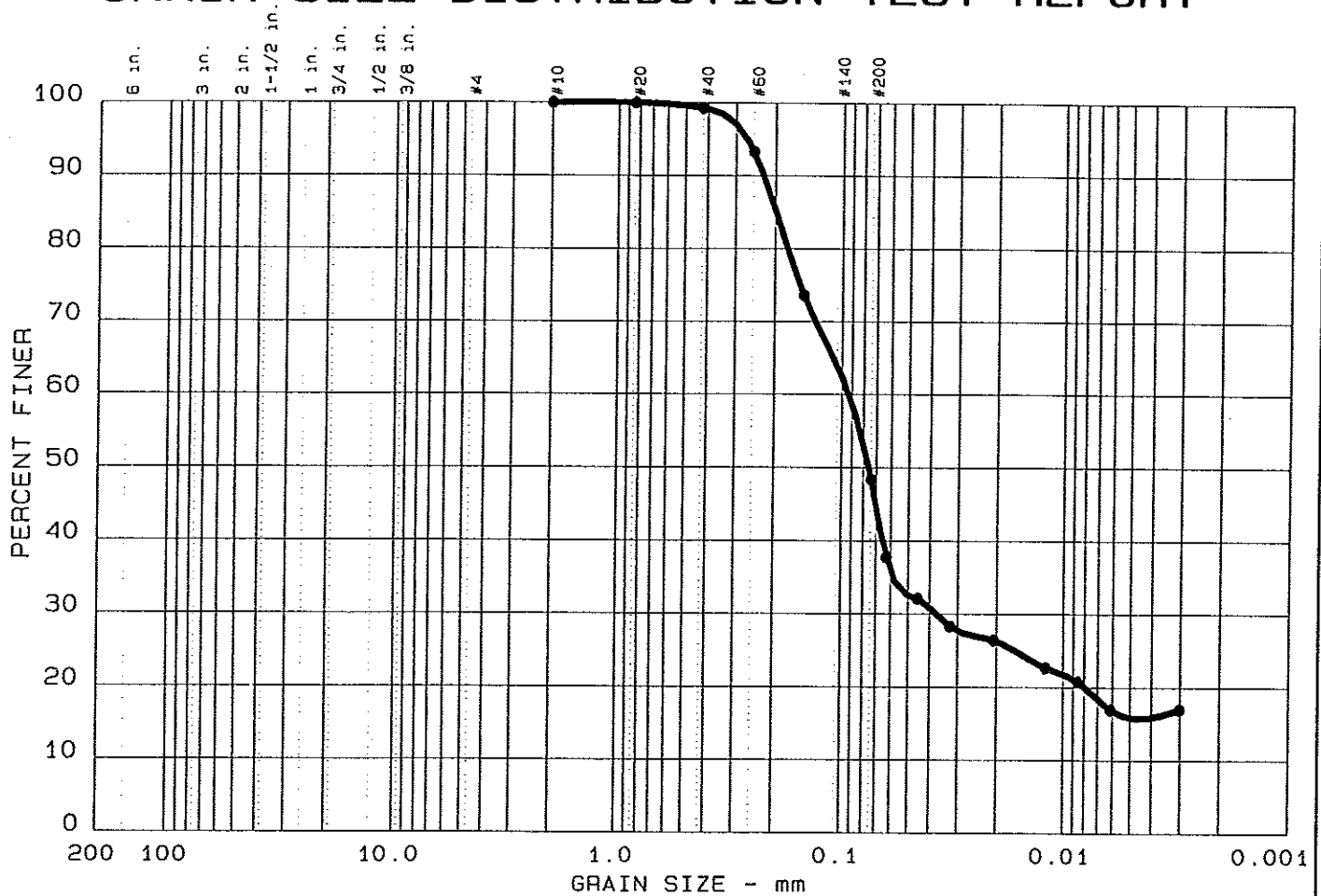
% +3"	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	7.4	46.0	46.6

LL	PI	D <sub>85</sub>	D <sub>60</sub>	D <sub>50</sub>	D <sub>30</sub>	D <sub>15</sub>	D <sub>10</sub>	C <sub>c</sub>	C <sub>u</sub>
54	27	0.05	0.01	0.01	0.003				

MATERIAL DESCRIPTION	USCS	AASHTO

Project No.: 222-7867-01 Project: PLEASANT ROAD BRIDGE • Location: B-6 @ 1'-2.5'  Date: JUNE 22, 1994	Remarks: NATURAL MOISTURE: 31.1%   Figure No. _____
GRAIN SIZE DISTRIBUTION TEST REPORT <b>LAW ENGINEERING</b>	

# GRAIN SIZE DISTRIBUTION TEST REPORT



% +3"	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	51.6	32.6	15.8

LL	PI	D <sub>85</sub>	D <sub>60</sub>	D <sub>50</sub>	D <sub>30</sub>	D <sub>15</sub>	D <sub>10</sub>	C <sub>c</sub>	C <sub>u</sub>
24	8	0.20	0.09	0.08	0.037				

MATERIAL DESCRIPTION	USCS	AASHTO

Project No.: 222-7867-01  
 Project: PLEASANT ROAD BRIDGE  
 Location: B-6 @ 3.5'-5.0'

Date: JUNE 22, 1994

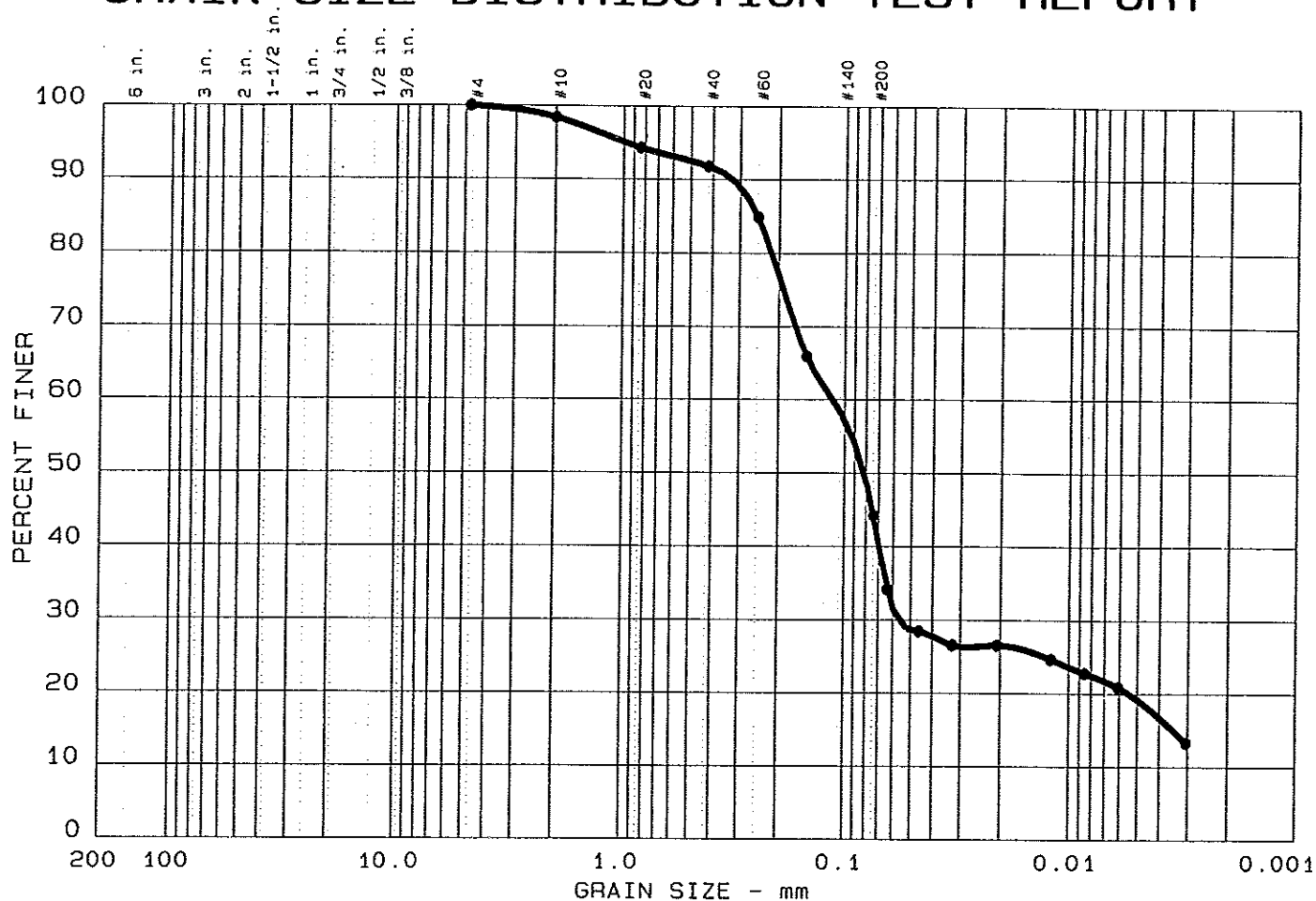
Remarks:  
 NATURAL MOISTURE: 21.2%

Figure No. \_\_\_\_\_

GRAIN SIZE DISTRIBUTION TEST REPORT

**LAW ENGINEERING**

# GRAIN SIZE DISTRIBUTION TEST REPORT



% +3"	% GRAVEL	% SAND	% SILT	% CLAY
0.0	0.0	55.8	25.0	19.2

LL	PI	D <sub>85</sub>	D <sub>60</sub>	D <sub>50</sub>	D <sub>30</sub>	D <sub>15</sub>	D <sub>10</sub>	C <sub>c</sub>	C <sub>u</sub>
30	13	0.25	0.11	0.08	0.057	0.0035			

MATERIAL DESCRIPTION	USCS	AASHTO

Project No.: 222-7867-01  
 Project: PLEASANT ROAD BRIDGE  
 • Location: B-6 @ 6.0'-7.5'

Date: JUNE 22, 1994

GRAIN SIZE DISTRIBUTION TEST REPORT  
**LAW ENGINEERING**

Remarks:  
 NATURAL MOISTURE: 18.2%

Figure No. \_\_\_\_\_



Grain size distribution curve showing Percent Finer versus Grain Size (mm). The curve is plotted on a semi-logarithmic scale. The Y-axis represents Percent Finer (0 to 100). The X-axis represents Grain Size in mm (200 to 0.001). The curve shows a well-graded soil with a peak at 100% finer for sizes down to 0.425 mm, followed by a steep drop between 0.425 mm and 0.075 mm, and then a gradual decrease to approximately 12% finer at 0.006 mm.

Grain Size (mm)	Percent Finer (%)
200	100
100	100
60	100
40	100
20	100
10	100
4.75	100
2.5	100
1.18	100
0.85	100
0.6	96
0.425	85
0.3	63
0.25	55
0.2	40
0.15	28
0.125	25
0.106	25
0.085	23
0.075	21
0.063	20
0.053	19
0.045	18
0.0375	17
0.03	17
0.025	16
0.02	15
0.016	14
0.0125	13
0.0106	12

[illegible]

MATERIAL DESCRIPTION	USCS	AASHTO
Project No.: 222-7867-01 Project: PLEASANT ROAD BRIDGE Location: B-6 @ 8.5'-10.0'  Date: JUNE 22, 1994	Remarks:  NATURAL MOISTURE: 26.8%	
GRAIN SIZE DISTRIBUTION TEST REPORT <b>LAW ENGINEERING</b>	Figure No. _____	



**LAW**

ENGINEERING AND ENVIRONMENTAL SERVICES

August 5, 1994

Mr. J. Michael Fry, P.E.  
Campco Engineering, Inc.  
Post Office Box 11326  
Rock Hill, South Carolina 29731-1326

**Subject: Addendum to Report of Geotechnical Exploration  
Replacement Bridge over Steele Creek  
Pleasant Road  
York County, North Carolina  
Law Engineering Job 222-07867-01**

Gentlemen:

The following additions and clarifications should be made to our original report dated June 30, 1994. This was discussed in a meeting at Campco on July 28, 1994.

Scour

A specific gravity of 2.7 should be used for scour calculations. We understand that Campco has calculated a scour depth of 13 ft below existing grade using this specific gravity and the grain size analyses included in our previous report.

As stated in our report, from a geotechnical load-capacity standpoint, either steel H-piles or drilled piers (caissons) could be used to support the bridge. However, we understand that footings for interior H-piles would have to be constructed such that the bottom of the footings will be below the scour depths. This would not be practical, considering the difficulties associated with shallow groundwater and also the fact that hard soil/partially weathered rock and rock was encountered at depths not much deeper than this scour depth in some of the borings.

The piles would "take-up" near these depths. Thus, with scour considerations, in our opinion, drilled piers would be the most practical foundations system to support the interior bents. Caissons could be drilled

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August 5, 1994

into the partially weathered rock to satisfy scour requirements, and if required, could be socketed into hard rock, using appropriate drilling tools such as rock augers and carbide steel drilling equipment.

#### Embankment Settlement

As recommended in our previous report, settlement plates should be installed on the existing grade prior to embankment fill placement. The settlement plates will allow survey measurement of the settlement vs. time of the alluvial soils under the fill loading. Several inches of settlement is expected in the loose alluvium. We anticipate that settlement due to the fill loadings should be completed within 3 to 4 weeks. Construction should be delayed until the settlement has been substantially completed.


#### Modification to Boring B-5

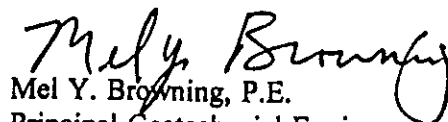
We include an updated boring log for B-5 describing the soil within the undisturbed sampling intervals. Boring B-5 in the original report did not include soil descriptions for the soil materials within these depths.

If you have any questions regarding the above, please contact us at your convenience.

Very truly yours,

LAW ENGINEERING, INC.

  
Randall D. Lipshay, P.E.  
Senior Geotechnical Engineer  
Registered, S.C. 13412

  
Mel Y. Browning, P.E.  
Principal Geotechnical Engineer  
Registered, S.C. 8807





RDL/MYB:lh

## KEY TO CLASSIFICATIONS AND SYMBOLS

### CORRELATION OF PENETRATION RESISTANCE WITH RELATIVE DENSITY AND CONSISTENCY

	<u>No. of Blows, N</u>	<u>Relative Density*</u>
Sands	0 - 4	Very Loose
	5 - 10	Loose
	11 - 20	Firm
	21 - 30	Very Firm
	31 - 50	Dense
	51 +	Very Dense
		<u>Consistency*</u>
Silts and Clays	0 - 1	Very Soft
	2 - 4	Soft
	5 - 8	Firm
	9 - 15	Stiff
	16 - 30	Very Stiff
	31 +	Hard

### SYMBOLS

	- Undisturbed Sample (UD) Recovered
50=2"	- Number of Blows (50) to Drive the Spoon a Number of Inches (2)
BQ,NX,NQ,NW	- Core Barrel Sizes Which Obtain Cores 1-7/16, 2-1/8 Inches, 1-7/8 Inches, 2-1/6 Inches in Diameter, Respectively
65%	- Percentage (65) of Rock Core Recovered (Compared to Cored Length)
RQD	- Rock Quality Designation - Percentage of Recovered Cored Length Consisting of Moderately Hard or Better core Segments 4 or More Inches Long
	- Water Table Approximately 24 Hours or More After Drilling
	- Water Table Approximately at Time of Drilling (Within 1 Hour)
	- Loss of Drilling Fluid
C-	- Borehole Caved at Depth Indicated

\* Terminology may be altered if presence of gravel, cobbles or boulders interferes with accurate measurement of standard penetration resistances.

DEPTH (FT.)	DESCRIPTION	ELEVATION (FT.)	● PENE ATION - BLOWS/FOOT										
			0	10	20	30	40	60	80	100			
0.0	Roadway Fill	565.0±											
		560.0±											
7.0	Alluvium - Soft Brown Micaceous Fine Sandy Silty Clay (Wet)	555.0±	●										4
		550.0±											
12.5	Alluvium - Gray Silty Fine to Medium Sand	545.0±											
		540.0±											
20.5	Residuum - Partially Weathered Rock Sampled as Brown Micaceous Silty Fine to Medium Sand	535.0±											
		530.0±											
25.6	Refusal to Roller cone and Boring Terminated at 25.6 Ft. Ground Water at 5.5 Ft on 6-20-94.												
	NOTE: Boring Drilled on Shoulder of Road. Due to Inaccessibility at Boring Location in Flood Plain.												
40.0													

SEE KEY SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS USED ABOVE

TEST BORING RECORD	
BORING NUMBER	B-5
DATE DRILLED	6-7-94
PROJECT NUMBER	222-07867-01
PROJECT	PLEASANT RD IMPROV.
PAGE 1 OF 1	
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