GRI

Geotechnical Resources, Incorporated
Consulting Engineers, Geologists, and Environmental Scientists

October 12, 1999

3061 GEOTECHNICAL RPT

Yost Grube Hall Architecture
1211 SW Fifth Avenue, Suite 2700
Portland, OR 97204

Attention: John Blumthal

SUBJECT: GEOTECHNICAL INVESTIGATION, PORTLAND EXPOSITION CENTER, HALL D REPLACEMENT, PORTLAND, OREGON

At your request, Geotechnical Resources, Inc. (GRI) has undertaken a geotechnical investigation for the Hall D replacement project at Portland Exposition Center. The general location of the site is shown on the Vicinity Map, Figure 1. The investigation was conducted to evaluate subsurface conditions at the site and develop conclusions and recommendations regarding earthwork and design and construction of foundation support. Our investigation consisted of a review of the available geotechnical information for the area, additional subsurface explorations, laboratory testing, and engineering studies and analyses. This report describes the work accomplished and provides our conclusions and recommendations for design and construction of the project.

As part of our studies, we reviewed the geotechnical and seismic report prepared for the Metropolitan Exposition Recreation Commission by AGRA Earth & Environmental (AGRA) for the recently completed Hall E facility located just south of Hall D. The report is entitled "Geotechnical Investigation, Metropolitan Exposition Center Expansion, Portland, Oregon," dated March 1996.

PROJECT DESCRIPTION

The project consists of removing the existing Hall D and constructing a larger replacement structure. The proposed location and configuration of the project elements are shown on the Site Plan, Figure 2. The facility will be a clear-span, steel-framed structure with a footprint of approximately 400 by 250 ft. Discussions with the project structural engineer, KPFF Consulting Engineers, indicates that maximum column loads will be approximately 600 kips. Cuts and fills will be less than 2 ft. We anticipate that new parking areas and lightly loaded auxiliary structures such as awnings, entryways, and support facilities, will also be constructed for the project. According to KPFF, maximum column loads for the auxiliary structures will be less than 100 kips.

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Topography

The majority of the site surrounding the existing structure is paved with asphaltic-concrete and/or Portland cement concrete pavement. The site is relatively flat, although some shaping has been accomplished to provide drainage. The ground surface across the site varies from about elevation 28 to 30 ft.

Geology

The site is typically mantled with about 10 to 20 ft of fill which is underlain by naturally occurring alluvial silt and sand. The silt and sand is underlain by gravel below a depth of about 120 ft.

SUBSURFACE CONDITIONS

General

A number of deep subsurface explorations have been made just south of project site for the recently completed Hall E facility (AGRA, 1996). For the Hall D project, GRI completed one deep boring, designated B-1; nine shallow borings, designated B-2 through B-10; and two cone penetration test (CPT) probes, designated P-1 and P-2. Boring B-10 was located south of the site in the proposed pavilion area to obtain preliminary subsurface information for future development. The locations of the explorations made by GRI and others in the vicinity of the proposed project are shown on the Site Plan, Figure 2. The deep boring and two probes performed for this project extended to a maximum depth of 81.5 and 120 ft, respectively. The shallow borings were located in the proposed structure and pavement areas and were drilled to a depth of 11.5 to 21.5 ft. A discussion of the field exploration program, together with detailed logs of the borings and probes is provided in Appendix A. Logs of the previous subsurface explorations made by AGRA in the vicinity of the Hall D replacement project are also provided in Appendix A.

The laboratory testing program conducted to evaluate pertinent physical and engineering properties of the soils encountered in the borings is described in Appendix A.

Soils

The subsurface explorations made in the existing paved areas indicate the thickness of the asphaltic-concrete pavement and crushed rock is typically 3 to 4 in. and 10 to 20 in., respectively. The pavement structure is underlain by a layer of sand that varies from about 12.5 to over 20 ft thick. The sand is probably dredge sand fill placed to raise low areas to existing grades. The fill is underlain by predominantly clayey or sandy silt with interbed layers of sand and silty sand. The silt and sand are underlain by gravel at a depth of about 115 to 120 ft.

For the purpose of discussion, the soils disclosed by the explorations have been grouped into the following categories based on their physical characteristics and engineering properties:

1. PAVEMENT
2. SAND FILL
3. Sandy or clayey SILT
4. SAND
5. GRAVEL

A detailed description of each soil unit and a discussion of groundwater conditions at the site is provided below.

1. PAVEMENT. The proposed building area is currently paved with asphaltic concrete with local areas of concrete slabs. The thickness of the pavement at the exploration locations varies from about 3 to 4 in. The pavement is typically underlain by a 10- to 20-in.-thick layer of crushed rock. Borings B-6 and B-8 encountered about 4.5 and 3.5 ft of crushed rock beneath the asphalt, respectively.

Boring B-9, which is located in the southwest corner of the site within an existing fenced storage area that will be paved in the future, encountered an approximately 4-ft-thick layer of dense, crushed rock at the ground surface.

2. SAND FILL. Beneath the pavement section within and in the vicinity of the proposed building footprint, the explorations encountered a variable thickness of dredged sand fill. The sand fill extends to a depth of at least 12.5 ft and typically extends to a depth of 15 to 20 ft. N-values ranging from about 4 to 14 blows/ft and CPT cone penetration resistances ranging from about 20 to 90 tsf indicate the relatively density of the sand fill varies from loose to medium dense; however, the sand is generally medium dense. The sand is generally gray-brown, fine to medium grained, and clean or contains a trace of silt. The natural moisture content of this material ranges from about 5 to 25%. Our past experience in this area indicates the sand fill frequently contains occasional thin layers of silt and silty sand.

3. Sandy or clayey SILT. The sand fill is underlain by sandy or clayey silt interbedded with layers of clean to silty, loose to medium dense sand. The explorations performed within and in the vicinity of the proposed building footprint indicate the silt extends to a depth of about 80 ft below the existing ground surface. CPT sleeve friction resistances ranging from about 0.20 to 1.0 tsf and Torvane shear strength values ranging from 0.40 to 0.45 tsf indicate the relative consistency of the silt ranges from medium stiff to stiff.

4. SAND. The silt is underlain by sand with interbedded layers of silt. The silt content of the sand varies from a trace to silty. This deposit was encountered at a depth of about 80 ft below the existing ground surface. Based on CPT cone penetration resistances of about 100 to 200 tsf, the relative density of the sand is considered medium dense to dense.

5. GRAVEL Logs of subsurface explorations performed by others in the vicinity of the site and the deep CPTs performed by GRI for this project indicate the sand is underlain by gravel at a depth of about 115 to 120 ft.

Groundwater

Our review of available groundwater information from the geotechnical investigation for Hall E and our experience in the vicinity of the site indicate the depth to groundwater typically ranges from about 15 to 20 ft below the existing ground surface. However, the groundwater levels at this site may approach the ground surface during the wet winter and spring months.

CONCLUSIONS AND RECOMMENDATIONS

General

In the vicinity of the proposed Hall D structure, the site is mantled with dredged sand fill underlain by alluvial deposits of compressible silty soils that contain interbeds of sand. Beneath the silty soils is a deposit of generally medium dense to dense sand underlain by gravel.

Our studies indicate the dredged sand fill that caps the site is too thin at some locations and generally too variable in thickness to support the larger column loads without excessive total and differential settlement. Based on our discussions with the project team and our experience with similar projects and subsurface conditions, we recommend that structural loads be supported by piles installed into the underlying silts and sands. Based mainly on economics, driven grout piles were used to support the recently completed Hall E structure located just south of the proposed Hall D. We anticipate support for the new structure will also be provided by driven grout piles.

Our studies indicate there is a significant risk that up to 3 in. of liquefaction-induced settlement could occur on the site during a major seismic event. The risk to the buildings due to liquefaction-induced settlement can be significantly reduced or eliminated if the structure is supported on piles.

The following sections of this report provide our conclusions and recommendations for design and construction of foundations, ground-floor support, and pavements.

Site Preparation and Earthwork

All debris from dismantling the existing structure and any existing concrete slabs or sidewalks, asphaltic-concrete pavement, and other structures demolished for the new construction should be removed from the site. Following clearing, the exposed subgrade should be observed by a geotechnical engineer and/or proof rolled with a loaded, 10 yd^3 dump truck. Soft or loose areas should be overexcavated and replaced with structural fill as described below.

The near-surface sandy fill soils will become disturbed during excavation to subgrade level in the parking or building areas. Following excavation to subgrade level, the upper 12 in. of the exposed sand within structural fill, pavement, and building areas should be compacted to at least 95% of the maximum density as determined by ASTM D 698. Generally, a minimum of six passes with a medium- to heavy-weight, smooth, steel-wheeled, vibratory roller are required to achieve the recommended compaction.

Due to the limited quantity of fill anticipated for the project, we recommend that all structural fill and utility trench backfill placed within building and pavement areas consist of approved on-site granular material or imported granular material. Imported material should have a maximum size of less than 2 in. and contain less than 5% passing the No. 200 sieve. The granular backfill material should be compacted to at least 95% of the maximum dry density as determined by ASTM D 698. Backfill placed in confined areas, such as adjacent to pile caps, grade beams, and utility trenches, and compacted by hand-operated compaction equipment should be placed in maximum 6-in.-thick lifts. In our opinion, flooding or jetting the backfill with water to achieve the recommended compaction should not be permitted.

Foundation Support

General. KPFF has indicated the maximum column loads for the main structure will be about 600 kips, and the maximum column loads for entryways and support structures will be less than 100 kips. In our opinion, support for the main structure should be provided by a deep foundation system that extends into the lower silt and sand that underlie the site. Based on the results of load tests performed on 16-in.-diameter, driven grout piles for the Hall E project and our previous experience with similar projects, we anticipate that 16-in.-diameter driven grout piles will be the most suitable for support of the structure. In our opinion, it is feasible to support the relatively lightly loaded entryways and support buildings on either shallow or deep foundations. We anticipate that the actual type of foundation selected for the lightly loaded structures will depend on several factors, such as tolerable total and differential settlements, economics, and schedule.

Foundation alternatives and associated settlement estimates for pile foundations and spread footings are provided below.

Piles. Allowable load capacities for piles will depend on pile diameter and/or size and depth of embedment. The following table summarizes our recommended allowable compression (downward) and uplift capacities for 16-in.-diameter driven grout piles. We acknowledge that other capacities are possible with different pile sections, lengths, and piles; however, the following pile type has been successfully used at this site in the past.

Pile Type	Embedment Beneath Bottom of Pile Cap, ft	Allowable Capacity, tons	
		Compression	Tension
16-in.-diameter Driven Grout Pile	60*	70	45
	80	100	65

*Minimum Embedment Depth Due to Liquefaction Considerations

In our opinion, the pile-driving operations should be observed on a full-time basis, and a continuous record of pile installation resistance versus depth of penetration should be maintained for each pile. All pile driving records should be reviewed by the geotechnical engineer.

The allowable capacities refer to real loads, i.e., the total of dead load plus frequently or permanently applied live loads. This value can be increased by one-third for the total of all loads; dead, live, and transient (wind or seismic). The allowable pile capacities are based on soil support considerations and include an estimated factor of safety of at least 2. The structural strength of the pile may limit the allowable capacities to lower values. We anticipate that the settlement of driven piles installed in accordance with the criteria presented herein will be less than 1 in.

Lateral structural loads can be resisted by piles in bending and the passive resistance of the soil adjacent to the pile cap. The lateral capacities and the corresponding estimated horizontal deflections for the existing and proposed pile types for the project are summarized in the following table. These estimated deflections assume that a fixed-end condition will be developed at the pile-to-pile cap connection. Additional lateral resistance will be developed by passive resistance developed by compacted granular fill adjacent to the pile cap and connecting grade beams. Passive soil resistance can be evaluated using an equivalent fluid pressure of 300 pcf. This value assumes that pile cap excavations will be backfilled with granular material, such as sand, sandy gravel, or crushed rock up to 2-in. maximum size, containing less than 5% fines (washed analysis), and compacted to at least 95% of the maximum dry density as determined by ASTM D 698.

	Lateral Single Pile Capacity, tons		Lateral Group Capacity, tons	
Pile Type	1/4-in. deflection	1/2 -in. deflection	1/4-in. deflection	1/2-in. deflection
16-in.-diameter Driven Grout Pile	6.0	11.0	4.0	8.5

Installation Criteria for Driven Piles. Piles should be installed with a center-to-center spacing of at least three pile diameters. The piles may be driven with an air, steam, or diesel hammer exerting at least 24,000 ft-lb of energy per blow, or as necessary to achieve the required pile tip elevation. It should be noted that the pile penetration criteria may be modified, in part, on the basis of pile installation resistance observed during the installation of production piles.

Shallow Foundations. In our opinion, the structural loads of the proposed lightly loaded structures, i.e., maximum column and continuous footing loads less than 100 kips and 4 kips/ft, respectively, can be supported on conventional spread footing foundations constructed in accordance with the following design criteria. Footings should be established in well-compacted, on-site granular material. Due to the presence of fill on the site, a qualified geotechnical engineer should observe the footing subgrades at the time of excavation. Footings should be established at a minimum depth of 18 in. below the lowest adjacent finished grade. In addition, isolated and continuous footings should have a minimum width of at least 2 and 1 1/2 ft, respectively. We recommend the use of a smooth-edged excavator to make the footing excavations. It is likely the footing subgrade in the sandy soils will be disturbed during excavation.

Therefore, the bottom of all footing excavations in sand should be wetted, if necessary, and compacted with several passes of a heavy, hand-operated vibratory plate compactor immediately prior to placing the reinforcing steel for the footing. Footings established in accordance with these criteria can be designed on the basis of an allowable soil bearing pressure of 2,000 psf. This value applies to the total of dead loads plus frequently and/or permanently applied live loads and can be increased by one-third for the total of all loads; dead, live, and wind or seismic.

The total settlement of footings due to static loads designed in accordance with the recommendations presented above is estimated to be less than 1 in. Due to the granular nature of the underlying soils, settlements will occur rapidly as the structural loads are applied. Differential settlements between adjacent foundation units should be less than half the total settlement. Estimated settlements due to liquefaction after a UBC zone 3 seismic event could be on the order of 3 in.

Horizontal shear forces can be resisted partially or completely by frictional forces developed between the base of spread footings and the underlying soil and by soil passive resistance. The total frictional resistance between the footing and soil is the normal force times the coefficient of friction between the soil and the base of the footing. We recommend a value of 0.40 for the coefficient of friction; the normal force is the sum of the vertical forces (dead load plus real live load). If additional lateral resistance is required, passive earth pressures against embedded footings can be computed on the basis of an equivalent fluid having a unit weight of 300 pcf. This design passive earth pressure would be applicable only if the footing is cast neat against undisturbed soils or if backfill for the footings is placed as granular structural fill.

Floor Slab

We anticipate that the slab-on-grade in the main structure may be subjected to heavy wheel loads from fork lifts and vehicles. Therefore, we recommend the floor slab be underlain by a minimum 8-in. thickness of relatively clean granular base course material. Suitable base course can consist of ¾-in.-minus crushed rock having less than 5% passing the No. 200 sieve (washed analysis). Base course material should be compacted to at least 95% of the maximum density as determined by ASTM D 698. In our opinion, it is appropriate to assume a coefficient of subgrade reaction of 175 pci for the design of floor slab constructed as recommended above.

In addition, it may be appropriate to install a durable vapor-retarding membrane beneath the slab-on-grade floor to limit the risk of damp floors in areas that will have moisture-sensitive materials placed directly on the floor. The vapor-retarding membrane should be installed in accordance with the manufacturer's recommendations.

Pavement Section

Based on our understanding of the project and our experience with similar projects, we recommend the following minimum pavement sections:

	Minimum Thickness of Crushed Rock Base Course, in.	Minimum Thickness of Asphaltic Concrete Pavement, in.
Automobile Parking: No Heavy Truck Traffic	6	2.5
Automobile Parking and Areas Subjected to Minor Truck Traffic	8	3
Heavy Truck Traffic	12	4

These sections assume that pavement subgrade consists of sand, and the upper 12 in. of subgrade and base course are compacted to at least 95% of the maximum dry density as determined by ASTM D 698. These sections also assume that all workmanship and materials conform to the standards of the Oregon Department of Transportation.

Seismic Considerations

The project site is presently assigned to seismic zone 3 in the Uniform Building Code (UBC, 1997). Based on the subsurface conditions at the site and our review of the UBC and the site specific seismic analysis performed for the Hall E site, we recommend using a site coefficient S_E to evaluate the seismic design of the structure.

Based on our review of the subsurface conditions at the Hall E and Hall D sites, it is our opinion that the results of the site-specific seismic study performed for Hall E is applicable to the design of this project (AGRA, 1996).

Based on the results of this study and our previous work in the project area, we anticipate that a portion of the silt and sand below the water table is susceptible to liquefaction during a moderate to large seismic event. Liquefaction of the lower sands and silt would probably result in some liquefaction-induced settlement of the ground surface. The amount of settlement would depend on the earthquake magnitude and the peak ground acceleration at the site. The results of our studies indicate that up to 3 in. of liquefaction-induced ground settlements could occur on the site during a major UBC zone 3 seismic event. Due to the predominantly silty nature of the soils beneath the site and the distance of the site from a free face, we anticipate the risk of lateral spreading at the site is low. Based on our studies, it is our opinion that the potential for earthquake-induced fault displacement, landslides, and damage by tsunamis and/or seiches at this site is low.

In our opinion, the risk to the building due to liquefaction will be significantly reduced or eliminated if the structure is supported on piles.

Design Review and Construction Inspection

We welcome the opportunity to review and discuss construction plans and specifications as they are being developed. Additionally, we are of the opinion that to observe compliance with the design concepts, specifications, and recommendations, all construction operations dealing with earthwork and foundations should be observed by a qualified geotechnical engineer or engineering geologist. We would be pleased to provide these services to you.

LIMITATIONS

This report has been prepared to aid the project team in the design of this project. The scope is limited to the specific project and location described herein, and our description of the project represents our understanding of the significant aspects of the project relevant to the design and construction of the earthwork, foundations, and pavements. In the event that any changes in the design and location of improvements as outlined in this report are planned, we should be given the opportunity to review the changes and to modify or reaffirm the conclusions and recommendations of this report in writing.

The conclusions and recommendations submitted in this report are based on the data obtained from the borings and probes performed at the locations indicated on Figure 2 and from other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in soil conditions may exist between exploration locations. This report does not reflect any variations that may occur between these explorations. The nature and extent of variation may not become evident until construction. If, during construction, subsurface conditions different from those encountered in the explorations are observed or encountered, we should be advised at once so that we can observe and review these conditions and reconsider our recommendations where necessary.

Please contact the undersigned if you have any questions regarding this report.

Sincerely,

GEOTECHNICAL RESOURCES, INC.

REGISTERED PROFESSIONAL ENGINEER
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OREGON
MARCH 16, 1988
MICHAEL W. REED

Michael W. Reed
12/00

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H. STANLEY KELSAY

H Stanley Kelsay
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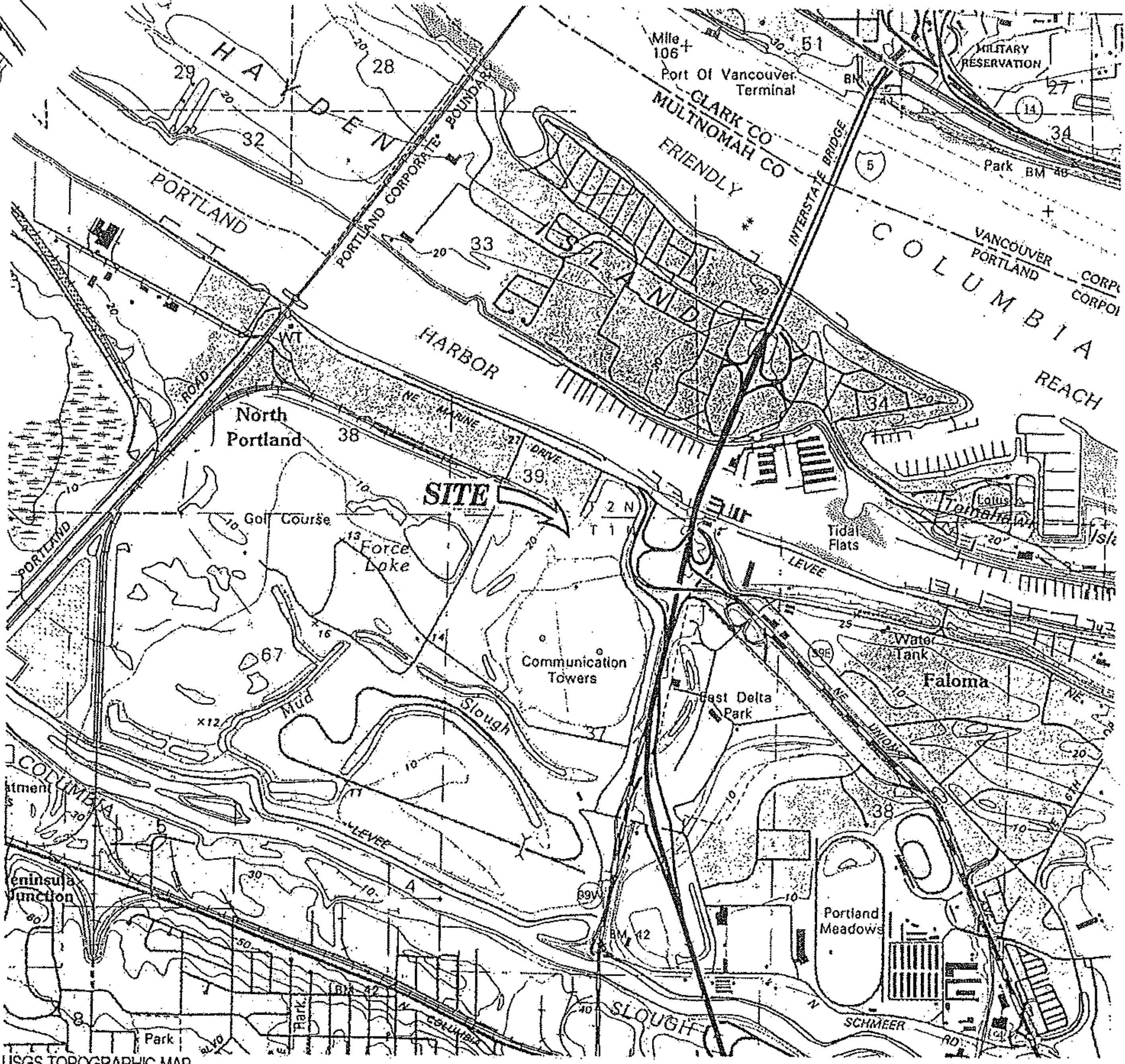
Michael W. Reed, P.E.
Associate

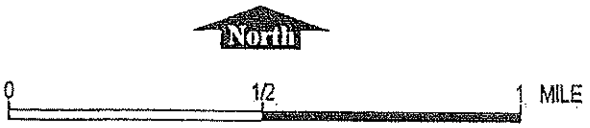
H. Stanley Kelsay, P.E.
Principal

Reference

AGRA Earth & Environmental, March 1996, "Geotechnical Investigation, Metropolitan Exposition Center Expansion, Portland, Oregon," prepared for the Metropolitan Exposition Recreation Commission.



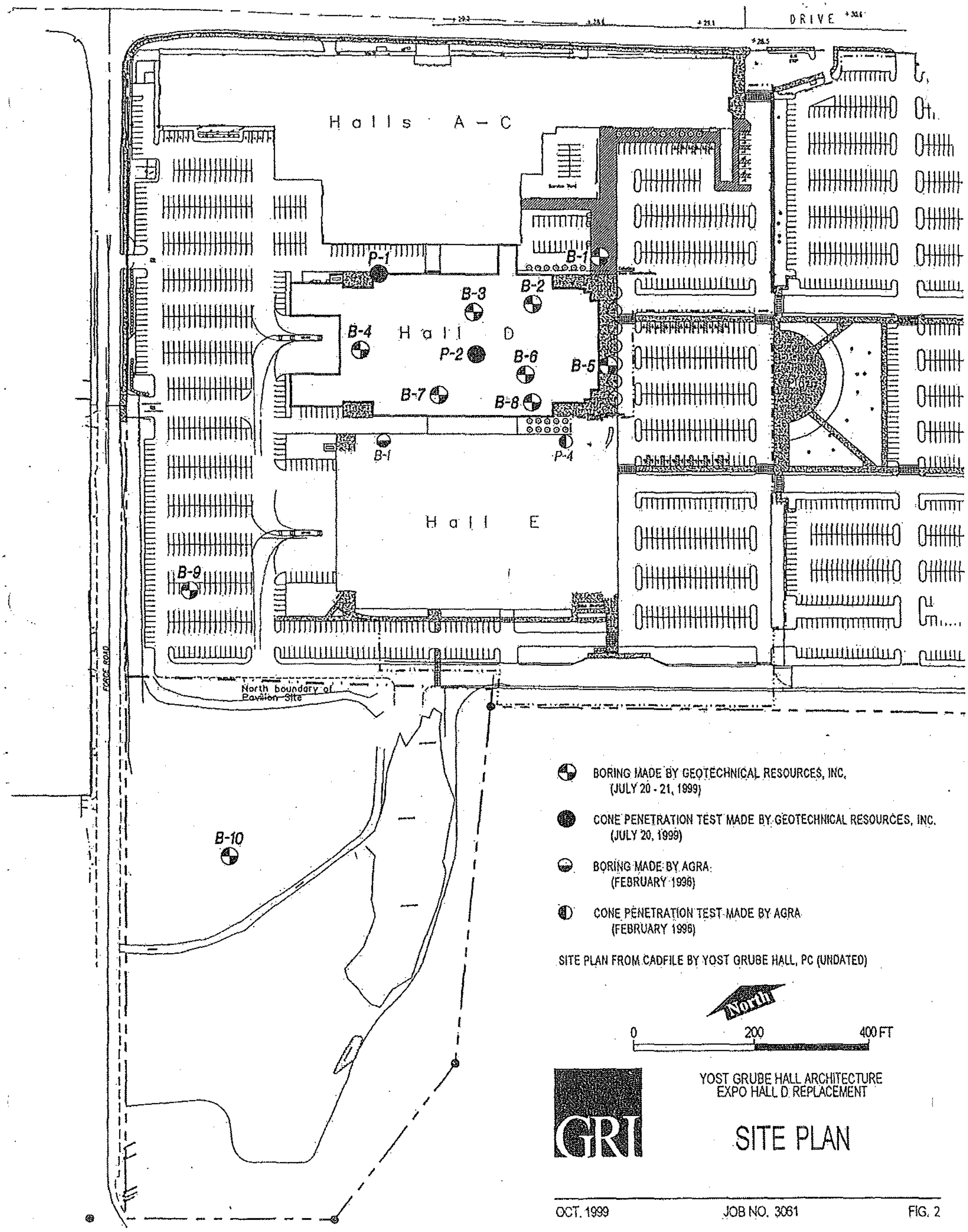
USGS TOPOGRAPHIC MAP
PORTLAND, OREG. (2ad) QUAD (1990)



YOST GRUBE HALL ARCHITECTURE
EXPO HALL D REPLACEMENT

VICINITY MAP

OCT. 1999 JOB NO. 3061 FIG. 1



⊕ BORING MADE BY GEOTECHNICAL RESOURCES, INC.
(JULY 20 - 21, 1999)

● CONE PENETRATION TEST MADE BY GEOTECHNICAL RESOURCES, INC.
(JULY 20, 1999)

◒ BORING MADE BY AGRA
(FEBRUARY 1996)

◐ CONE PENETRATION TEST MADE BY AGRA
(FEBRUARY 1996)

SITE PLAN FROM CADFILE BY YOST GRUBE HALL, PC (UNDATED)

GRI

YOST GRUBE HALL ARCHITECTURE
EXPO HALL D REPLACEMENT

SITE PLAN

OCT. 1999 JOB NO. 3061 FIG. 2

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

General

Subsurface materials and conditions in the project area were investigated on July 20 and 21, 1999, with 10 borings, designated B-1 through B-10, and two cone penetration test probes, designated P-1 and P-2. The borings were drilled to depths of 11.5 to 81.5 ft; the probes were extended to a maximum depth of 120 ft. The locations of the explorations are shown on the Site Plan, Figure 2.

Borings

The borings were made using hollow-stem and mud-rotary techniques with a truck-mounted CME-55 drill rig provided and operated by Geo-Tech Explorations, Inc. of Tualatin, Oregon. The borings were observed by a geotechnical engineer provided by our firm who maintained a detailed log of the conditions and materials encountered and collected representative soil samples. Disturbed and undisturbed samples were obtained from the borings at 2.5- to 5-ft intervals of depth. Disturbed samples were obtained using a standard split-spoon sampler. At the time of sampling, the Standard Penetration Test (SPT) was conducted. This test consists of driving a standard split-spoon sampler into the soil a distance of 18 in. (or to refusal), using a 140-lb hammer dropped 30 in. The number of blows required to drive the sampler the last 12 in. is known as the standard penetration resistance, or N-value. N-values, or blow counts, provide a measure of compactness of granular soils, such as sand, and the degree of softness or stiffness of cohesive soils, such as clays or silts. Samples obtained in the split-spoon sampler were saved in airtight plastic jars for further examination and physical testing in our laboratory. In addition, relatively undisturbed Shelby tube samples were collected and returned to our laboratory.

Cone Penetration Test Probes

The cone penetration tests were performed and interpreted by Vandehey Soil Explorations of Banks, Oregon. The cone penetration test consists of forcing a hardened steel cone vertically into the soil at a constant rate of penetration. The thrust required to cause penetration at a constant rate can be related to the bearing capacity of the soil immediately surrounding the point of the penetrometer cone. This value is known as the cone penetration resistance. After making the cone thrust measurement, a measurement is obtained of the magnitude of thrust required to force a special friction sleeve, attached above the cone, through the soil. The thrust required to move the friction sleeve can be related to the undrained shear strength of fine-grained soils. The dimensionless ratio of sleeve friction to point bearing capacity provides an indication of the type of soil penetrated. The cone penetration resistance and the sleeve friction are determined at about 8-in. intervals in the probe hole and can be used to evaluate the relative density of cohesionless soils and the relative consistency of cohesive soils, respectively.

Logs of Subsurface Explorations

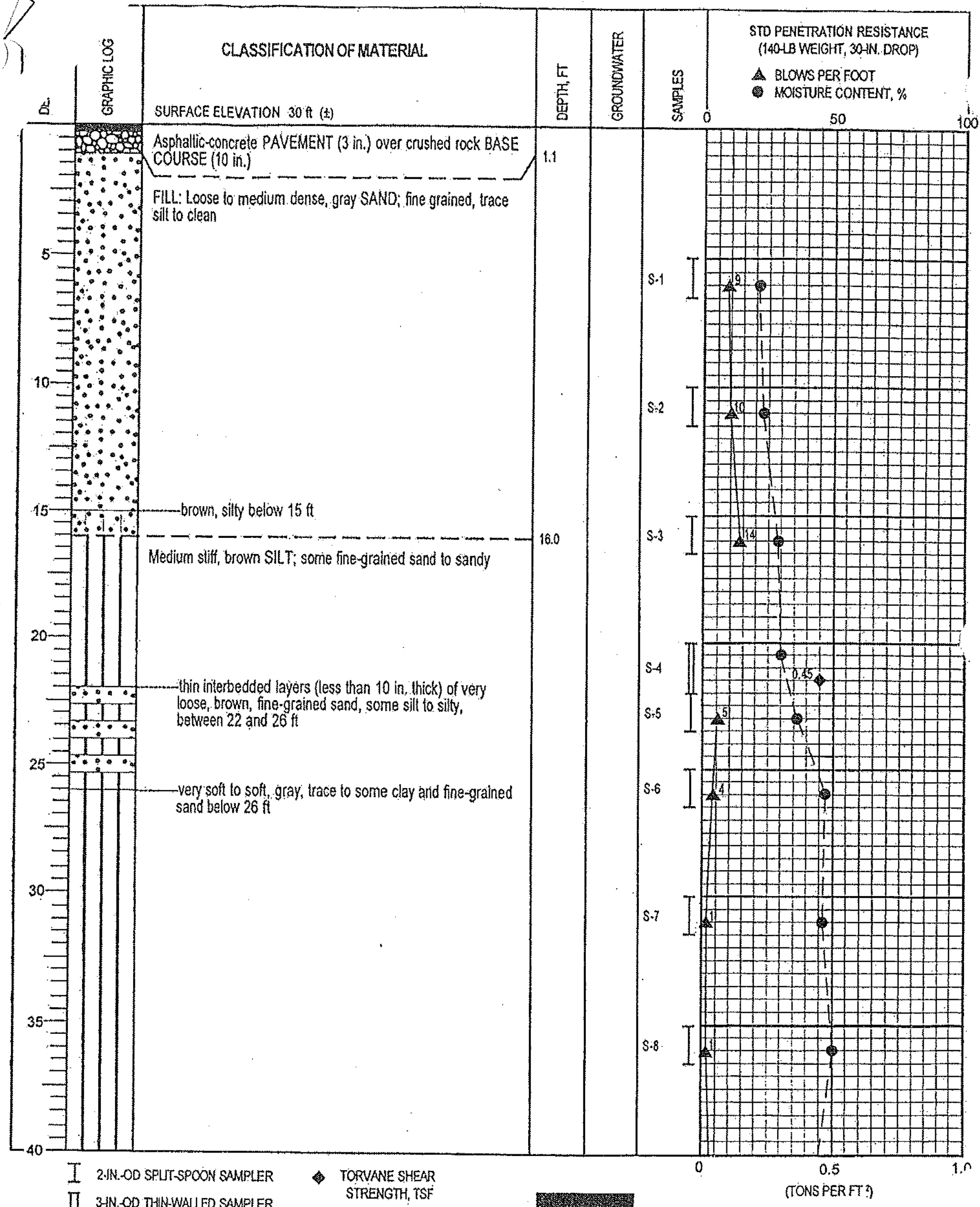
The logs of borings B-1 through B-10 are provided on Figures 1A through 10A. The logs of cone penetration test probes P-1 and P-2 are provided on Figure 11A and 12A. Each log presents a descriptive summary of the various types of materials encountered in the explorations and notes the depths where the materials and characteristics of the materials change. The boring logs show the depths and types of samples taken, along with natural moisture contents, standard penetration resistance, and Torvane shear strengths. The terms used to describe the soils are defined in Tables 1A and 2A.

LABORATORY TESTING

General

The samples obtained from the borings were examined in our laboratory. The physical characteristics of the samples were noted, and the field classifications were modified where necessary. At the time of classification, the natural moisture content of each sample was determined in conformance with ASTM D 2216. The results are summarized on the Boring Logs, Figures 1A through 10A.

The approximate undrained shear strength of the fine-grained soils obtained in the Shelby tubes was determined using the Torvane shear device. The Torvane is a hand-held apparatus with vanes which are inserted into the soil. The torque required to fail the soil in shear around the vanes is measured using a calibrated spring. The results of this testing are summarized on the Boring Logs, Figures 1A through 10A.



Asphaltic-concrete PAVEMENT (3 in.) over crushed rock BASE COURSE (10 in.)

FILL: Loose to medium dense, gray SAND; fine grained, trace silt to clean

—brown, silty below 15 ft

Medium stiff, brown SILT; some fine-grained sand to sandy

—thin interbedded layers (less than 10 in. thick) of very loose, brown, fine-grained sand, some silt to silty, between 22 and 26 ft

—very soft to soft, gray, trace to some clay and fine-grained sand below 26 ft

I 2-IN.-OD SPLIT-SPOON SAMPLER	◆ TORVANE SHEAR STRENGTH, TSF
II 3-IN.-OD THIN-WALLED SAMPLER	▨ UNDRAINED SHEAR STRENGTH, TSF
G GRAB SAMPLE OF DRILL CUTTINGS	* NO RECOVERY
▮ NX CORE RUN	Liquid Limit Moisture Content Plastic Limit
SLOTTED PVC PIPE	
▼ Water Level (date)	

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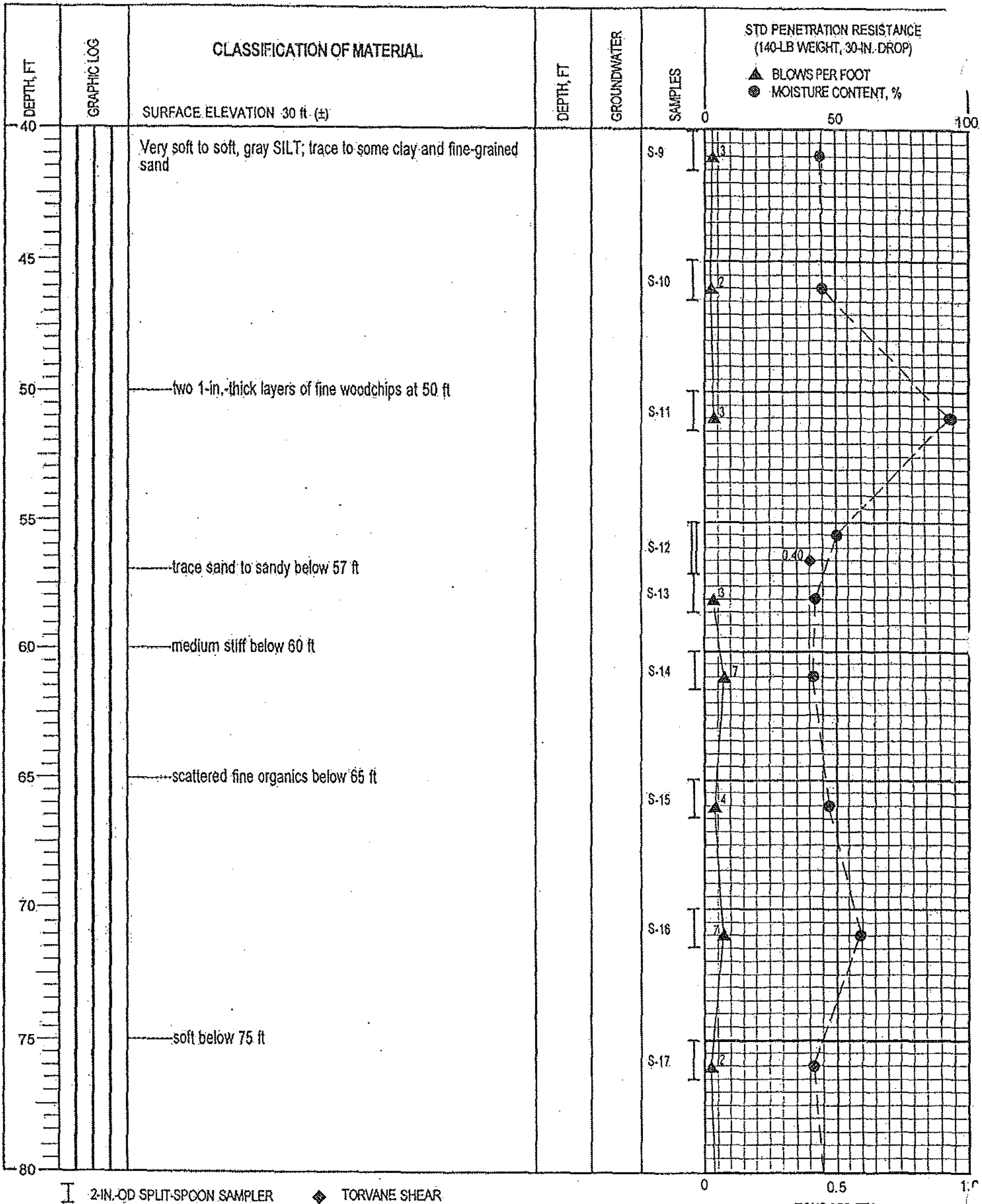
BORING B-1

OCT. 1999

JOB NO. 3061

FIG. 1A



Very soft to soft, gray SILT; trace to some clay and fine-grained sand

two 1-in.-thick layers of fine woodchips at 50 ft

trace sand to sandy below 57 ft

medium stiff below 60 ft

scattered fine organics below 65 ft

soft below 75 ft

I 2-IN.-OD SPLIT-SPOON SAMPLER

II 3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

▮ NX CORE RUN

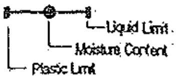
|— SLOTTED PVC PIPE

▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

■ UNDRAINED SHEAR STRENGTH, TSF

* NO RECOVERY



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BORING B-1 (cont.)

OCT. 1999

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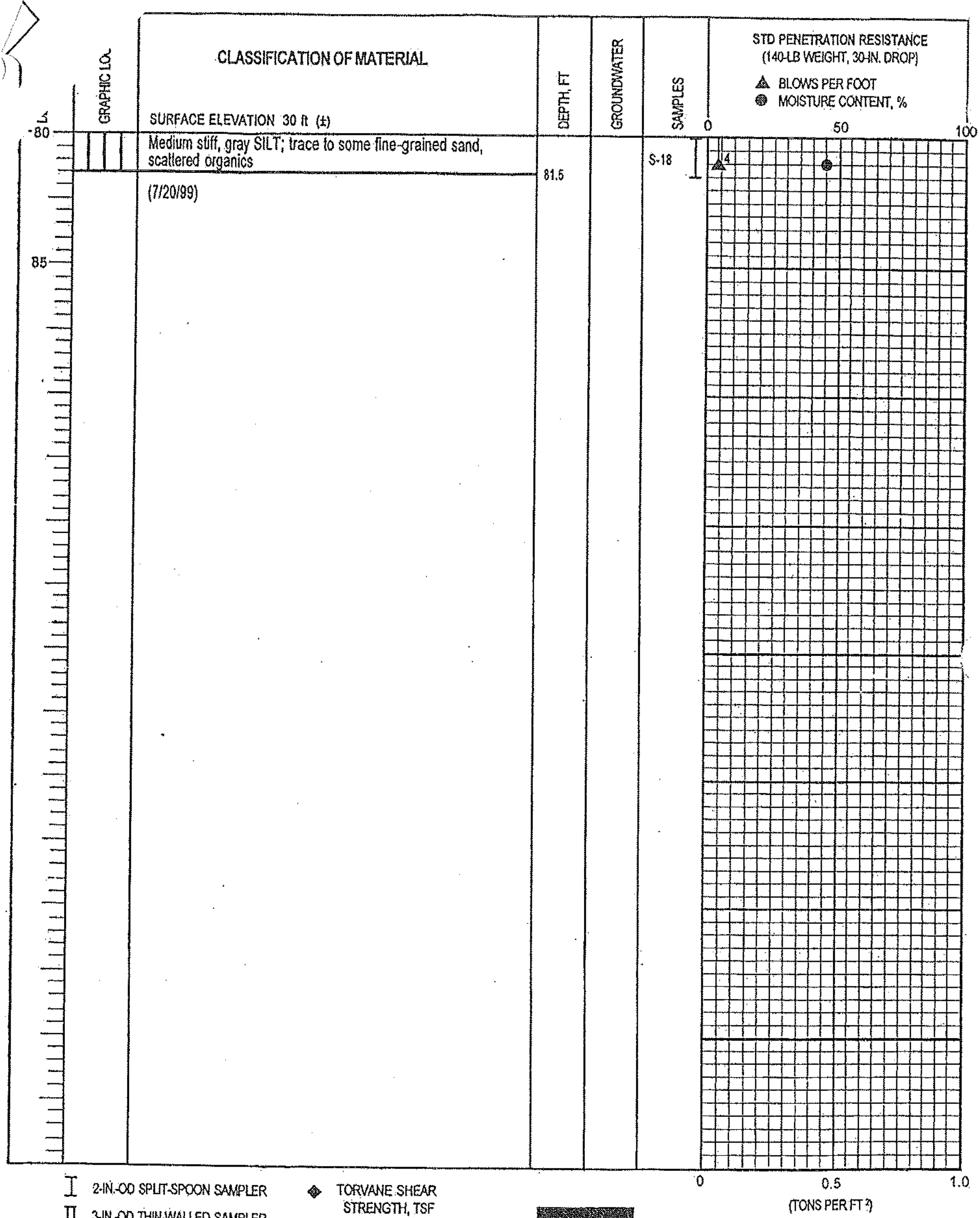
FIG. 1A



CLASSIFICATION OF MATERIAL

SURFACE ELEVATION 30 ft (±)

Medium stiff, gray SILT; trace to some fine-grained sand, scattered organics

(7/20/99)

- Ɪ 2-IN.-OD SPLIT-SPOON SAMPLER
- Ⅱ 3-IN.-OD THIN-WALLED SAMPLER
- G GRAB SAMPLE OF DRILL CUTTINGS
- ▮ NX CORE RUN
- ⊢ SLOTTED PVC PIPE
- ▼ Water Level (date)
- ◈ TORVANE SHEAR STRENGTH, TSF
- ▩ UNDRAINED SHEAR STRENGTH, TSF
- * NO RECOVERY
- Plastic Limit — Moisture Content — Liquid Limit

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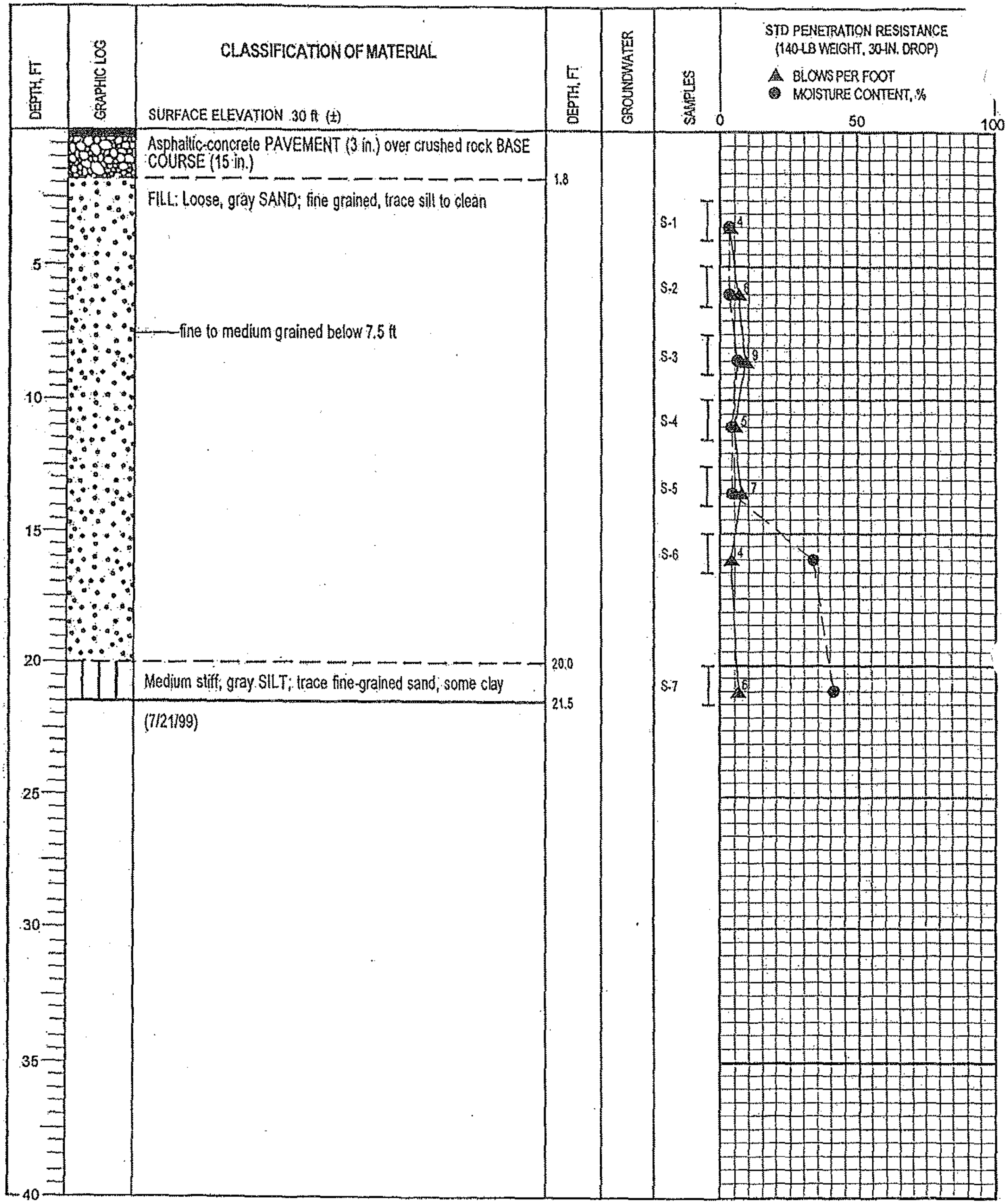
BORING B-1 (cont.)

OCT. 1999 JOB. NO. 3061 FIG. 1A



I 2-IN.-OD SPLIT-SPOON SAMPLER

II 3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

▮ NX CORE RUN

|— SLOTTED PVC PIPE

▽ Water Level (date)

◈ TORVANE SHEAR STRENGTH, TSF

▨ UNDRAINED SHEAR STRENGTH, TSF

* NO RECOVERY

Liquid Limit

Moisture Content

Plastic Limit

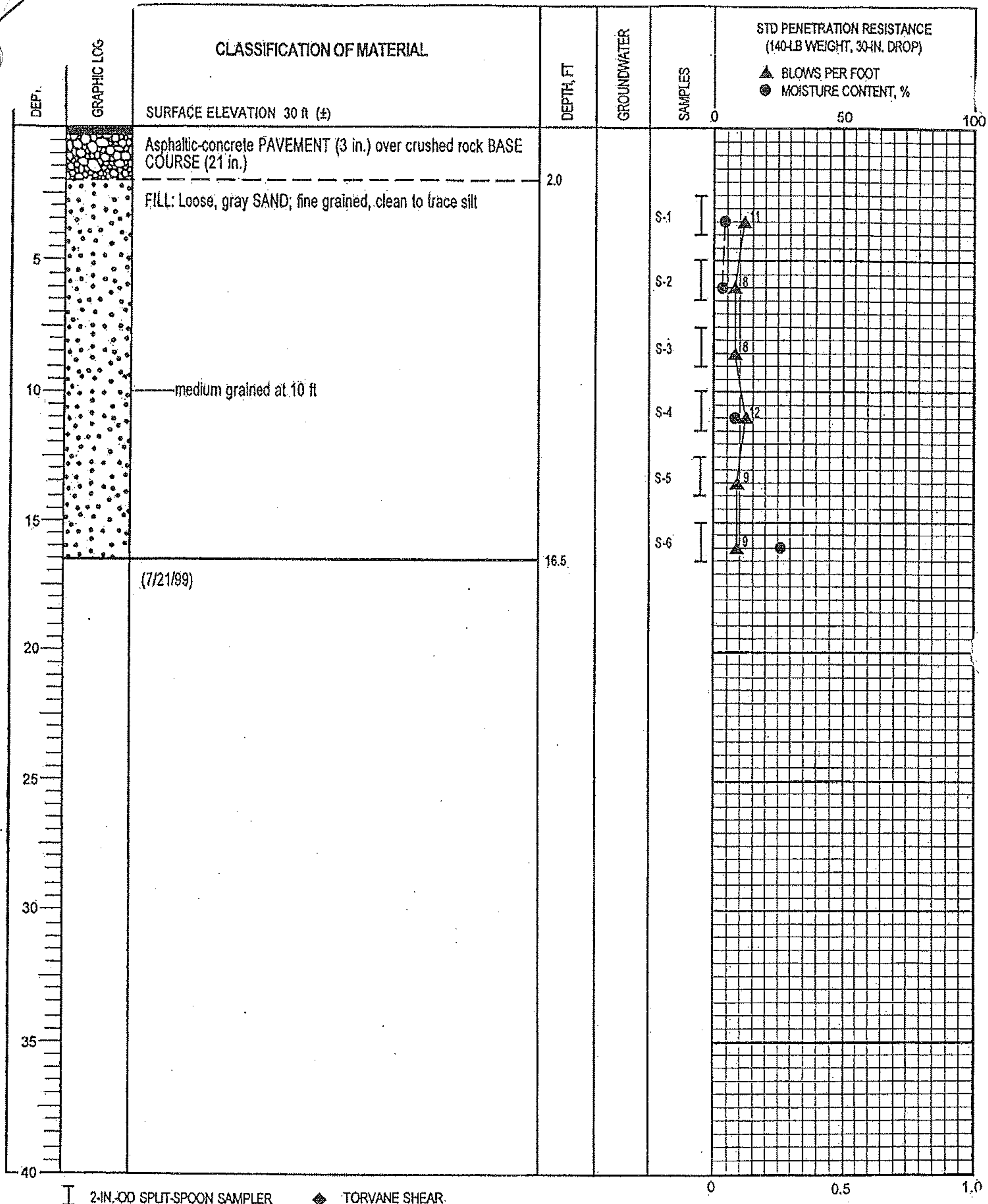
GRI

BORING B-2

OCT. 1999

JOB. NO. 3061

FIG. 2A



I 2-IN.-OD SPLIT-SPOON SAMPLER

II 3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

▮ NX CORE RUN

|—SLOTTED PVC PIPE

▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

■ UNDRAINED SHEAR STRENGTH, TSF

* NO RECOVERY

Liquid Limit

Moisture Content

Plastic Limit

GRI

BORING B-3

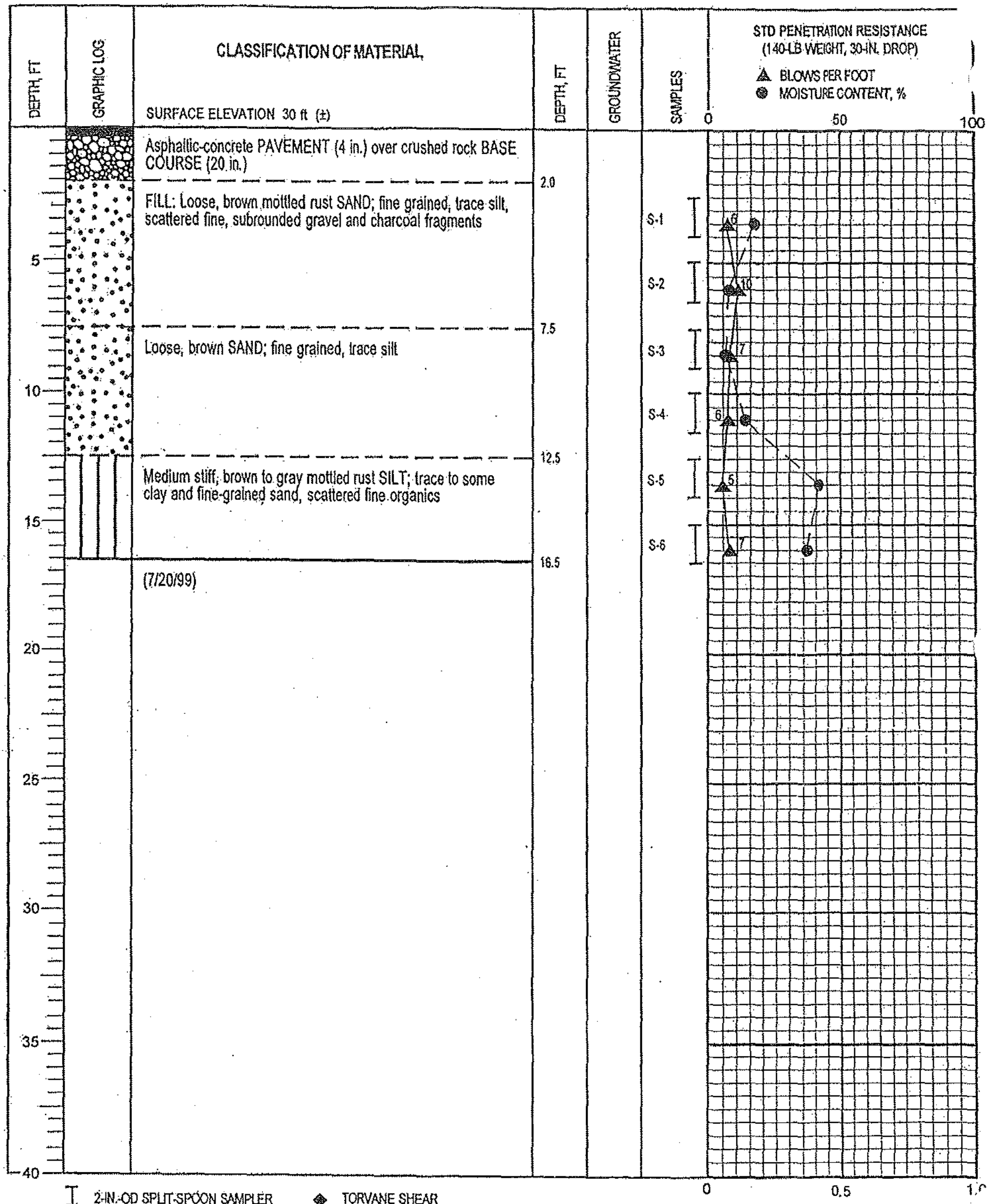
OCT. 1999

JOB. NO. 3061

FIG. 3A

SURFACE ELEVATION 30 ft (±)

DEPTH, FT	CLASSIFICATION OF MATERIAL	DEPTH, FT	GROUNDWATER	SAMPLES	STD PENETRATION RESISTANCE (140-LB WEIGHT, 30-IN. DROP) BLOWS PER FOOT
	Asphaltic-concrete PAVEMENT (4 in.) over crushed rock BASE COURSE (20 in.)	2.0			
	FILL: Loose, brown mottled rust SAND; fine grained, trace silt, scattered fine, subrounded gravel and charcoal fragments	7.5		S-1 S-2	6 10
	Loose, brown SAND; fine grained, trace silt	12.5		S-3 S-4	7 6
	Medium stiff, brown to gray mottled rust SILT; trace to some clay and fine-grained sand, scattered fine organics	16.5		S-5 S-6	5 7
	(7/20/99)				



I 2-IN.-OD SPLIT-SPOON SAMPLER

II 3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

▮ NX CORE RUN

|—SLOTTED PVC PIPE

▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

▨ UNDRAINED SHEAR STRENGTH, TSF

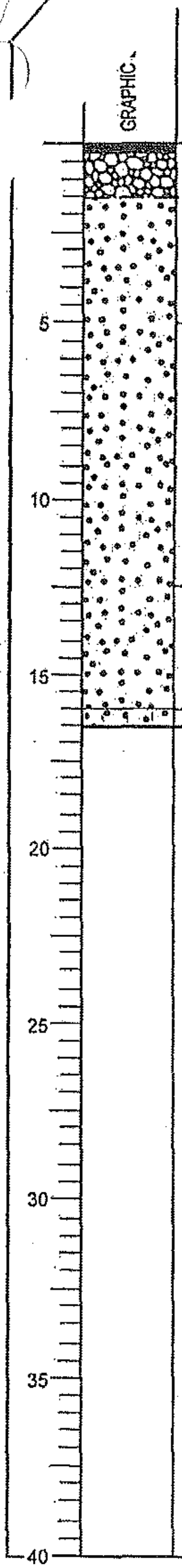
* NO RECOVERY

Liquid Limit
Moisture Content
Plastic Limit

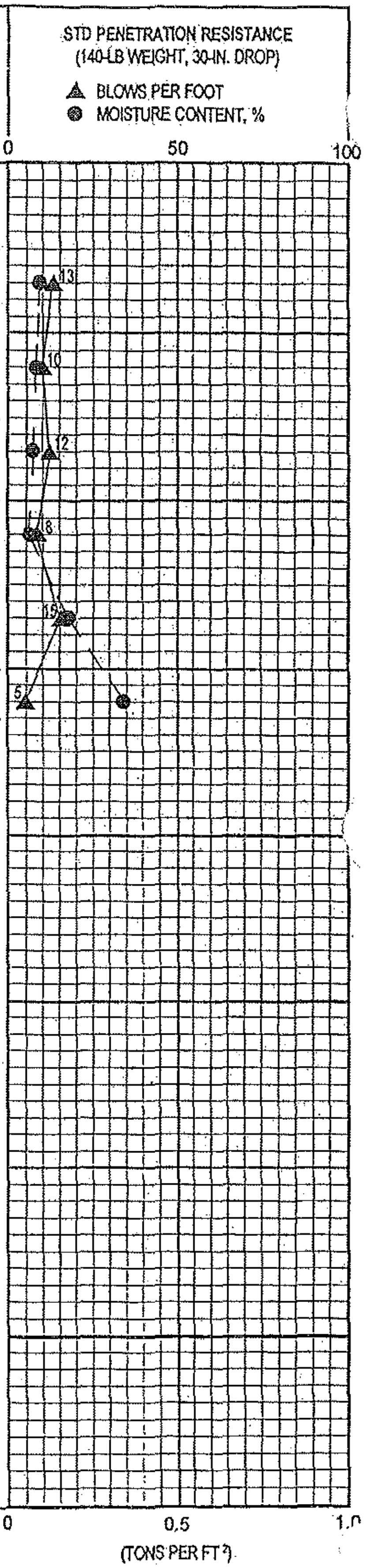
GRI

BORING B-4

OCT. 1999 JOB. NO. 3061 FIG. 4A



CLASSIFICATION OF MATERIAL SURFACE ELEVATION 28 ft (±)	DEPTH, FT	GROUNDWATER	SAMPLES
Asphaltic-concrete PAVEMENT (3 in.) over crushed rock BASE COURSE (15 in.)	1.5		
FILL: Loose to medium dense, brown SAND; fine to medium grained, trace silt			S-1
—gray below 5 ft			S-2
			S-3
			S-4
—brown mottled rust below 12.5 ft			S-5
—silty below 16 ft	16.5		S-6
(7/20/99)			



I 2-IN.-OD SPLIT-SPOON SAMPLER
II 3-IN.-OD THIN-WALLED SAMPLER
G GRAB SAMPLE OF DRILL CUTTINGS
▮ NX CORE RUN
|— SLOTTED PVC PIPE
▼ Water Level (date)

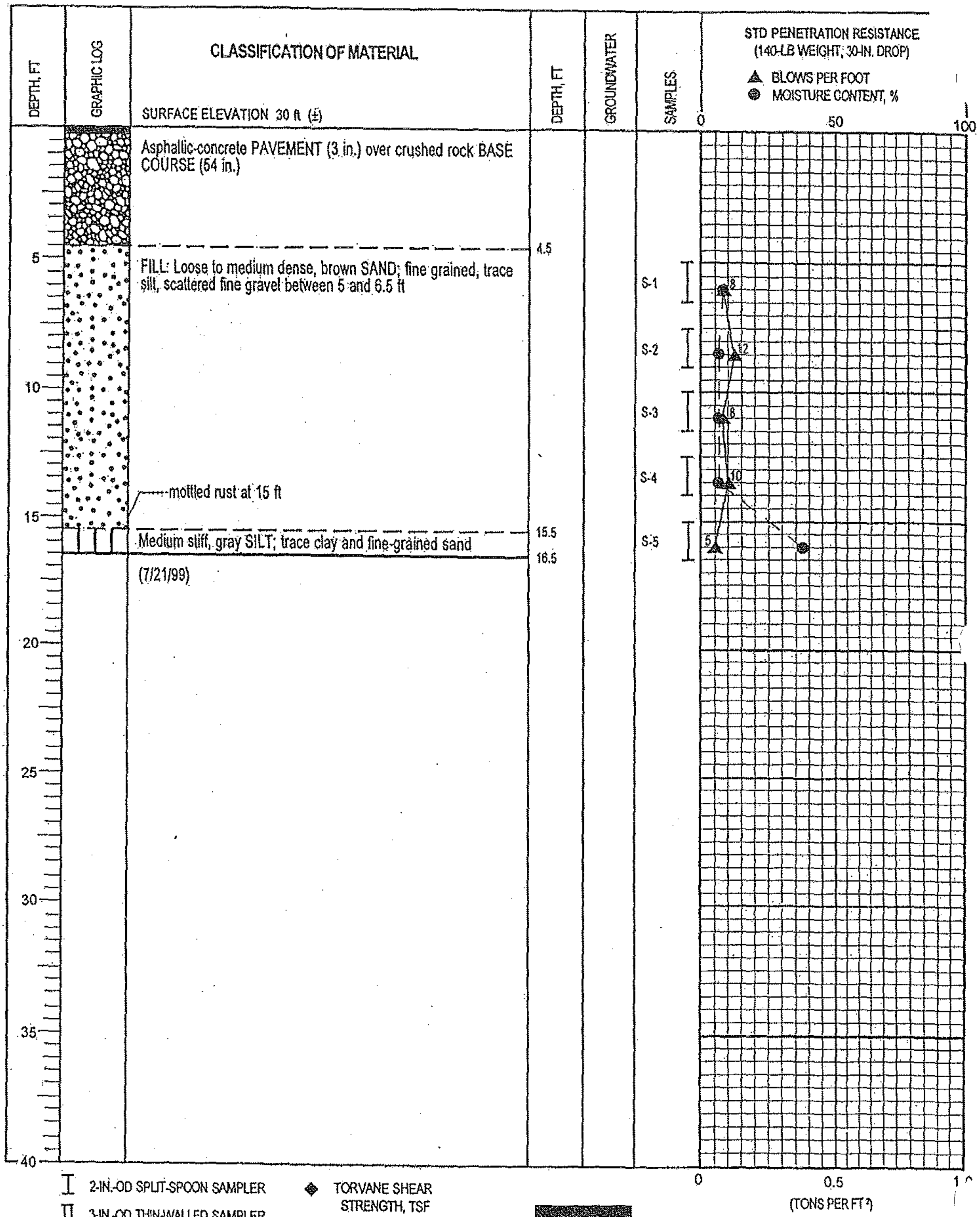
◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF
* NO RECOVERY

Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-5

OCT. 1999 JOB. NO. 3061 FIG. 5A



CLASSIFICATION OF MATERIAL

SURFACE ELEVATION 30 ft (±)

Asphaltic-concrete PAVEMENT (3 in.) over crushed rock BASE COURSE (54 in.)

4.5

FILL: Loose to medium dense, brown SAND; fine grained, trace silt, scattered fine gravel between 5 and 6.5 ft

mottled rust at 15 ft

15.5

Medium stiff, gray SILT; trace clay and fine-grained sand

16.5

(7/21/99)

- I 2-IN.-OD SPLIT-SPOON SAMPLER
- II 3-IN.-OD THIN-WALLED SAMPLER
- G GRAB SAMPLE OF DRILL CUTTINGS
- ▮ NX CORE RUN
- |—— SLOTTED PVC PIPE
- ▼ Water Level (date)
- ◆ TORVANE SHEAR STRENGTH, TSF
- ▩ UNDRAINED SHEAR STRENGTH, TSF
- * NO RECOVERY
- Liquid Limit, Moisture Content, Plastic Limit

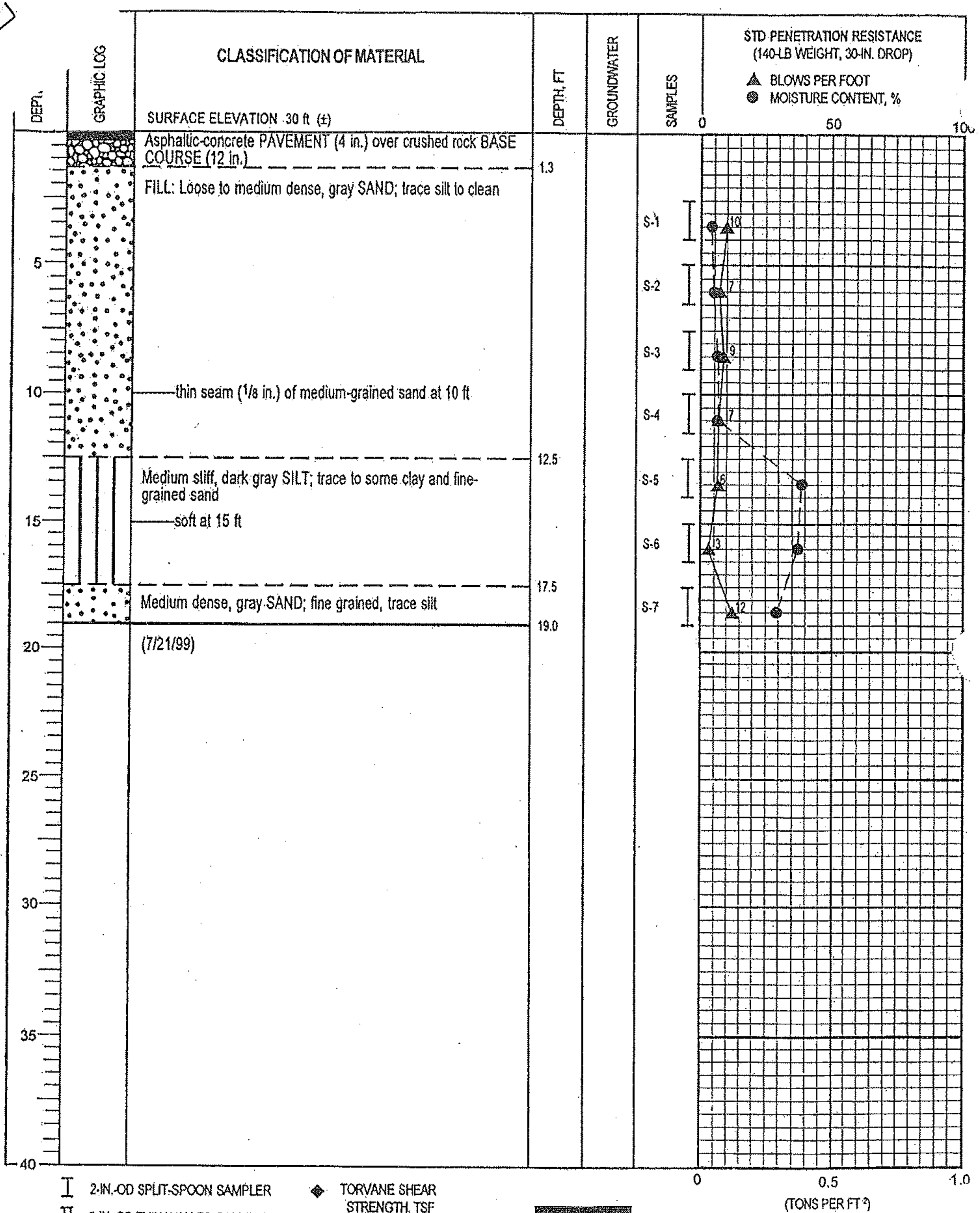
GRI

BORING B-6

OCT. 1999

JOB. NO. 3061

FIG. 6A



CLASSIFICATION OF MATERIAL

SURFACE ELEVATION 30 ft (±)

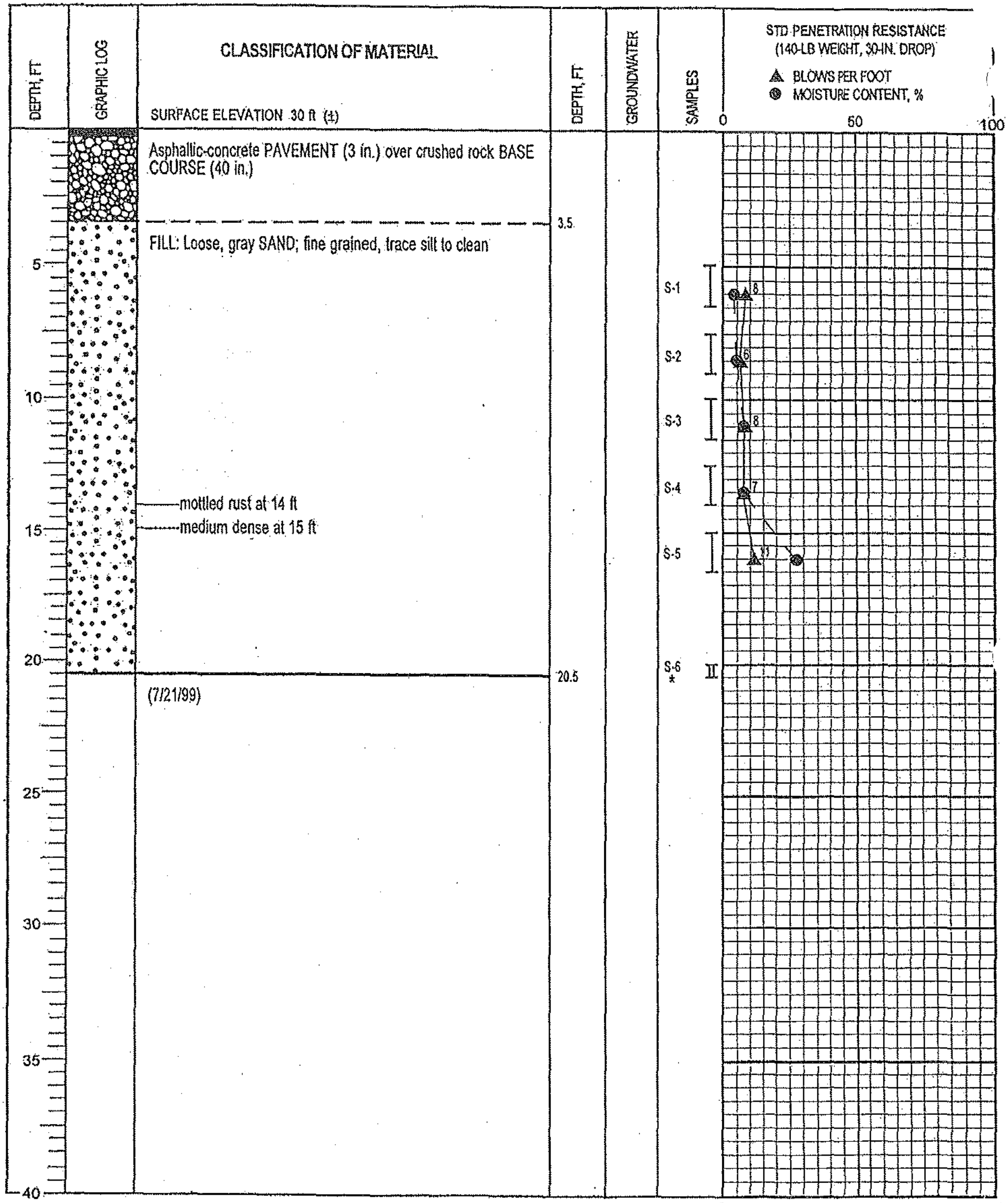
Classification of material	Depth, ft
Asphaltic-concrete PAVEMENT (4 in.) over crushed rock BASE COURSE (12 in.)	1.3
FILL: Loose to medium dense, gray SAND; trace silt to clean ——thin seam (1/8 in.) of medium-grained sand at 10 ft	12.5
Medium stiff, dark gray SILT; trace to some clay and fine-grained sand ——soft at 15 ft	17.5
Medium dense, gray SAND; fine grained, trace silt	19.0
(7/21/99)	

- I 2-IN.-OD SPLIT-SPOON SAMPLER
- II 3-IN.-OD THIN-WALLED SAMPLER
- G GRAB SAMPLE OF DRILL CUTTINGS
- ▮ NX CORE RUN
- ◆ TORVANE SHEAR STRENGTH, TSF
- ■ UNDRAINED SHEAR STRENGTH, TSF
- * NO RECOVERY
- SLOTTED PVC PIPE
- ▼ Water Level (date)
- Liquid Limit, Moisture Content, Plastic Limit

GRI

BORING B-7

OCT. 1999 JOB NO. 3061 FIG. 7A



Asphaltic-concrete PAVEMENT (3 in.) over crushed rock BASE COURSE (40 in.)

FILL: Loose, gray SAND; fine grained, trace silt to clean

mottled rust at 14 ft

medium dense at 15 ft

(7/21/99)

Ⅰ 2-IN.-OD SPLIT-SPOON SAMPLER

Ⅱ 3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

▮ NX CORE RUN

|—— SLOTTED PVC PIPE

▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

▩ UNDRAINED SHEAR STRENGTH, TSF

* NO RECOVERY

Plastic Limit — Moisture Content — Liquid Limit

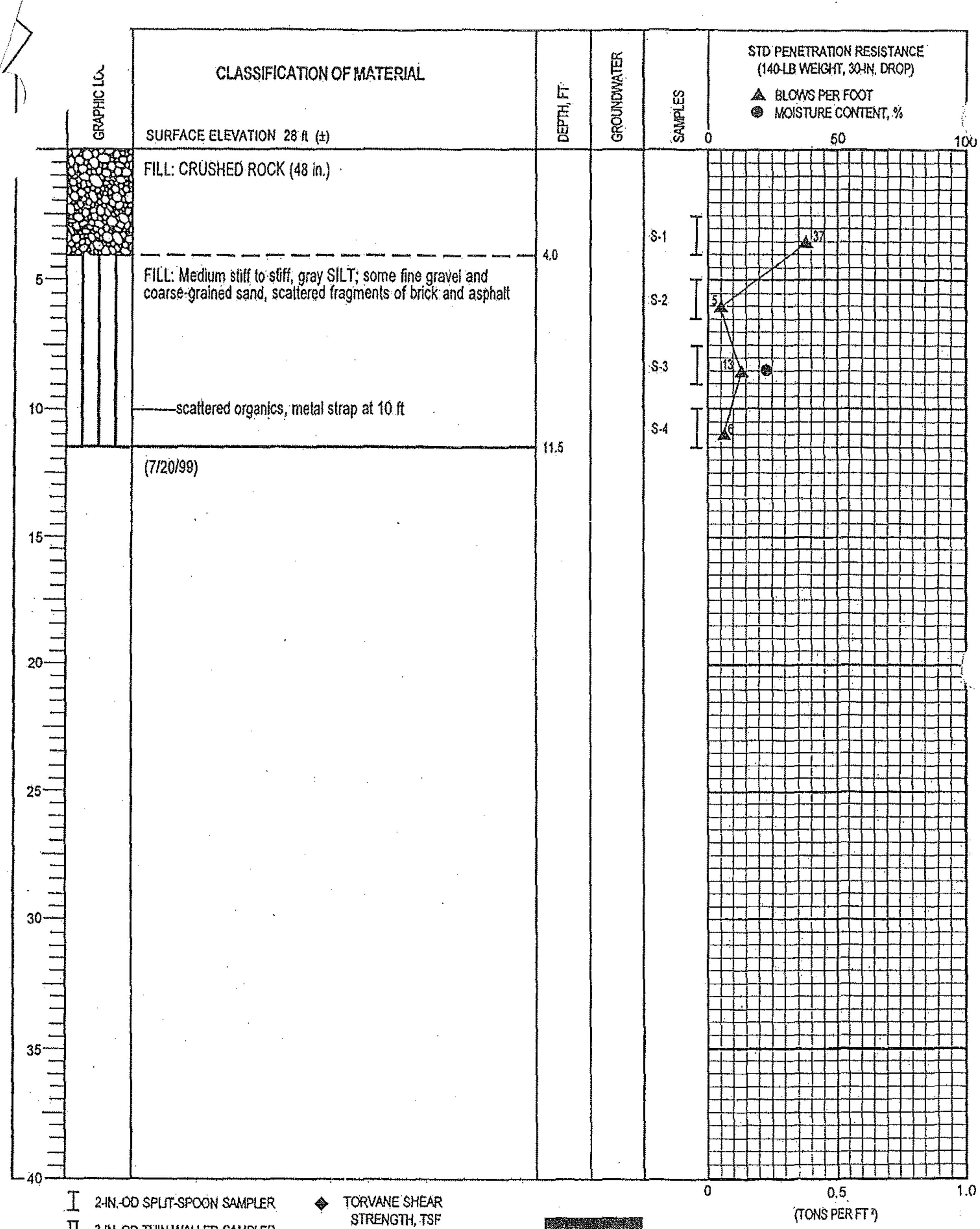
GRI

BORING B-8

OCT. 1999

JOB. NO. 3061

FIG. 8A



SURFACE ELEVATION 28 ft (±)

FILL: CRUSHED ROCK (48 in.)

FILL: Medium stiff to stiff, gray SILT; some fine gravel and coarse-grained sand, scattered fragments of brick and asphalt

——scattered organics, metal strap at 10 ft

(7/20/99)

I 2-IN.-OD SPLIT-SPOON SAMPLER

II 3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

▮ NX CORE RUN

¦—SLOTTED PVC PIPE

▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

■ UNDRAINED SHEAR STRENGTH, TSF

* NO RECOVERY

Liquid Limit

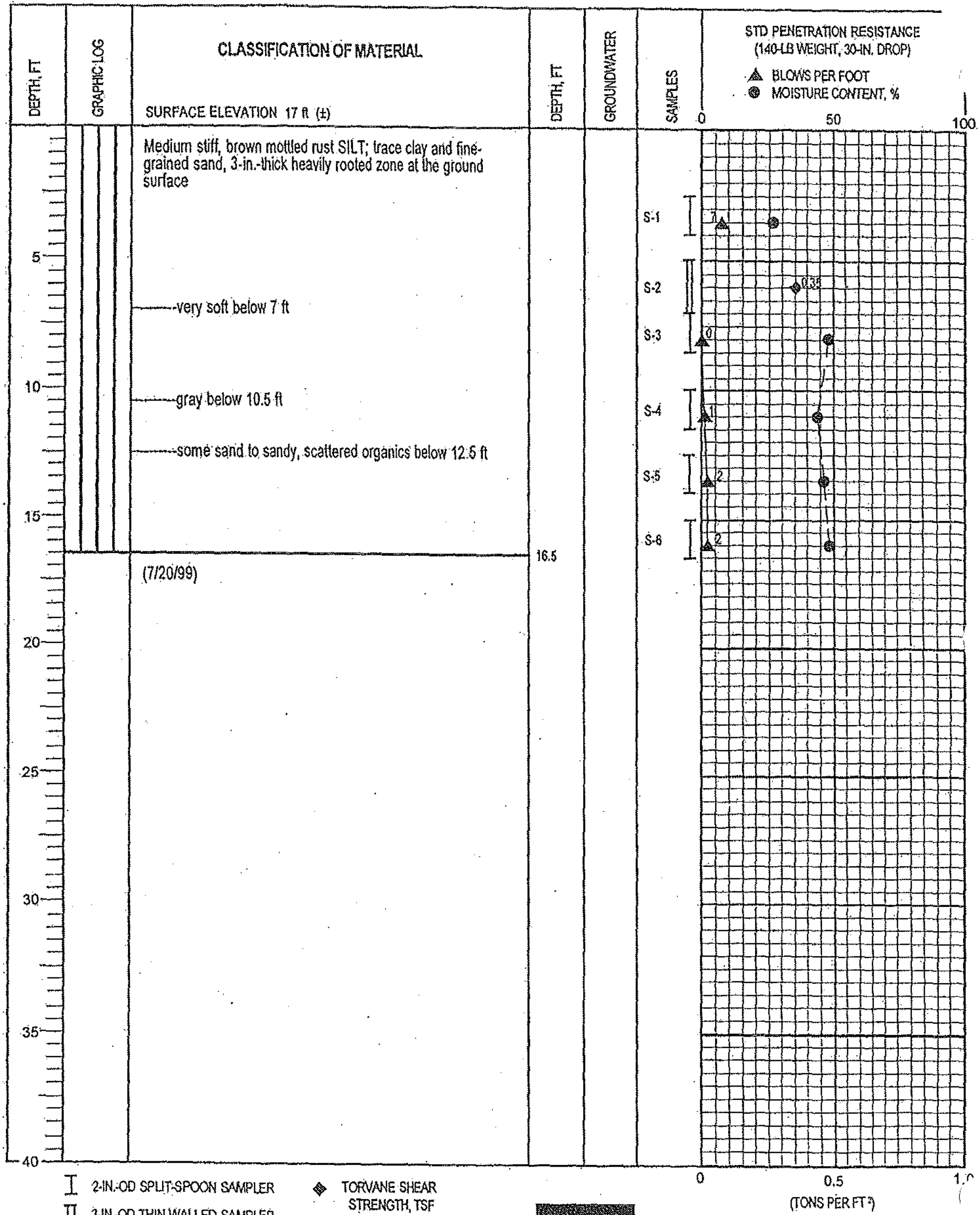
Moisture Content

Plastic Limit

GRI

BORING B-9

OCT. 1999 JOB NO. 3061 FIG. 9A



Medium stiff, brown mottled rust SILT; trace clay and fine-grained sand, 3-in.-thick heavily rooted zone at the ground surface

--very soft below 7 ft

--gray below 10.5 ft

--some sand to sandy, scattered organics below 12.5 ft

(7/20/99)

I 2-IN.-OD SPLIT-SPOON SAMPLER

II 3-IN.-OD THIN-WALLED SAMPLER

G GRAB SAMPLE OF DRILL CUTTINGS

▮ NX CORE RUN

|— SLOTTED PVC PIPE

▼ Water Level (date)

◆ TORVANE SHEAR STRENGTH, TSF

▩ UNDRAINED SHEAR STRENGTH, TSF

* NO RECOVERY

Plastic Limit — Moisture Content — Liquid Limit

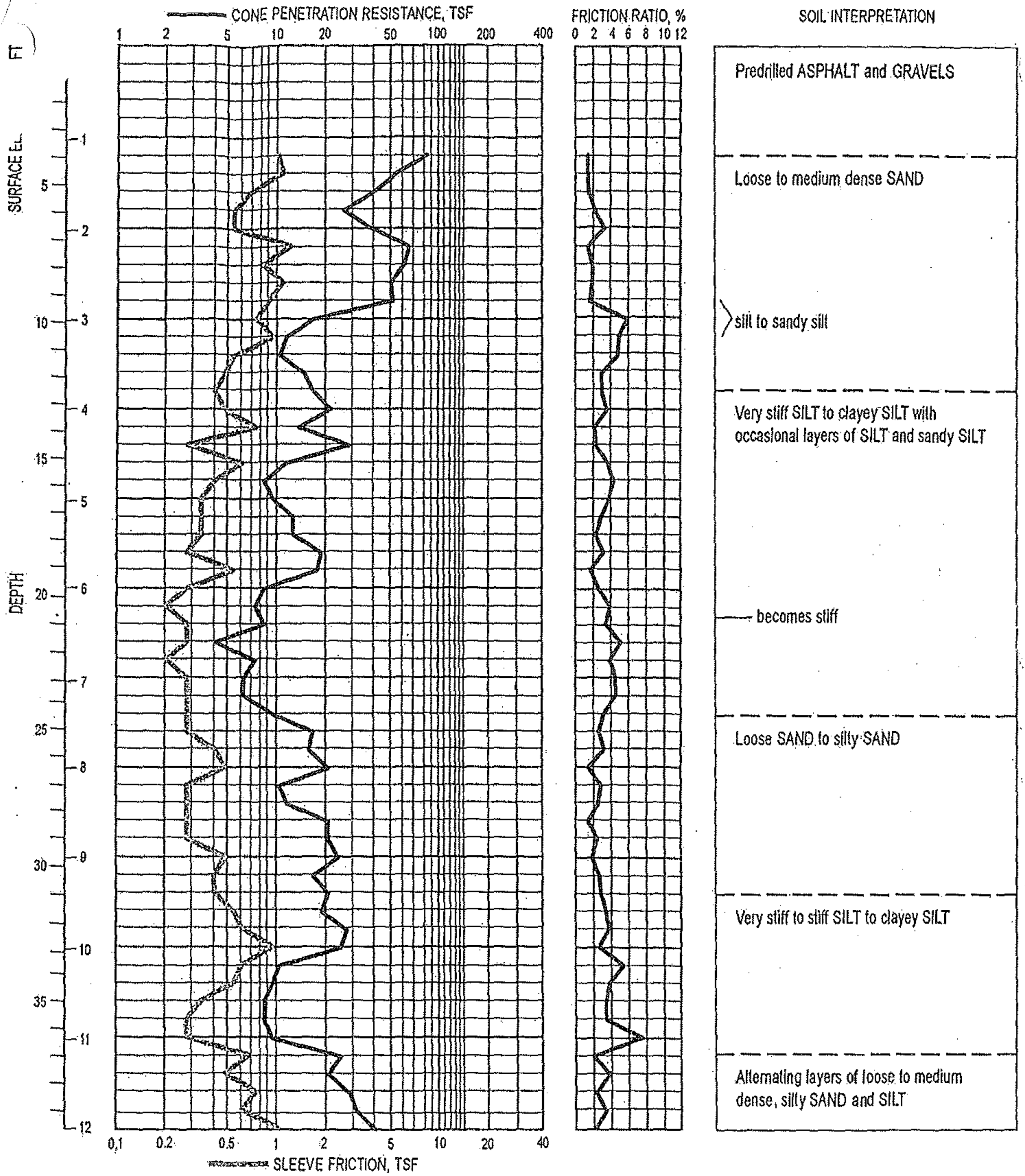
GRI

BORING B-10

OCT. 1999

JOB. NO. 3061

FIG. 10A



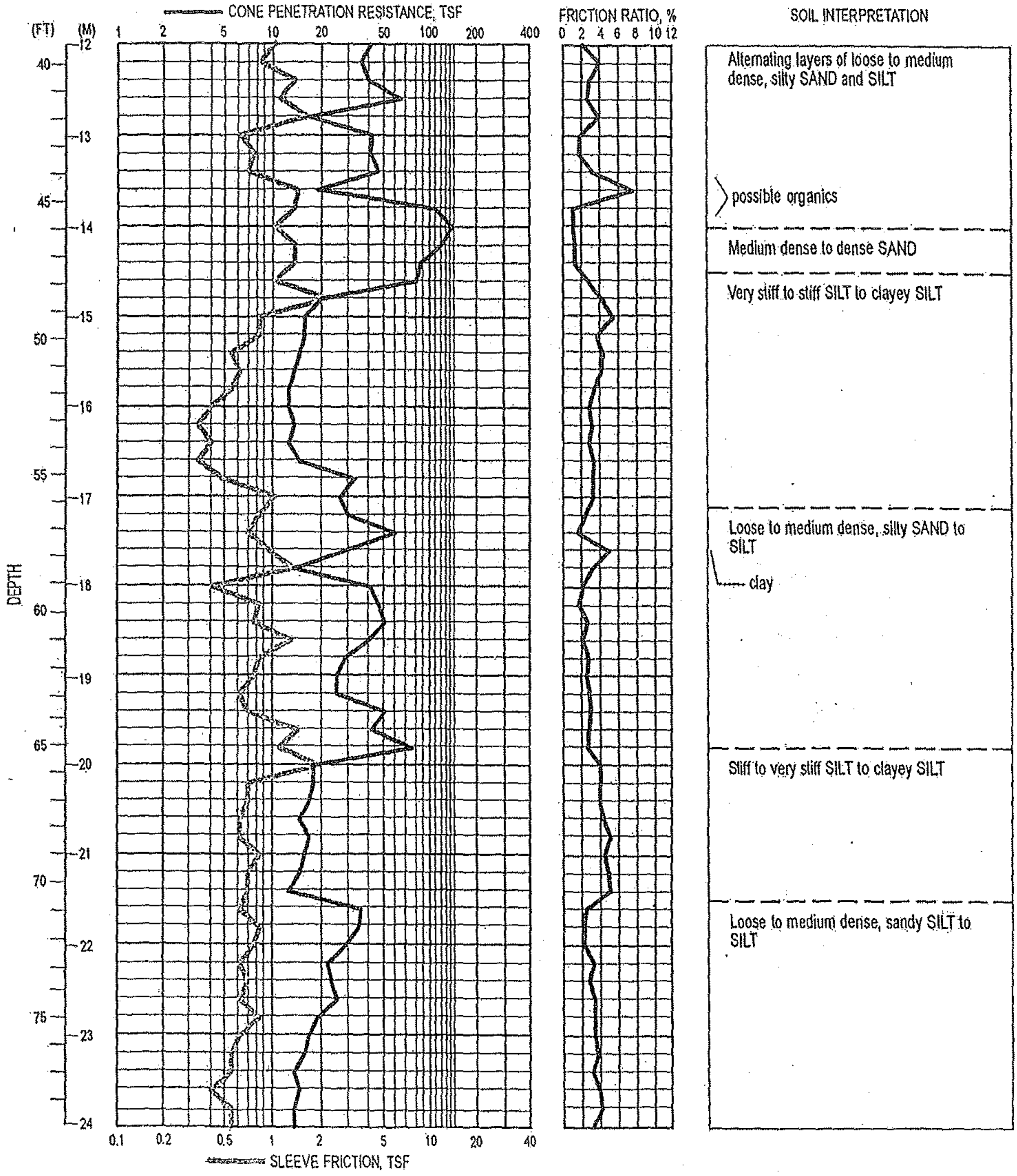
FRICTION RATIO IS EQUAL TO SLEEVE FRICTION DIVIDED BY CONE PENETRATION, EXPRESSED AS A PERCENT

TEST PERFORMED AND INTERPRETED BY VANDEHEY SOIL EXPLORATIONS, BANKS, OREGON

GRI

PROBE P-1

OCT. 1999 JOB NO. 3061 FIG. 11A



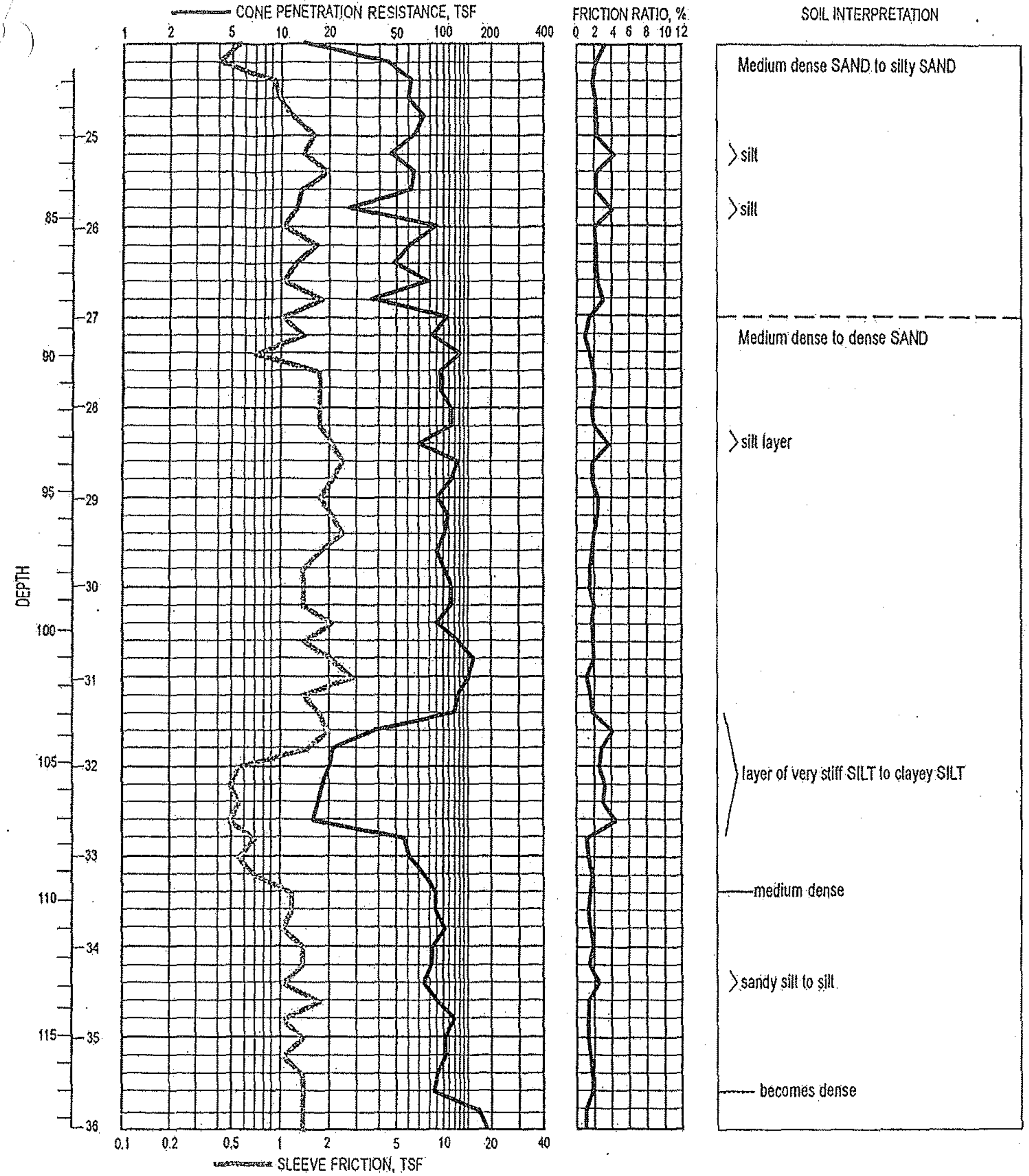
FRICTION RATIO IS EQUAL TO SLEEVE FRICTION DIVIDED BY CONE PENETRATION, EXPRESSED AS A PERCENT

TEST PERFORMED AND INTERPRETED BY VANDEHEY SOIL EXPLORATIONS, BANKS, OREGON

GRI

PROBE P-1 (cont.)

OCT. 1999 JOB NO. 3061 FIG. 11A

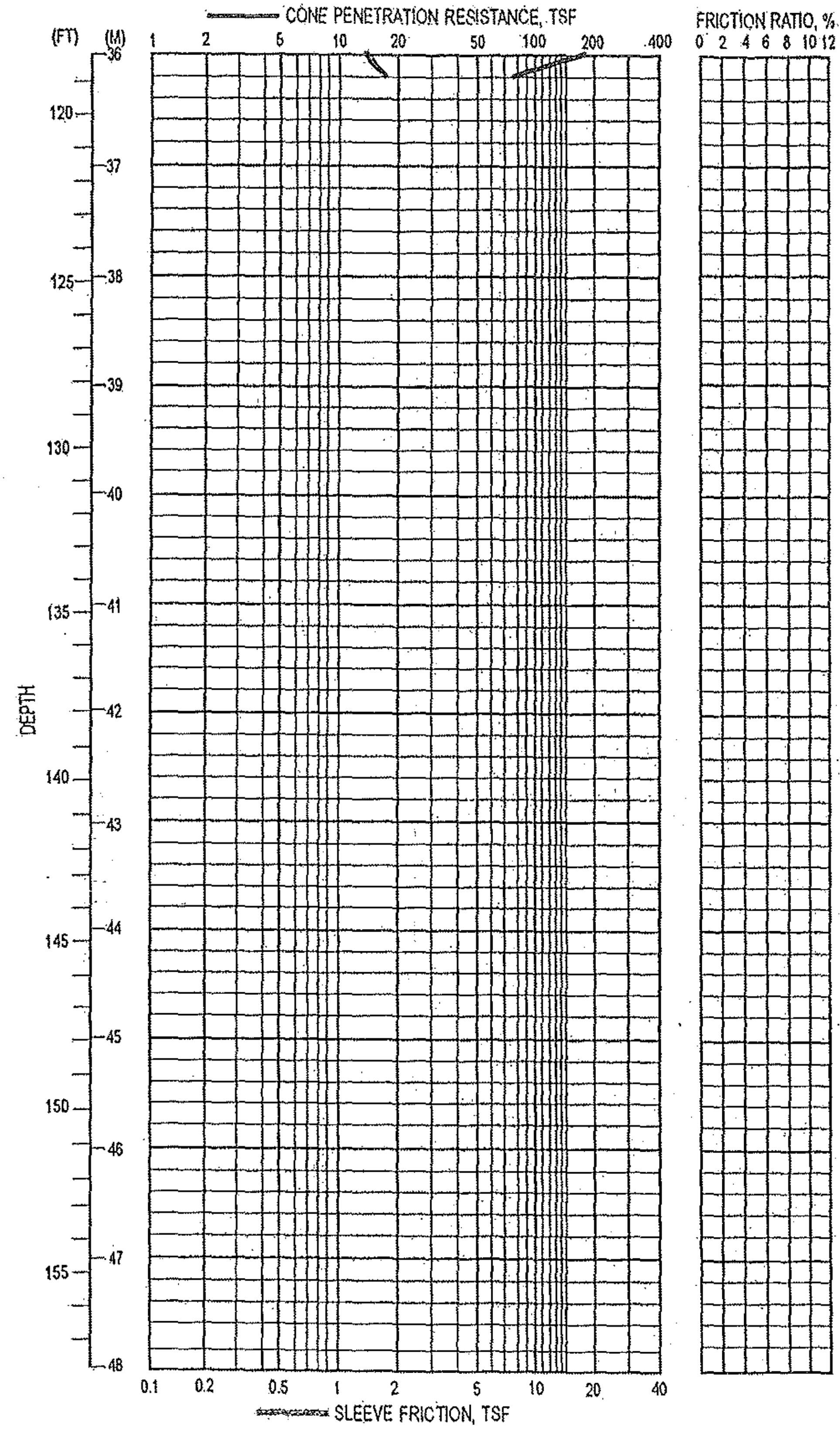


FRICTION RATIO IS EQUAL TO SLEEVE FRICTION DIVIDED BY CONE PENETRATION, EXPRESSED AS A PERCENT

TEST PERFORMED AND INTERPRETED BY VANDEHEY SOIL EXPLORATIONS, BANKS, OREGON

PROBE P-1 (cont.)

OCT. 1999 JOB NO. 3061 FIG 11A



SOIL INTERPRETATION

Dense SAND

Bottom of probe 119 ft (7/20/99)

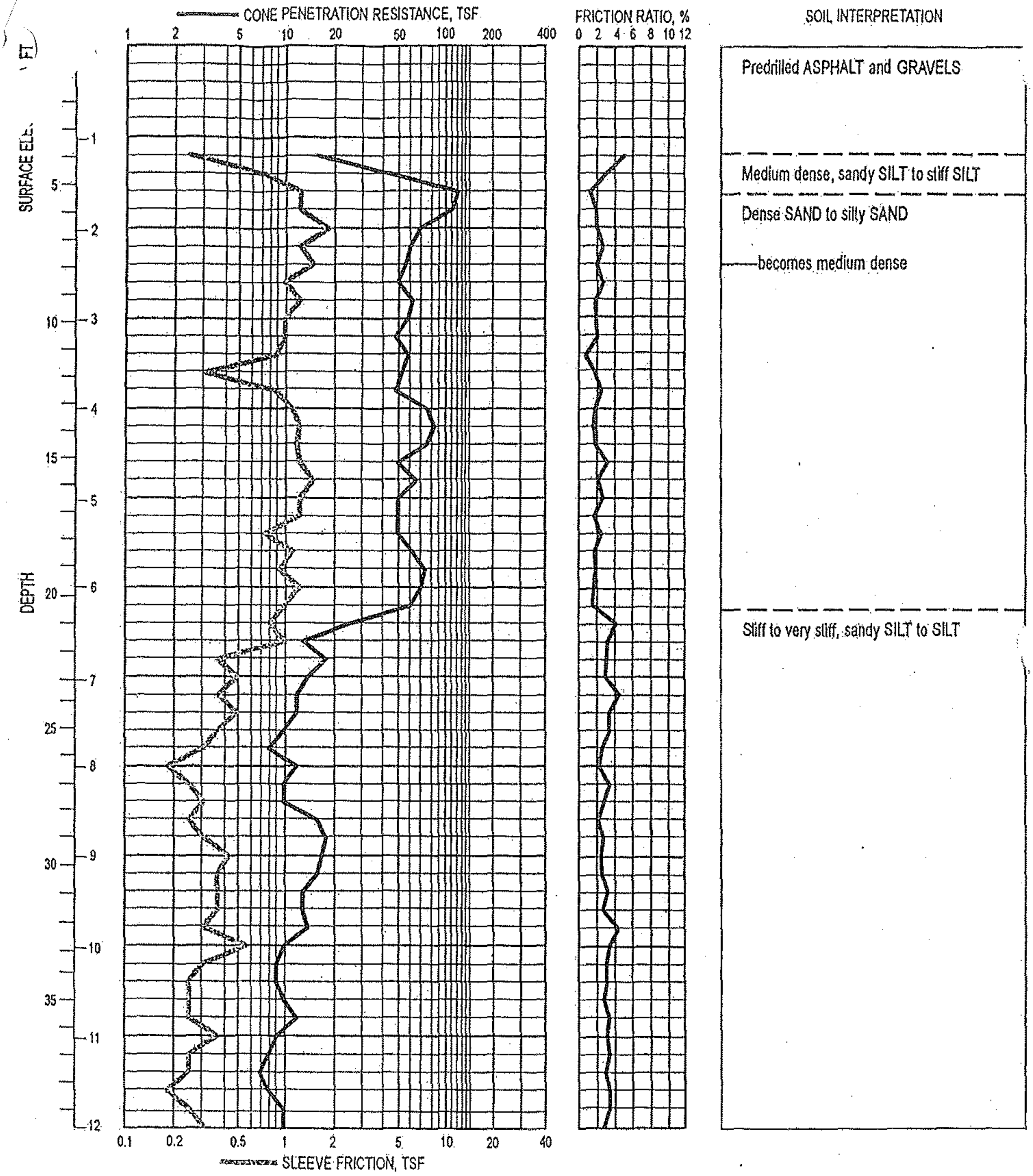
FRICTION RATIO IS EQUAL TO SLEEVE FRICTION DIVIDED BY CONE PENETRATION, EXPRESSED AS A PERCENT

TEST PERFORMED AND INTERPRETED BY VANDEHEY SOIL EXPLORATIONS, BANKS, OREGON

GRI

PROBE P-1 (cont.)

OCT. 1999 JOB NO. 3061 FIG 11A



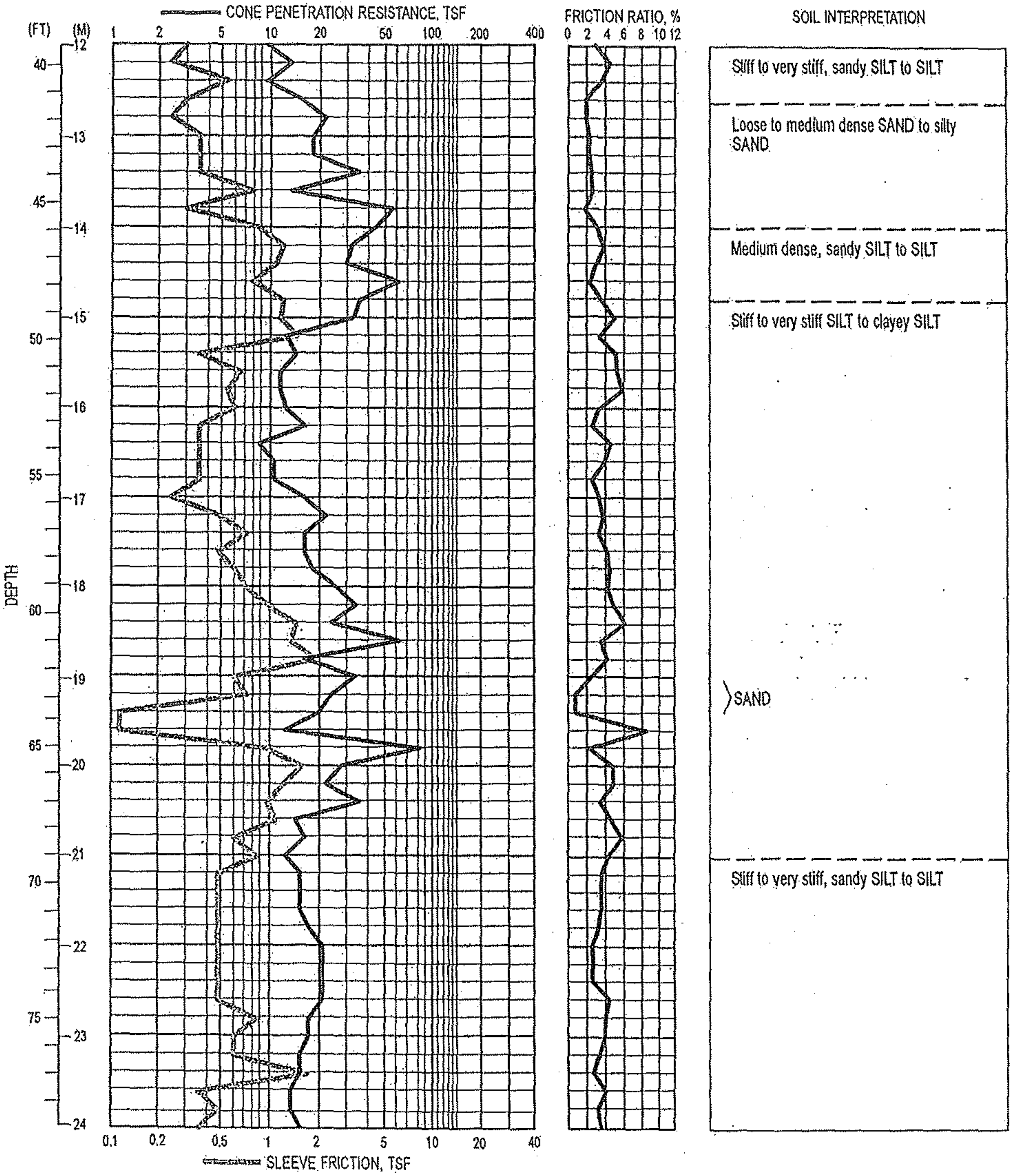
FRICTION RATIO IS EQUAL TO SLEEVE FRICTION DIVIDED BY CONE PENETRATION, EXPRESSED AS A PERCENT

TEST PERFORMED AND INTERPRETED BY VANDEHEY SOIL EXPLORATIONS, BANKS, OREGON

GRI

PROBE P-2

OCT. 1999 JOB NO. 3061 FIG 12A



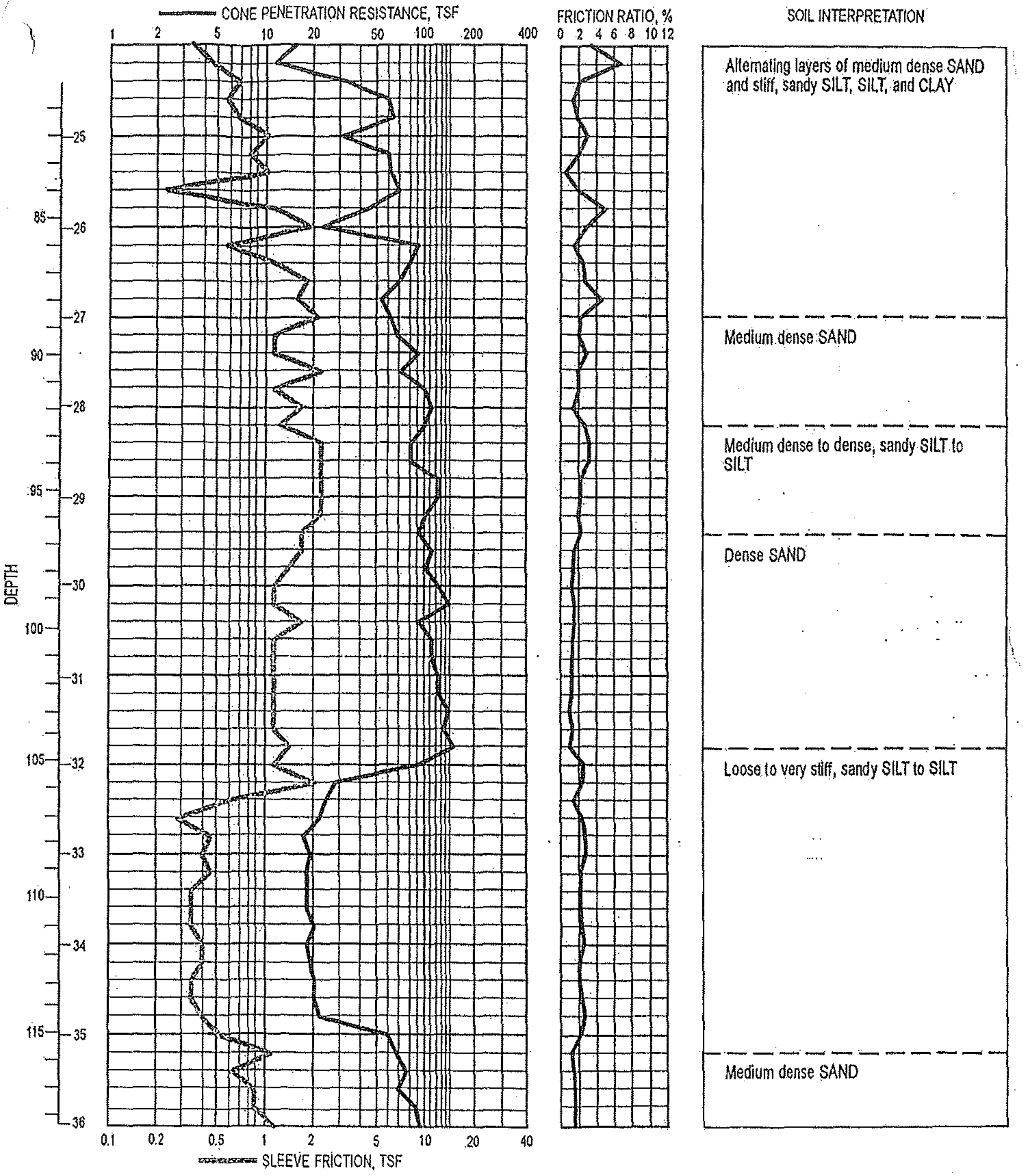
FRICTION RATIO IS EQUAL TO SLEEVE FRICTION DIVIDED BY CONE PENETRATION, EXPRESSED AS A PERCENT

TEST PERFORMED AND INTERPRETED BY VANDEHEY SOIL EXPLORATIONS, BANKS, OREGON.

GRI

PROBE P-2 (cont.)

OCT. 1999 JOB NO. 3061 FIG. 12A



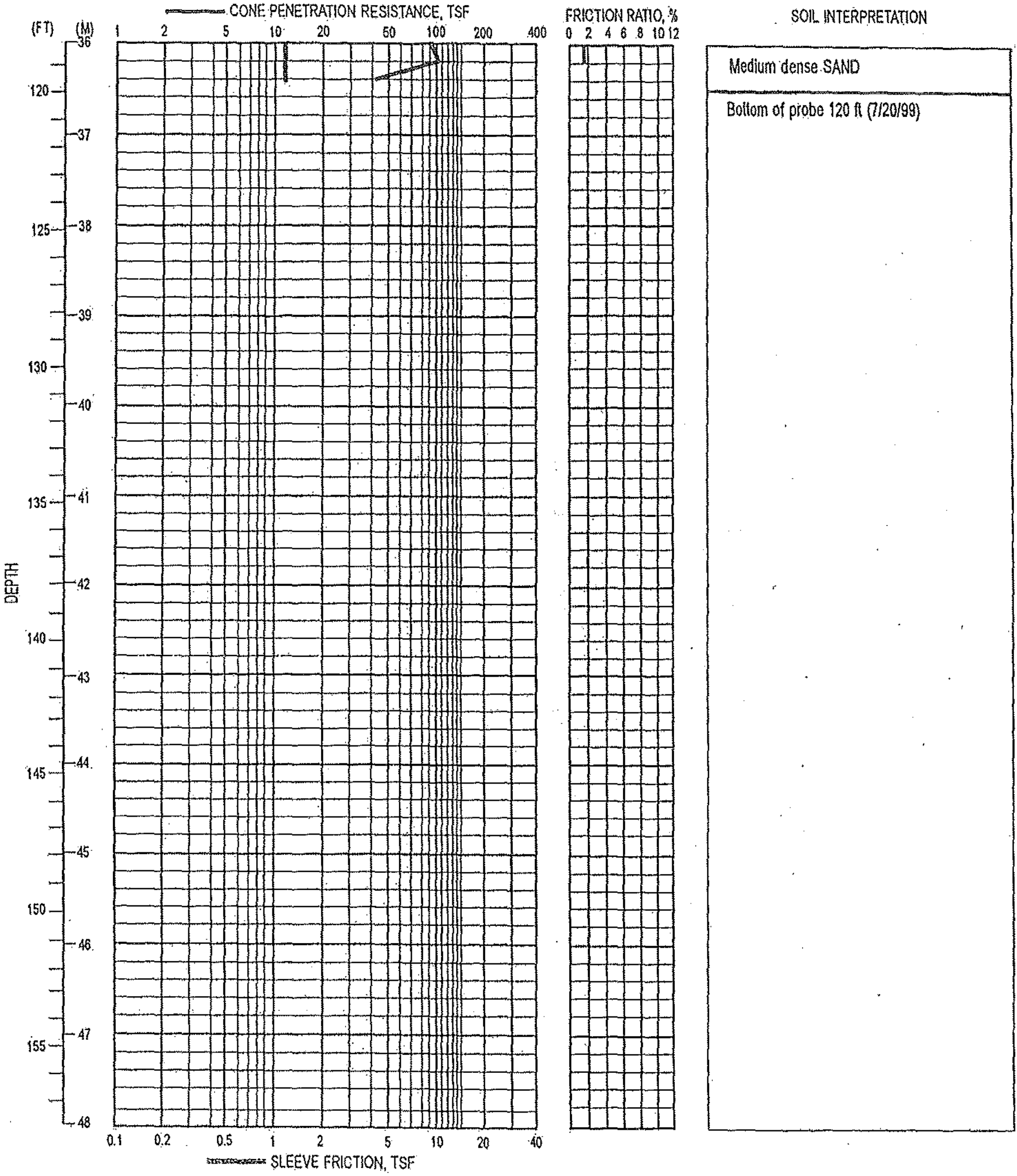
FRICTION RATIO IS EQUAL TO SLEEVE FRICTION DIVIDED BY CONE PENETRATION, EXPRESSED AS A PERCENT

TEST PERFORMED AND INTERPRETED BY VANDEHEY SOIL EXPLORATIONS, BANKS, OREGON

GRI

PROBE P-2 (cont.)

OCT. 1999 JOB NO. 3061 FIG 12A



Medium dense SAND

Bottom of probe 120 ft (7/20/99)

FRICTION RATIO IS EQUAL TO SLEEVE FRICTION DIVIDED BY CONE PENETRATION, EXPRESSED AS A PERCENT

TEST PERFORMED AND INTERPRETED BY VANDEHEY SOIL EXPLORATIONS, BANKS, OREGON

GRI

PROBE P-2 (cont.)

OCT. 1999 JOB NO. 3061 FIG 12A