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September 28, 2000

GT_003310

3324 GEOTECHNICAL REPORT

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2138/4

~~7707~~ 7777 NE Airport Way

Flightcraft, Inc.
7505 NE Airport Way
Portland, OR 97218

Attention: Mitch Berck, Director of Technical Services

SUBJECT: Geotechnical Investigation
Flightcraft Maintenance Hangar
NE Airport Way
Portland, Oregon

At the request of Aron Faegre & Associates, GRI has performed a geotechnical investigation for the proposed Flightcraft maintenance hangar at Portland International Airport. The proposed hangar will be located between Flightcraft's existing Hangar K and PacifiCorp's hangar. 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 foundation support. Our investigation consisted of a review of available geotechnical information for the area, 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 proposed structure.

GRI completed a geotechnical investigation for Flightcraft's business aviation terminal and maintenance hangar located southwest of the terminal. This work is summarized in our August 9, 1990, report to Flightcraft, Inc., entitled, "Geotechnical Investigation, Flightcraft Maintenance Hangar and Business Aviation Terminal, Portland International Airport (PIA), Portland, Oregon."

PROJECT DESCRIPTION

The location and configuration of the proposed 32,000 ft^2 maintenance hangar is shown on the Site Plan, Figure 2. We understand the building will have metal siding and a slab-on-grade floor. The building will not have a basement or significant below-grade areas. You have indicated that maximum column loads will be on the order of 200 kips (compression) and 30 kips (uplift), and the maximum lateral loads will be about 50 kips. Floor live loads are expected to be less than 50 psf. The project architect has indicated that the anticipated maximum fill height will be less than 2 to 3 ft, and cuts will be minimal to site the building. We also understand that the existing asphaltic-concrete pavement will be demolished within the limits of the new building and replaced with a new concrete floor.

SITE DESCRIPTION

Topography

Topographic information provided by Olson Engineering indicates the site is essentially flat at about elevation 16 to 18 ft (NGVD 1947). The site is currently an asphaltic concrete-paved parking lot.

Geology

The site is mantled with dredged Columbia River sand, which is underlain by naturally occurring alluvial silts and sands. At a depth of about 80 ft, the alluvium becomes a fairly continuous deposit of sand that extends to the underlying gravel. Locally, drainage sloughs were excavated in the floodplain and the silty spoils sidecast in mounds along the sides of the slough. At these locations, the sand cap may be thin.

SUBSURFACE CONDITIONS

General

Subsurface conditions at the site were explored on September 11, 2000, with two borings, designated B-1 and B-2, and two cone penetration test (CPT) probes, designated P-1 and P2. The locations of the explorations are shown on Figure 2. The borings were drilled to a depth of 41.5 ft, and the probes were advanced to depths of 40.5 to 71 ft. Our understanding of local subsurface conditions was supplemented by reviewing logs of borings and probes made by GRI at adjacent sites. A discussion of the field exploration program together with logs of the borings and probes is provided in Appendix A. The laboratory testing program conducted to evaluate pertinent physical and engineering properties of the soils encountered in the borings is also described in Appendix A.

Soils

For the purpose of discussion, the soils disclosed by the subsurface investigation have been grouped into the following categories:

1. **PAVEMENT**
2. **Dredged SAND FILL**
3. **Alluvial SILT with layers of SAND**
4. **SAND**

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

1. PAVEMENT. The site is paved with asphaltic concrete (AC). A 2- to 3-in.-thick section of AC pavement was encountered at the ground surface at the location of the borings and probes. The AC is generally underlain by a 6- to 12-in.-thick base course that consists of gravel.

2. Dredged SAND FILL. All of the borings and probes encountered clean, gray, dredged sand fill beneath the existing pavement section. The fill extends to depths of about 7.5 to 10 ft. We understand that most of the sand was placed during the mid- to late-1950s.

N-values ranging from about 12 to 33 blows/ft and cone penetration resistances ranging from about 82 to 130 tsf indicate the fill is generally medium dense to dense. The natural moisture content of this material ranges from about 15 to 30%. Based on past experience in this area, we anticipate the sand fill contains occasional thin layers of silt and silty sand.

3. Alluvial SILT with layers of SAND. The sand fill is underlain by alluvial silt with varying percentages of clay and fine-grained sand. The silt contains occasional 2- to 5-ft-thick layers of relatively clean to silty, medium dense sand. The borings and probes were terminated in the silt deposit at depths of 40.5 to 71 ft.

Probe sleeve friction resistances ranging from about 0.10 to 1.0 tsf and Torvane shear strength values ranging from about 0.15 to 0.25 tsf indicate the relative consistency of the silt ranges from soft to medium stiff above a depth of about 60 ft. Below 60 ft, the silt becomes stiff, as indicated by sleeve friction resistances of 0.5 to 1.0 tsf. The natural moisture content of the silt ranges from about 30 to 90%. The dry unit weight of the silt ranges from about 55 to 70 pcf.

The results of a one-dimensional consolidation test performed on a representative sample of silt are shown on Figure 5A. The test indicates the silt has a low compressibility in the overconsolidated range of pressures and moderate to high compressibility in the normal compression range. The silt also displays some preconsolidation above the existing overburden pressure.

4. SAND. Based on our review of boring and probe logs from nearby adjacent sites, we anticipate the silt is underlain by a generally continuous deposit of relatively clean sand. Sand was encountered at a depth of about 81 ft in a probe made by GRI near Flightcraft's existing maintenance hangar. The relative density of the sand is medium dense to dense.

Groundwater

We understand that drainage canals and sloughs across this area are pumped to control the water levels in the floodplain. The pumping is done under the jurisdiction of several drainage districts. These pumping stations maintain the water level in the sloughs and drainages at about elevation +8 ft MSL at the project site. However, seasonal variations can occur which are associated with extreme highs and lows of the Columbia River. We understand the 100-year design flood level at this site is +14 ft MSL. During the wet winter months, groundwater levels at locations away from the sloughs are commonly higher than the water level in the sloughs, and may approach the ground surface.

Groundwater levels in probes P-1 and P-2 were measured at depths of 10.8 ft and 8.5 ft, respectively.

CONCLUSIONS AND RECOMMENDATIONS

General

The subsurface explorations indicate the site is mantled with a 7.5- to 10-ft-thick layer of medium dense sand fill. Below the sand fill is a thick deposit of compressible silt. The results of our investigation indicate the proposed structure can be supported on spread footing foundations established in the sand fill. The following sections of this report provide our conclusions and recommendations concerning site preparation, earthwork, and design and construction of foundations for the proposed structure.

Site Preparation and Grading

The site is currently paved with asphaltic concrete (AC). We recommend the AC be removed from within the limits of the proposed building. The upper 12 in. of floor slab subgrades should be compacted to at least 95% of the maximum dry density as determined by ASTM D 1557. Moderately heavy to heavy, smooth, steel-wheeled vibratory rollers are most effective for compaction of the sand fill. Wetting of the sand will probably be required to achieve the recommended compaction. The existing sand fill should provide an all-weather working surface for construction traffic.

In new pavement areas, the existing pavement can remain in place, provided it is broken into pieces of about 12-in. maximum dimension and is at least 12 in. below subgrade for the new pavements. In our opinion, any removed AC could be crushed to a maximum nominal size of 1½ in. and used only in pavement areas as structural fill, trench backfill, or a substitution for a portion of the granular base course outside the limits of the new building.

Structural Fill

In our opinion, granular materials consisting of material with a maximum size of up to 6 in. and with not more than about 5% fines passing the No. 200 sieve (washed analysis) should be used to construct structural fills. The dredged sand present in the site vicinity is generally suitable for this purpose. The granular fill should be placed in 12-in.-thick lifts (loose). All lifts should be compacted with a medium-weight (48-in.-diameter drum), smooth, steel-wheeled, vibratory roller until well keyed. Generally, a minimum of four passes with the roller is required to achieve compaction. Sand should be compacted to at least 95% of the maximum density as determined by ASTM D 1557. All structural fills should extend a minimum horizontal distance of 10 ft beyond the limits of the building.

Utilities

Depending on the depth of utilities and the groundwater level at the time of construction, groundwater seepage, running soil conditions, and unstable trench sidewalls or soft trench subgrades may be encountered. These conditions, if encountered, will require dewatering of the excavation and use of shoring for sidewall support. The temporary excavation plan is the responsibility of the contractor; however, all excavation sidewalls should be properly sloped or shored to conform to applicable local, state, or federal regulations. Some overexcavation of the trench bottom may also be necessary to permit the installation of stabilization material. Clean, 4-in.-minus, crushed rock is typically used for this purpose. The actual depth of overexcavation will depend on the contractor's method of operation and the conditions encountered, and should be established at the time of construction. Usually about 2 ft of coarse-graded rock beneath the normal pipe zone and bedding is adequate to stabilize a soft trench bottom. Placement of the stabilization rock also serves to facilitate dewatering from sumps established within the trench or adjacent to manholes.

The control of groundwater will depend on the types of materials and groundwater levels encountered in the excavations. In this regard, we anticipate that relatively small groundwater inflows will be encountered in silty soils; larger inflows and possibly running soil conditions will be encountered in sandy soils. In our opinion, dewatering silty soils can likely be accomplished by pumping from sumps. In those areas where sandy soils and running soil conditions are encountered and cannot be tolerated, it may be necessary to use other methods of groundwater control, such as pumping from well points installed adjacent to the excavation or using tight-joint sheet piling for excavation support. Any proposed dewatering system should be capable of maintaining groundwater levels below the base of the excavation or as required to maintain a stable trench bottom.

All utility trench excavations within building and pavement areas should be backfilled with relatively clean, granular material, such as sand, sandy gravel, or crushed rock of up to 2-in. maximum size and having less than 5% passing the No. 200 sieve (washed analysis). In our opinion, clean Columbia River dredged sand would be suitable for this purpose. Additionally, trench backfill within pavement areas could also consist of AC grindings, prepared as described in the Site Preparation section. The granular backfill material should be compacted to at least 92% of the maximum dry density as determined by ASTM D 1557. The use of hoe-mounted vibratory plate compactors is usually most efficient for this purpose. Lift thicknesses should be evaluated on the basis of field density tests; however, particular care should be taken when operating hoe-mounted compactors to prevent damage to the newly placed conduits. Flooding or jetting to compact the trench backfill should not be permitted. Native materials can be used above the pipe zone for trench backfill in unimproved areas where a soft trench and future settlement of the backfill can be tolerated.

Settlement of the completed utilities can be the result of improperly dewatering the trench excavation during construction, improperly stabilizing the trench bottom or bedding the conduit, or due to consolidation of the subsoils as a result of the placement of thick sections of fill above the completed installation.

Foundation Design

In our opinion, spread footings may be used to support the structural loads of the proposed buildings. All footings should be founded in the medium dense sand fill that mantles the site or structural fill placed to site the building. We recommend the footings be designed using a maximum allowable soil bearing pressure of 2,000 psf. This allowable bearing pressure applies to the total of dead load 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 depth of embedment of all footings should be 1½ ft below the lowest adjacent finished grade. The minimum width of any footing founded in the sand or structural fill should be 2 ft. All footings should be excavated with a smooth-edge bucket. For footings founded on sand, it is likely that the footing subgrade will be disturbed during excavation. Therefore, we recommend that the bottoms of all footing excavations founded in sand be wetted and compacted by several passes with a heavy, hand-operated vibratory plate compactor just prior to placing the reinforcing steel for the footing. The bottom of the footings should be wetted prior to placing the concrete.

We anticipate the maximum total settlement of the wall and column footings founded in the sand should be less than 1 in. In our opinion, differential settlement between any two adjacent column footings should be less than half of the total settlement of similarly loaded columns. The estimated settlement is based on the assumption that footings will be placed about 1½ ft below the finished site grade, will have a minimum width of 2 ft, and will have at least one footing width between the edges of adjacent footings. It is also anticipated that floor live loads will be less than 50 psf.

Horizontal forces due to wind or seismic loads can be resisted partially or completely by frictional forces developed between the base of spread footings and the underlying soils. The total shearing resistance between the footing and the soil should be taken as the normal force, i.e., the sum of all vertical forces (dead load plus real live load), times the coefficient of friction between the sand and the base of the footing. We recommend a coefficient of friction of 0.40 for mass concrete placed directly onto sand. If additional lateral resistance is required, passive soil resistance from embedded footings may be evaluated on the basis of an equivalent fluid having a unit weight of 200 pcf. This design passive soil resistance would only be effective if the backfill for the footing is placed and compacted as recommended in this report for granular structural fill.

Floor Slabs

We anticipate that the finish floor elevation will be established near existing site grades. Therefore, we recommend placing a minimum of 6-in.-thick granular base course beneath all concrete slabs. Base course material can consist of 3/4-in.-minus crushed rock compacted to at least 95% of the maximum density as determined by ASTM D 1557. In addition, it may be appropriate to install a suitable vapor-retarding membrane beneath slab-on-grade floors in the office building areas where damp-proofing may be needed. The details of the vapor-retarding membrane are shown on Figure 3, which shows a minimum 6-in.-thick granular base course beneath the concrete floor slab. Assuming the floor slab subgrade and base course are suitably prepared, we recommend using a modulus of subgrade reaction of 225 pci for the design of concrete slabs that are subjected to heavy floor loads.

Seismic Considerations

The project site is assigned to seismic zone 3 in the Uniform Building Code (UBC). Based on the results of our subsurface investigation and review of the UBC, we recommend that the structure be evaluated using soil profile type S_E.

We have estimated the magnitude of seismically induced settlement (liquefaction) at the site will be in the range of 2 to 3 in., assuming groundwater at a depth of about 10 ft and earthquake magnitudes, focal distances, and accelerations consistent with a UBC zone 3 seismic event. It should be assumed that about half of this settlement could occur as differential settlement between adjacent columns. In our opinion, earthquake-induced differential settlement will be minimized due to the presence of the medium dense to dense cap of dredged sand fill at this site and the confinement provided by the alluvial silt. We also anticipate that a significant percentage of the settlement will occur after the shaking stops.

In our opinion, the potential for earthquake-induced fault displacement, landslides, lateral ground movement, and damage by tsunamis and/or seiches at this site is low.

Design Review and Construction Services

We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GRI should be retained to review all geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations provided in our report. Additionally, to observe compliance with the intent of our recommendations, design concepts, and the plans and specifications, we are of the opinion that all construction operations dealing with earthwork and foundations should be observed by a GRI representative. Our construction-phase services will allow for timely design changes if site conditions are encountered that are different from those described in this report. If we do not have the opportunity to confirm our interpretations, assumptions, and analyses during construction, we cannot be responsible for the application of our recommendations to subsurface conditions that are different from those described in this report.

LIMITATIONS

This report has been prepared to aid the architect and/or engineer 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 maintenance hangar. In the event that any changes in the design and location of the building 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 made 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 the boring and probe 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.

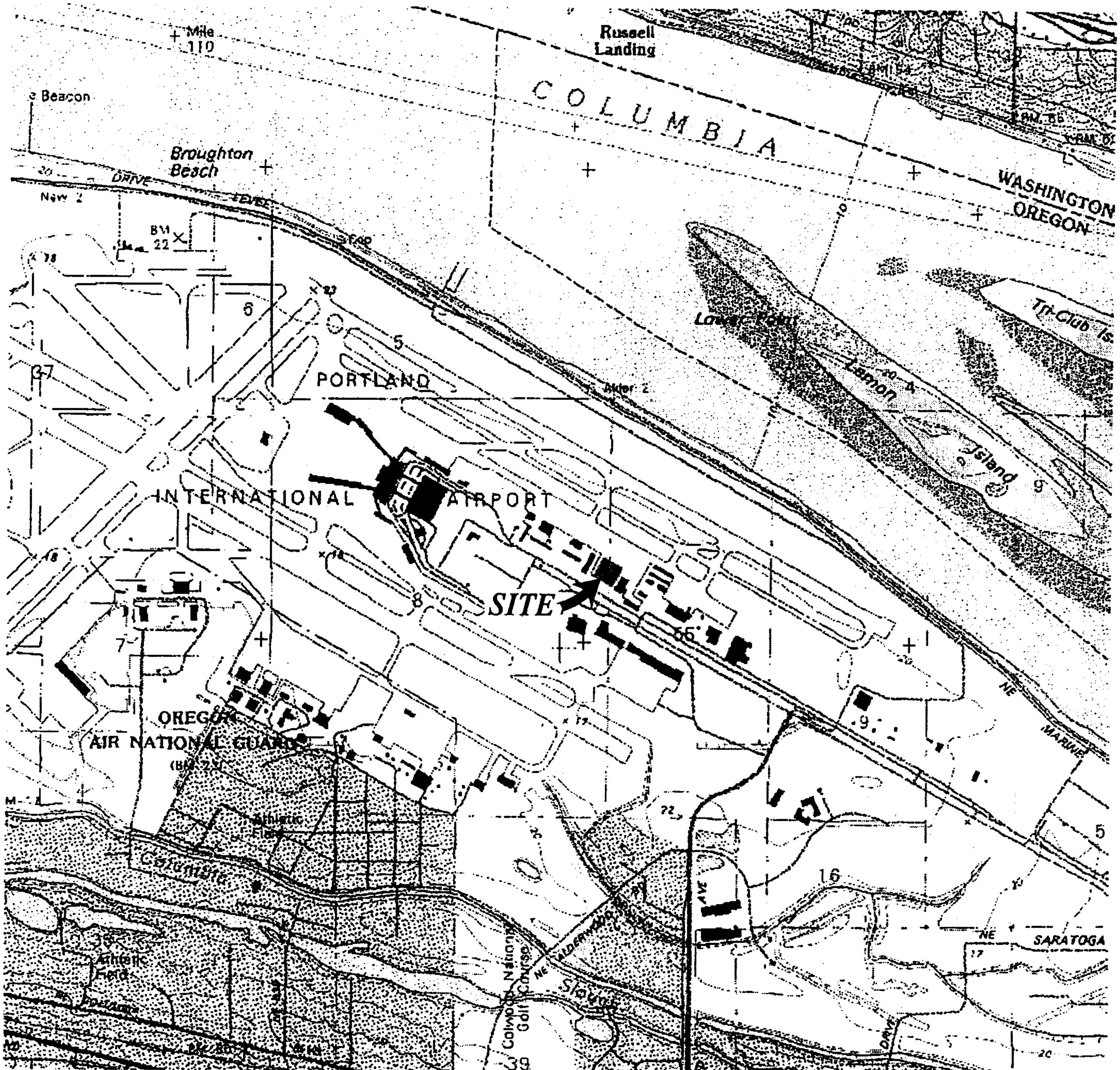
Submitted for GRI,

REGISTERED PROFESSIONAL ENGINEER
8045
OREGON
JULY 22, 1974
H. STANLEY KELSAY
6/01

REGISTERED PROFESSIONAL ENGINEER
17299
OREGON
JULY 19, 1994
GENE M. TUPPER
Renew Date 6/02

H. Stanley Kelsay, PE
Principal

Gene M. Tupper, PE
Project Engineer



DELORME 3-D TOPOQUADS, OREGON WEST
MOUNT TABOR, OREG. (2db) 1999

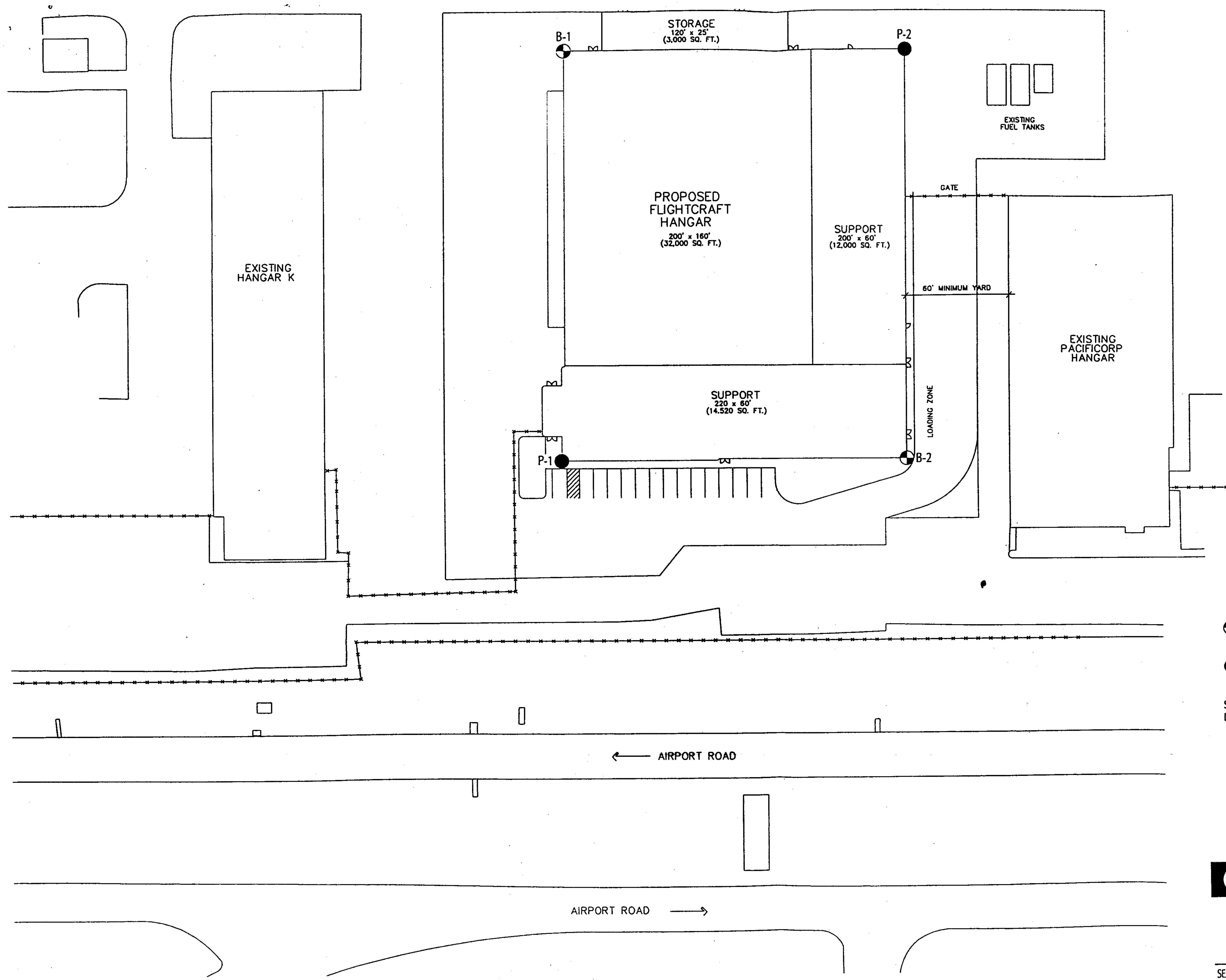
North

0 1/2 1 MILE

GRI FLIGHTCRAFT, INC.
PDX MAINTENANCE HANGAR

VICINITY MAP

SEPT. 2000 JOB NO. 3324 FIG. 1



BORING MADE BY GRI (SEPTEMBER 11, 2000)

CONE PENETRATION TEST MADE BY GRI (SEPTEMBER 11, 2000)

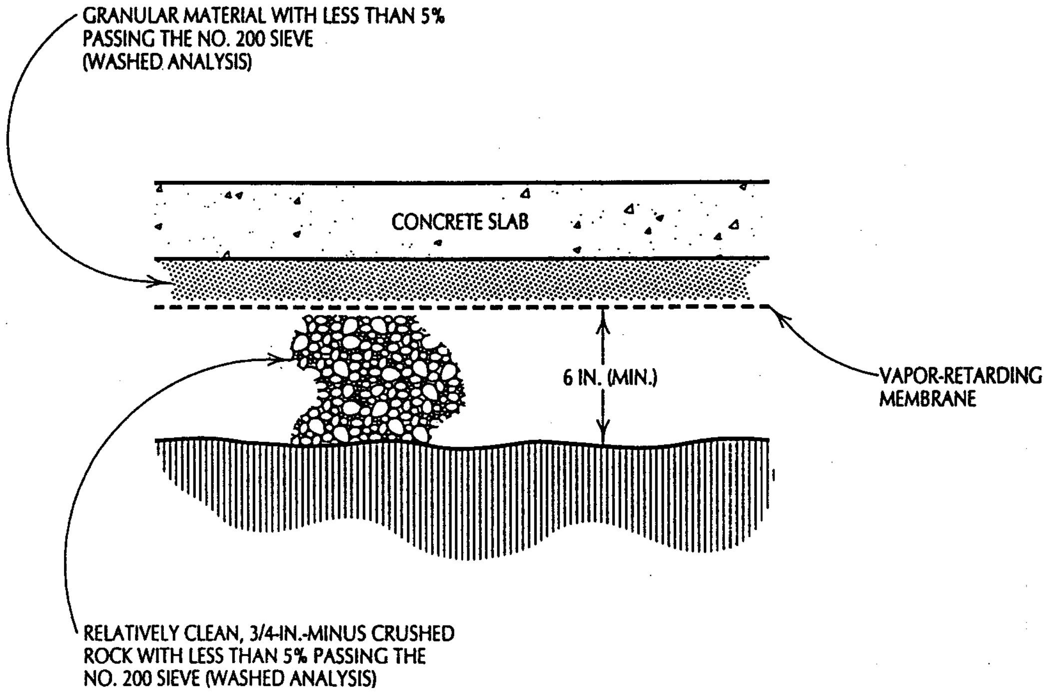
SITE PLAN FROM CADFILE BY ARON FAEGRE AND ASSOCIATES, INC., DATED AUGUST 7, 2000

GRI

FLIGHTCRAFT, INC.
PDX MAINTENANCE HANGAR

SITE PLAN

SEPT. 2000 | JOB NO. 3324 | FIG. 2



A VAPOR-RETARDING MEMBRANE SYSTEM IS RECOMMENDED FOR MOISTURE-SENSITIVE AREAS. DETAILS REGARDING INSTALLATION OF THE SYSTEM SHOULD BE REVIEWED BY THE DESIGN TEAM.

GRI

FLIGHTCRAFT, INC.
PDX MAINTENANCE HANGAR

UNDERSLAB DRAINAGE DETAIL

SEPT. 2000 | JOB NO. 3324 | FIG. 3

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

General

The subsurface materials and conditions at the site were investigated on September 11, 2000, with two borings, designated B-1 and B-2, and two cone penetration test probes, designated P-1 and P-2. The borings were drilled to a depth of about 40 ft, and the probes were extended to depths of about 40.5 to 71 ft. The approximate locations of the explorations are shown on the Site Plan, Figure 2.

Borings

The borings were drilled using mud-rotary techniques with a truck-mounted CME-55 drill rig provided and operated by Geo-Tech Explorations 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 was conducted. This test consists of driving a standard split-spoon sampler into the soil a distance of 18 in. 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. The N-values provide a measure of the relative density of granular soils, such as sand, and the relative consistency, or stiffness, of cohesive soils, such as silt. The soil samples obtained in the split-spoon sampler were carefully examined in the field and representative portions were saved in airtight jars for further examination and physical testing in our laboratory. In addition, relatively undisturbed Shelby tube samples were collected and returned to our laboratory.

Logs of the borings are provided on Figures 1A and 2A. Each log presents a descriptive summary of the various types of materials encountered in the boring and notes the depth where the materials and/or characteristics of the materials change. To the right of the descriptive summary, the numbers and types of samples taken during the drilling operation are indicated. Farther to the right, N-values are shown graphically, along with the natural moisture contents and Torvane shear strength values. The terms used to describe the soils are defined in Table 1A.

Cone Penetration Test Probes

The cone penetration tests were performed and interpreted by G2 CPT, LLC of Forest Grove, 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 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.

The logs of CPT probes P-1 and P-2 are provided on Figures 3A and 4A. The logs show the values of cone penetration resistance and sleeve friction. To the right, the friction ratio is given (i.e., sleeve friction divided by the cone penetration resistance), as well as an interpretation of the data with respect to the basic type of soil penetrated. Qualitative descriptions of relative consistency based on cone penetration resistance and sleeve friction are also provided on the logs. The terms used to describe the soils are defined in Table 2A.

LABORATORY TESTING

General

All samples obtained from the borings were returned to our laboratory where the physical characteristics of the samples were noted, and the field classifications were modified where necessary. The laboratory testing program included determinations of natural moisture content, Torvane shear strength, and consolidation characteristics. The following paragraphs describe the testing program in more detail.

Natural Moisture Content

Natural moisture content determinations were made in conformance with ASTM D 2216. The results are shown on the Boring Logs, Figures 1A and 2A.

Torvane Shear Strength

The approximate undrained shear strength of relatively undisturbed soil samples was determined using a Torvane shear device. The Torvane is a hand-held apparatus with vanes that 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 the Torvane shear tests are shown on the Boring Logs, Figures 1A and 2A.

Dry Unit Weight

The dry unit weight of undisturbed samples was determined in the laboratory in accordance with ASTM D 2937 by cutting a cylindrical specimen of soil from a Shelby tube sample. The dimensions of the specimen were carefully measured, the volume calculated, and the specimen weighed. After oven drying, the specimen was reweighed and the water content calculated. The dry unit weight was then computed. The dry unit weights are summarized below.

SUMMARY OF
UNIT WEIGHT DETERMINATIONS

Boring	Sample	Depth, ft	Natural Moisture Content, %	Dry Unit Weight, pcf	Soil Type
B-1	S-6	14.6	70	58	SILT
	S-8	21.0	51	70	SILT
B-2	S-3	8.8	23	91	Dredged SAND FILL
	S-5	13.5	73	56	SILT
	S-8	25.8	55	68	SILT

One-Dimensional Consolidation Test

One consolidation test was performed in accordance with ASTM D 2435 to obtain data on the compressibility characteristics of a relatively undisturbed soil sample extruded from the Shelby tubes. The results of the test are shown on Figure 5A in the form of a curve showing effective stress versus percent strain. The initial and final moisture content and dry unit weight of the sample were determined in conjunction with the test and are summarized at the top of the figure.

Table 1A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

Relative Density	Standard Penetration Resistance (N-values) blows per foot
very loose	0 - 4
loose	4 - 10
medium dense	10 - 30
dense	30 - 50
very dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soils

Consistency	Standard Penetration Resistance (N-values) blows per foot	Torvane Undrained Shear Strength, tsf
very soft	2	less than 0.125
soft	2 - 4	0.125 - 0.25
medium stiff	4 - 8	0.25 - 0.50
stiff	8 - 15	0.50 - 1.0
very stiff	15 - 30	1.0 - 2.0
hard	over 30	over 2.0

Sandy silt materials which exhibit general properties of granular soils are given relative density description.

Grain-Size Classification

Boulders
12 - 36 in.

Cobbles
3 - 12 in.

Gravel
1/4 - 3/4 in. (fine)
3/4 - 3 in. (coarse)

Sand
No. 200 - No. 40 sieve (fine)
No. 40 - No. 10 sieve (medium)
No. 10 - No. 4 sieve (coarse)

Silt/Clay - pass No. 200 sieve

Modifier for Subclassification

Adjective	Percentage of Other Material In Total Sample
clean	0 –2
trace	2 –10
some	10 –30
sandy, silty, clayey, etc.	30 –50

Table 2A

SOIL CLASSIFICATION
BASED ON CONE PENETRATION TEST

Friction Ratio (Percent)	Soil Classification
0 to 2	Clean sand or slightly silty sand
2 to 5	Silty sand, clayey sand, or silt
> 5	Clayey silt, silty clay, or clay

COHESIVE SOILS

Sleeve Friction, tsf	Relative Consistency
<0.12	Very Soft
0.12 to 0.25	Soft
0.25 to 0.50	Medium Stiff
0.50 to 1.00	Stiff
1.00 to 2.00	Very Stiff
>2.00	Hard

COHESIONLESS SOILS

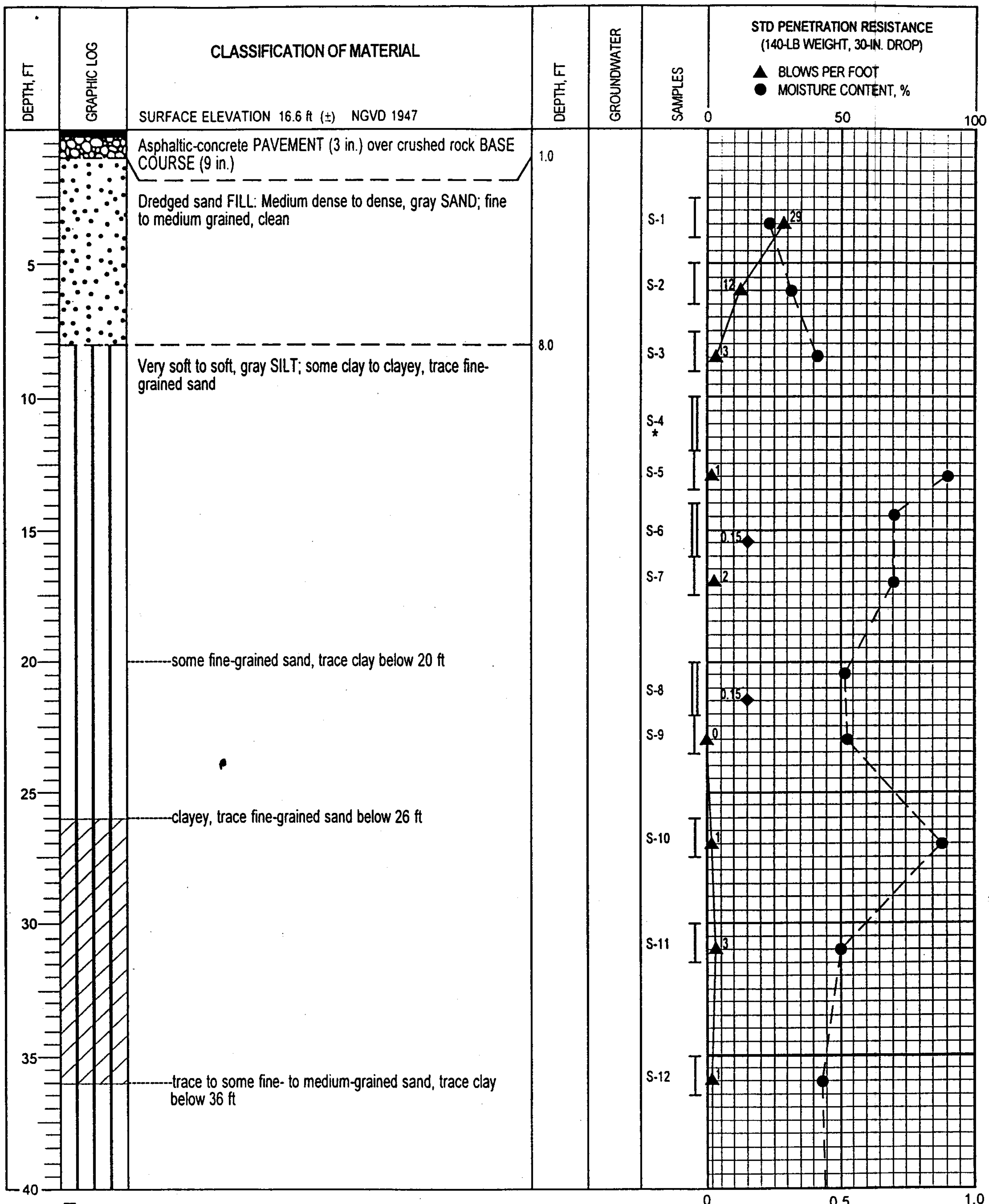
	Soil Type*			
	ML, SM	SM, SP, SW	SP, SW, GW	SW, GP
Relative Density	Cone Penetration Resistance, tsf			
Very Loose	0 - 8	0 - 14	0 - 20	0 –24
Loose	8 - 20	14 - 35	20 - 50	24 - 60
Med. Dense	20 - 60	35 - 105	50 - 150	60 –180
Dense	60 - 100	105 - 175	150 - 250	180 - 300
Very Dense	> 100	> 175	> 250	> 300

* Unified Soil Classification System

1) Friction ratio is equal to sleeve friction (tsf) divided by cone penetration (tsf) expressed as a percent.

2) Cone penetration test performed and interpreted by G2 CPT, LLC of Forest Grove, Oregon.

GRI



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

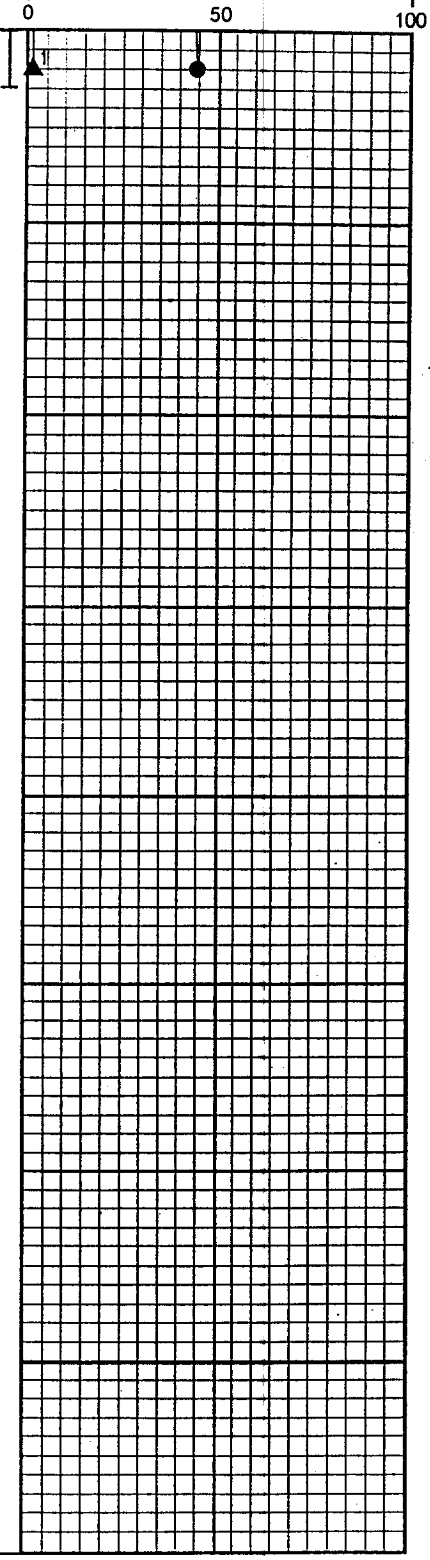
BORING B-1

SEPT. 2000 JOB. NO. 3324 FIG. 1A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION 16.6 ft (±) NGVD 1947	DEPTH, FT	GROUNDWATER	SAMPLES
40		Very soft, gray SILT; trace to some fine- to medium-grained sand, trace clay	41.5		S-13
		(9/11/2000)			
		Ground surface elevation provided by Olson Engineering			
45					

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)
▲ BLOWS PER FOOT
● MOISTURE CONTENT, %

0 50 100



0 0.5 1.0
(TONS PER FT ²)

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)

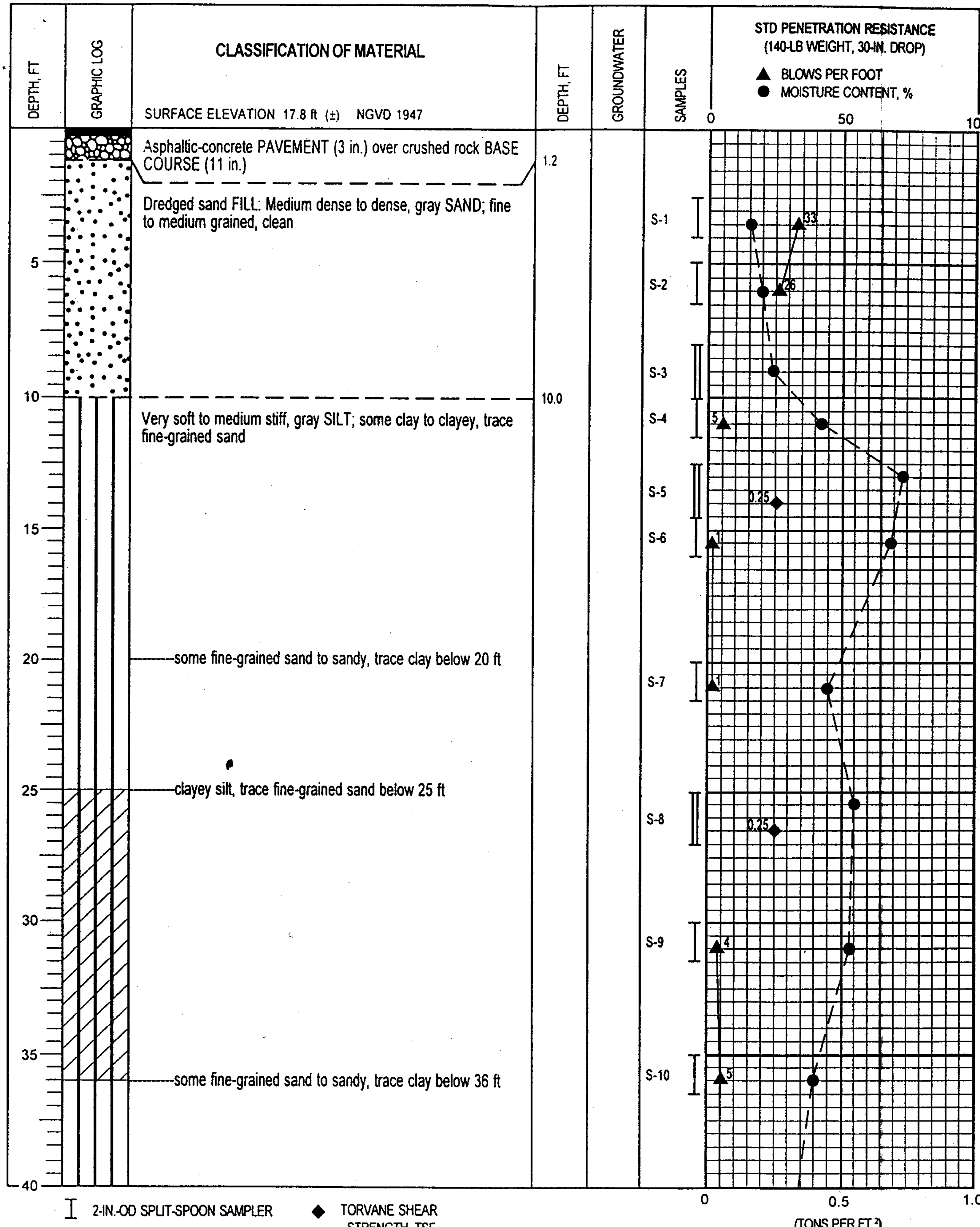
◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF
* NO RECOVERY

Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-1 (cont.)

SEPT. 2000 JOB. NO. 3324 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

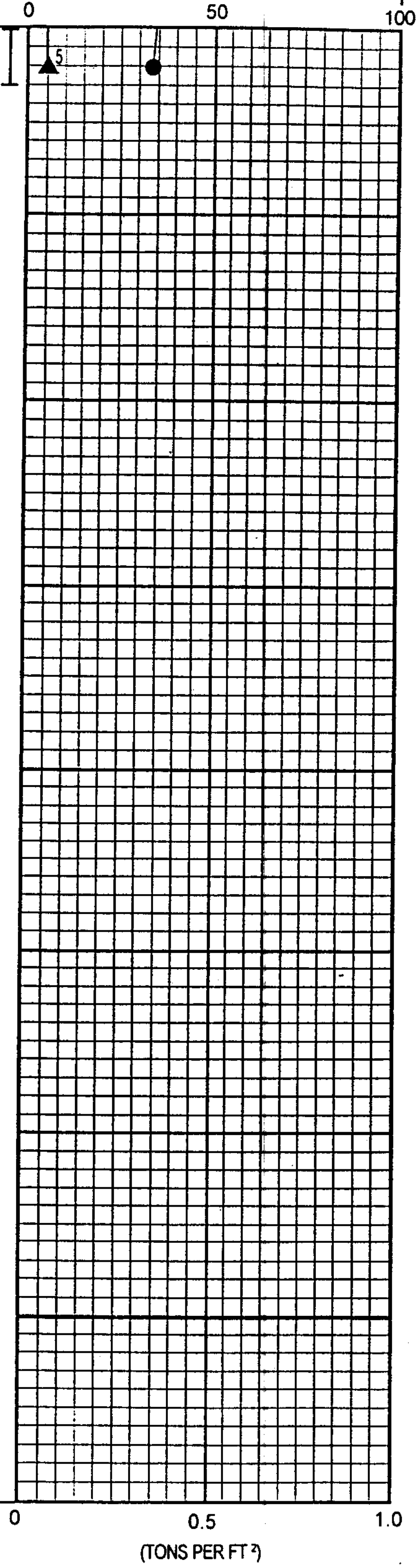
Plastic Limit

GRI

BORING B-2

SEPT. 2000 JOB. NO. 3324 FIG. 2A

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL SURFACE ELEVATION 17.8 ft (±) NGVD 1947	DEPTH, FT	GROUNDWATER	SAMPLES	STD PENETRATION RESISTANCE (140-LB WEIGHT, 30-IN. DROP) ▲ BLOWS PER FOOT ● MOISTURE CONTENT, %
40		Soft to medium stiff, gray SILT; some fine-grained sand to sandy, trace clay	41.5		S-11	
		(9/11/2000)				
		Ground surface elevation provided by Olson Engineering				
45						



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

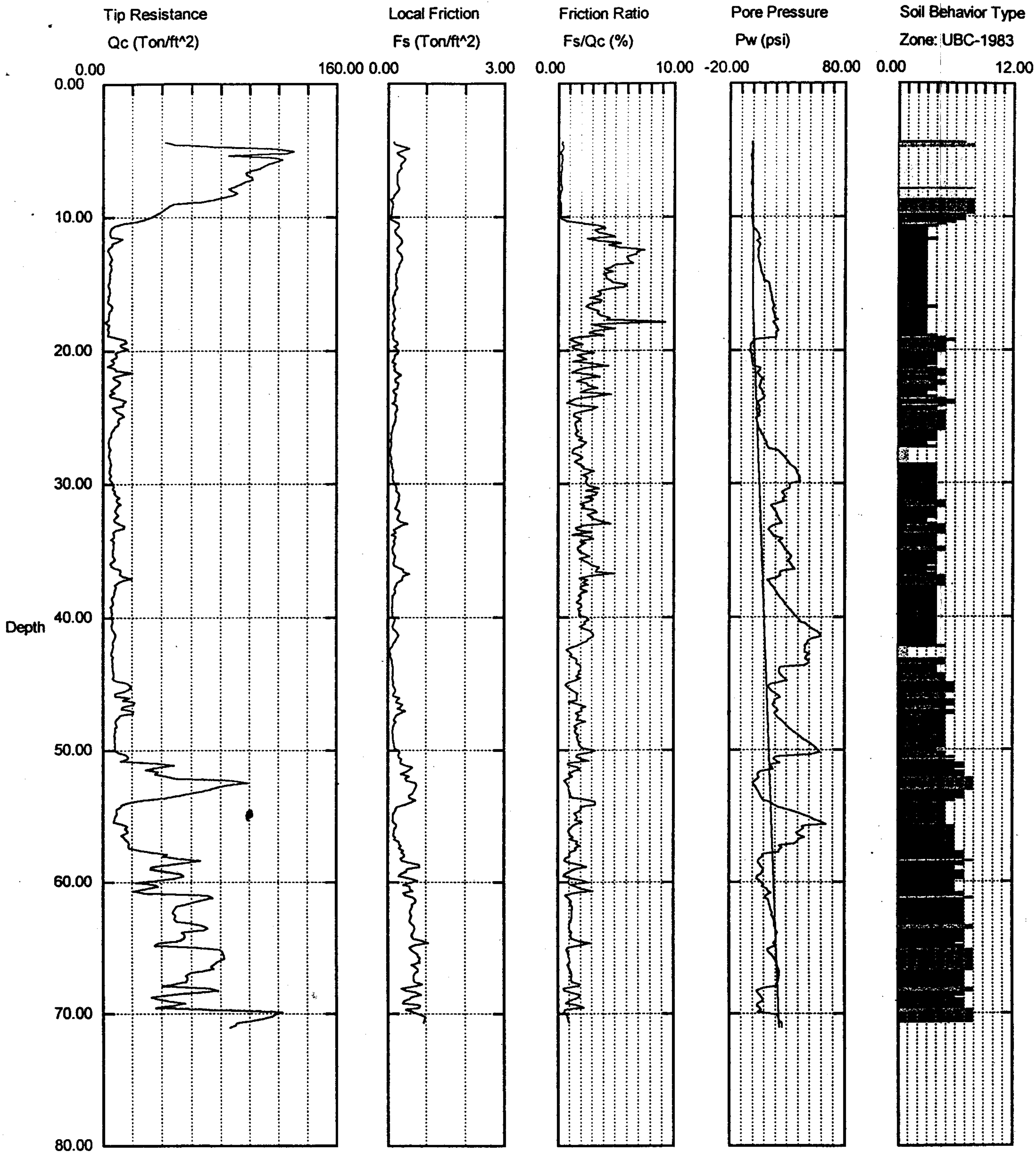
* NO RECOVERY

Liquid Limit
Moisture Content
Plastic Limit

GRI

BORING B-2 (cont.)

SEPT. 2000 JOB. NO. 3324 FIG. 2A



Maximum Depth = 71.03 feet

Depth Increment = 0.16 feet

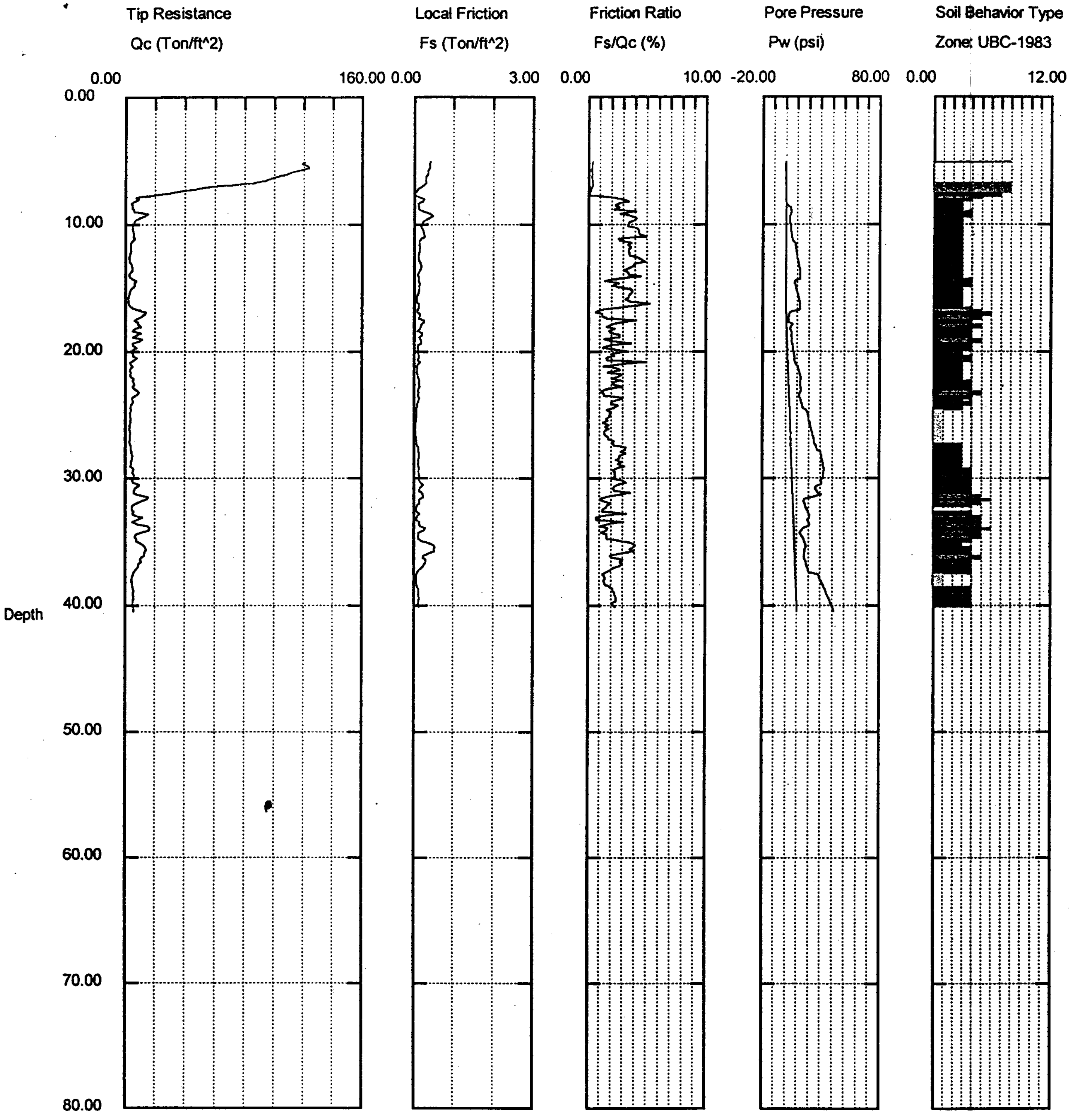
1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

GROUND SURFACE ELEVATION = 17.7 FT (PORT OF PORTLAND NGVD, 1947; ELEVATION PROVIDED BY OLSON ENGINEERING)

GRI

CONE PENETRATION TEST P-1

SEPT. 2000 JOB NO. 3324 FIG. 3A



Maximum Depth = 40.52 feet

Depth Increment = 0.16 feet

1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

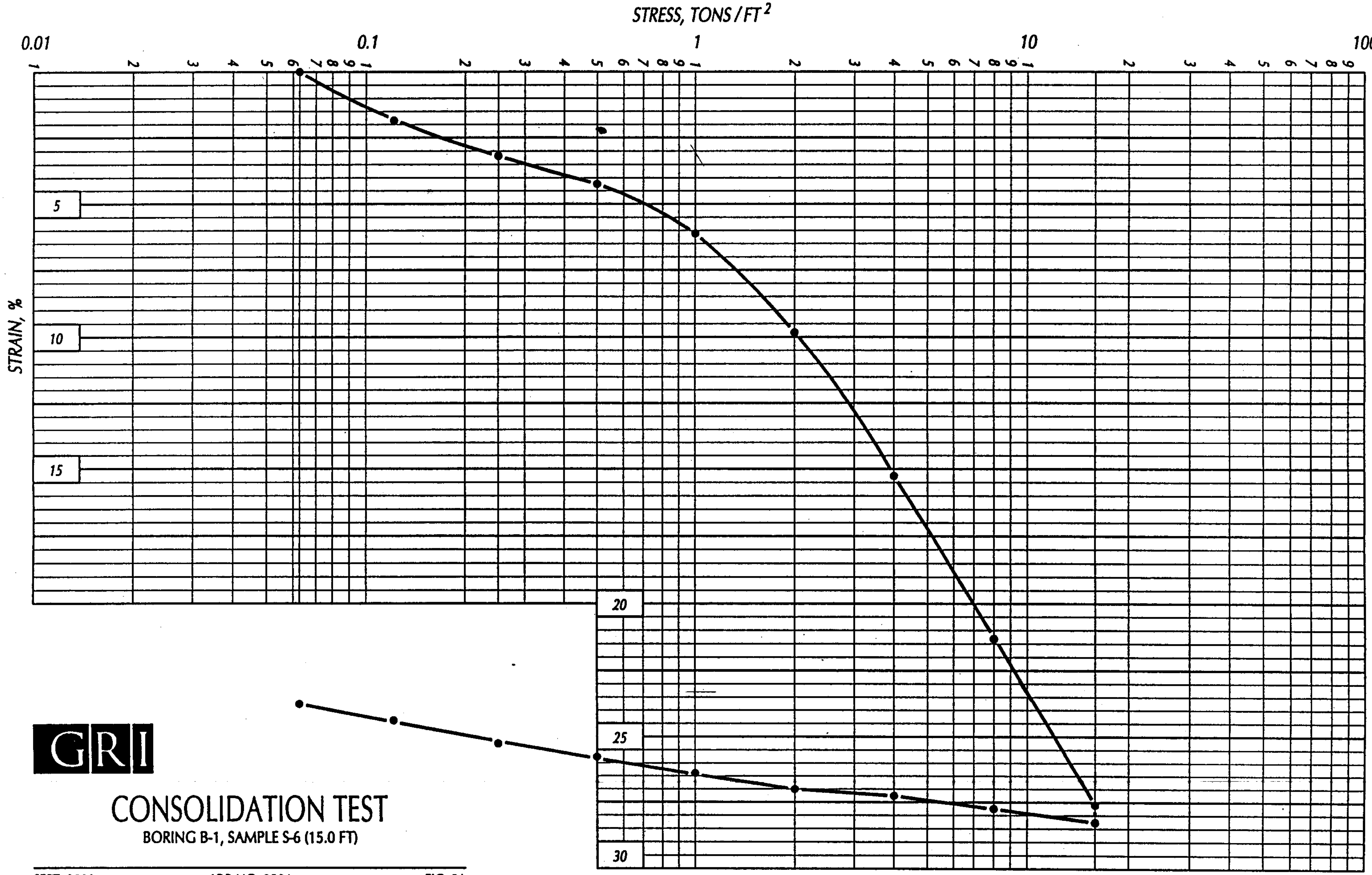
GROUND SURFACE ELEVATION - 16.2 FT (PORT OF PORTLAND NGVD, 1947; ELEVATION PROVIDED BY OLSON ENGINEERING)

GRI

CONE PENETRATION TEST P-2

SEPT. 2000 | JOB NO. 3324 | FIG. 4A

BORING	SAMPLE	DEPTH, FT	MOISTURE CONTENT, % (INITIAL)	MOISTURE CONTENT, % (FINAL)	DRY UNIT WEIGHT, PCF (INITIAL)	DRY UNIT WEIGHT, PCF (FINAL)	SOIL DESCRIPTION
B-1	S-6	15.0	55	39	66	87	GRAY SILT; TRACE CLAY AND SOME FINE-GRAINED SAND



GRI

CONSOLIDATION TEST

BORING B-1, SAMPLE S-6 (15.0 FT)

SEPT. 2000 JOB NO. 3324 FIG. 5A