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March 15, 2000

3188 GEOTECHNICAL RPT REV 1
(ISSUED 5/16/00)

gLAs Architectural Group
111 West 7th Avenue, Suite 300
Eugene, OR 97401

Attention: Trace Ward

SUBJECT: Geotechnical Investigation
Proposed National Oceanic and Atmospheric Administration (NOAA) Building
Hatfield Marine Science Center
Newport, Oregon

At your request, Geotechnical Resources, Inc. (GRI) has conducted a geotechnical investigation for the proposed National Oceanic and Atmospheric Administration (NOAA) Building at the Hatfield Marine Science Center (HMSC) on Yaquina Bay in Newport, Oregon. The Vicinity Map, Figure 1, shows the general location of the site. The NOAA site is located in the northeast portion of the HMSC campus. The purpose of our investigation was to evaluate subsurface conditions at the site and develop conclusions and recommendations for site preparation, foundation support, and other design and construction considerations. The investigation consisted of subsurface explorations, limited laboratory testing, and engineering analyses. This report describes the work accomplished and presents our recommendations regarding the geotechnical aspects of the project.

GRI completed a geotechnical investigation for the nearby research vessel dock improvements and ship operation building. The results of our investigation were summarized in our September 6, 1995, report to KPFF Consulting Engineers entitled, "Geotechnical Investigation, OSU Hatfield Marine Science Center Waterfront Improvements, Newport, Oregon."

PROJECT DESCRIPTION

Design of the new building has not been completed; however, based on preliminary information, we understand the site will be developed with one or two buildings with a total of 15,000 ft^2 of office, storage, and support space. Based on our discussions with you, we understand the buildings will likely be two-story, wood-frame structures with a concrete slab-on-grade floor. Based on our discussions with the project structural engineer, KPFF Consulting Engineers, we understand that preliminary column and wall loads are anticipated to be on the order of 60 kips and 5 kips/ft, respectively. Other than removing the existing settling pond structures, we anticipate site grading will consist of cuts and fills of less than 2 ft in height. No significant below-grade structures are planned.

SITE DESCRIPTION

General

Existing site conditions are shown on the Site Plan, Figure 2. As shown on Figure 2, the majority of the site is developed with two above-grade, concrete-lined settlement ponds. The existing ground surface around the settlement ponds typically ranges from 11 to 15 ft. All elevations discussed in this report are referenced to the National Geodetic Vertical Datum (NGVD). The top of the berm around the ponds is typically about elevation 20 to 21 ft, and berm slopes are approximately 2H:1V. The majority of the site, including the outer slopes of the pond berm, is covered with sparse vegetation.

The existing buildings at the HMSC are typically single-story, wood-frame structures. Based on our discussions with you, we understand the existing buildings are supported on spread footing foundations.

Geology

The site is underlain by marine deposits, which typically consist of sands and occasional gravels, over siltstone and sandstone. Some fill has probably been placed to achieve existing grades.

SUBSURFACE CONDITIONS

General

Subsurface materials and conditions at the site were investigated on February 25, 2000, with three borings, designated B-1 through B-3. The locations of the borings are shown on Figure 2. Available drilling locations were limited due to the steep side slopes of the existing seawater settling pond berms and the presence of numerous utilities including underground power and several seawater lines. Drilling locations were selected as close to the building footprint as site conditions would allow. The depth of the borings ranged from 36.5 to 46.5 ft. The field exploration and laboratory testing programs completed for the investigation are described in Appendix A. Logs of the borings are shown on Figures 1A through 3A. The terms used to describe the materials are defined in Table 1A.

Soils

Borings B-2 and B-3 indicate the site is mantled with at least 46.5 ft of sand. Based on our previous work for the waterfront improvements at the HMSC, we anticipate the site is underlain by approximately 50 ft of generally medium dense to dense sand. The sand is typically gray, medium to fine grained, and contains a trace to some silt and scattered shell fragments and organics. The relative density of the sand typically ranges from medium dense to dense based on N-values ranging from 13 to 48 blows/ft of penetration of the split-spoon sampler. Local zones of sand with a relative density of loose were encountered at depths of 27 to 37 ft and 28 to 35 ft in borings B-2 and B-3, respectively. The three borings were terminated in dense to very dense sand at depths of 36.5 to 46.5 ft. The natural moisture content of the sand samples typically ranges from about 20 to 30%. The majority of the sand contains a trace of silt and/or clay, but local layers have some silt and/or clay.

The surficial brown sand extends to a depth of 3 to 5 ft and is likely fill. The sand fill is substantially similar in appearance to the natural sand and is therefore difficult to differentiate.

Groundwater

Based on the appearance of the materials encountered in the borings, we anticipate that groundwater occurs at depths of between 5 and 10 ft below ground surface in the vicinity of the borings. Our previous experience in the vicinity of the site indicates the groundwater level will respond to tidal fluctuations and seasonal precipitation.

CONCLUSIONS AND RECOMMENDATIONS

General

Our borings disclosed that the site is mantled by at least 45 ft of sand. In our opinion, the proposed building or buildings can be supported by conventional spread footings. Local zones of relatively loose sand were encountered during completion of our subsurface investigation. In our opinion, there is significant risk of liquefaction of these sands during a subduction zone seismic event. In addition, the site is located in an area mapped as subject to tsunami hazards (Priest, 1995). The following sections of this report provide our conclusions and recommendations for support of the proposed building.

Site Preparation

The ground surface within the building area should be stripped of vegetation, surface organics, and loose surface soils. Stripping should generally be accomplished to a depth of about 3 to 6 in. Locally, greater amounts of stripping may be required to remove near-surface soils with significant amounts of organic material. In our opinion, the loose, organic surface soils should be removed from the site or stockpiled on site for use in landscape areas. The existing slabs that line the seawater settling ponds should be broken into pieces and removed from the site. Following site stripping, removal of the existing settling ponds, and excavation to subgrade level, the resulting subgrade surface should be evaluated by the geotechnical engineer. Areas of unsuitable materials such as organics, if present, should be excavated and replaced with structural fill. Soil from excavation of the pond berms may be suitable for use as structural fill, but should be evaluated by the geotechnical engineer prior to use.

Following stripping or excavation to subgrade level, the upper 12 in. of the exposed surface within pavement and building areas should be compacted to at least 95% of the maximum density as determined by ASTM D 698 with a medium- to heavy-weight, smooth, steel-wheeled, vibratory roller. Wetting of the soil during compaction may be required.

Structural Fill

We anticipate only minor amounts of structural fill (less than 2 ft) will be required for site grading of the building pad. In our opinion, all fills should consist of granular material, such as the on-site sand or sandy gravel, or fragmental rock with a maximum size of up to 6 in. Granular material used to construct structural fills during wet weather should not contain more than about 5% passing the No. 200 sieve (washed analysis). Fill should be placed in 10-in.-thick (loose) lifts and compacted with a vibratory roller to at least 95% of the maximum dry density as determined by ASTM D 698. Coarse fill material should be compacted until well keyed. Generally, a minimum of four to six passes with the roller are required to achieve compaction. Structural fill should extend a minimum horizontal distance of 10 ft beyond the limits of the building. Fill side slopes should be no steeper than 2H:1V.

All backfill placed in utility trench excavations within the limits of the building and paved areas should consist of sand, sand and gravel, or crushed rock with a maximum size of up to 1½ in., and with not more than 5% passing the No. 200 sieve (washed analysis). In our opinion, the granular backfill should be placed in lifts and compacted using vibratory plate compactors or tamping units to at least 95% of the maximum dry density as determined by ASTM D 698. Flooding or jetting the backfilled trenches with water to achieve the recommended compaction should not be permitted.

Foundation Support

Based on the current available design information, we understand that the building or buildings will consist of a relatively light, two-story structure. Maximum column and wall loads will be about 60 kips and 5 kips/ft, respectively. Foundation support for the building can be provided by conventional wall- and column-type spread footings. Footings should be established in firm, undisturbed soil or compacted structural fill at a minimum depth of 18 in. below the lowest adjacent finished grade. The width of footings should not be less than 18 in. for wall footings or 24 in. for isolated column footings. Footings established in accordance with these criteria can be designed on the basis of an allowable soil-bearing pressure of 3,000 psf. This value 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.

It is likely that during and after excavation, footing subgrades in sand will dry and become disturbed or loosened by foot traffic. Therefore, the bottom of all footing excavations in sand should be wetted and compacted with several passes of a hand-operated vibratory plate compactor immediately prior to placing the reinforcing steel for the footing. If the exposed subgrade subsequently dries and begins to "fluff," the subgrade should be wetted prior to placing concrete.

We estimate that the total settlement of spread footings designed in accordance with the recommendations presented above will be less than ½ in. Differential settlements between adjacent footings should be less than half the total settlement. Past experience indicates that these settlements will occur rapidly, with the majority of the settlement occurring during building construction.

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 passive soil resistance. The total frictional resistance between the footing and the soil is the normal force times the coefficient of friction between the soil and the base of the footing. We recommend an ultimate 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 soil or if backfill for the footings is granular structural fill.

Floor Support

We recommend the installation of a 6-in.-thick granular base course beneath the floor slab to provide more uniform support between the slab and the subgrade soil. The base course should consist of crushed rock of up to 1-in. size and having less than about 5% passing the No. 200 sieve (washed analysis). The base course material should be installed in a single lift and compacted to at least 95% of the maximum density as determined by ASTM D 698. Prior to installation of the base course material, the upper 12 in. of the subgrade should be compacted to at least 95% of the maximum dry density as determined by ASTM D 698. In our opinion, it is appropriate to assume a coefficient of subgrade reaction, k, of 200 pci for the design of floor slabs constructed as recommended above. It may also be appropriate to install a moisture-retarding membrane beneath slabs that will receive floor coverings or will be used to store moisture-sensitive materials. The membrane is typically installed in accordance with manufacturer's recommendations.

Seismic Considerations

The project site is currently assigned to seismic zone 3 in the 1997 Uniform Building Code (UBC). Based on the results of our subsurface explorations for this site and our previous work for the waterfront improvements at the HMSC, we anticipate the site is underlain by approximately 50 ft of generally medium dense to dense sand. A local layer of loose sand, approximately 10 ft thick, was encountered at a depth of approximately 30 ft in two of the borings. Previous work completed by GRI at the HMSC (north of the NOAA site) did not encounter similar layers of loose soil. Based on our review of the UBC and our subsurface explorations, we recommend use of a soil profile type S_D for the site. However, we anticipate that the majority of the Oregon coast, including the Newport area, will soon be assigned to seismic zone 4.

Based on our studies, we anticipate that the potential for earthquake-induced fault displacement and landslides at this site is low. However, due to the presence of relatively loose sands and the proximity of the tectonic plate boundaries to the site, the risk of liquefaction due to a relatively large, subduction zone seismic event (magnitude 8.5 or larger) is high, with estimated settlements on the order of 2 to 4 in. In addition, the risk of liquefaction due to a local crustal seismic event (magnitude 6.0 or larger) is moderate to high, with estimated settlements on the order of 1 to 3 in. We anticipate the settlement due to liquefaction will occur relatively rapidly, during and shortly after the seismic event, and will be relatively uniform across the site. Potential mitigation methods, if considered appropriate, include densification of the loose sand zones with ground improvement methods or supporting the building on piles. In addition, recent mapping of tsunami hazards along the Oregon coast indicate the site is located in an area of significant risk of inundation during or following a subduction zone seismic event (Priest, 1995).

DESIGN REVIEW AND CONSTRUCTION OBSERVATIONS

We welcome the opportunity to review and discuss plans and specifications as they are being developed. We are also of the opinion that to observe compliance with the design concepts and intent of the plans and specifications, a qualified geotechnical engineer should observe all operations dealing with earthwork and foundation construction.

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. 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, etc. In the event that any changes in the design and location of the facilities, 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 analyses and recommendations submitted in this report are based on the data obtained from the borings made at the locations indicated on the Site Plan 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 locations and, also, that groundwater levels may fluctuate from time to time. This report does not reflect any variations that may occur between these explorations. The nature and extent of variations may not become evident until construction. If, during construction, subsurface conditions different from those encountered in the borings are observed or encountered, we should be advised at once so that we can observe these conditions and reconsider our recommendations where necessary.

Submitted for GRI,

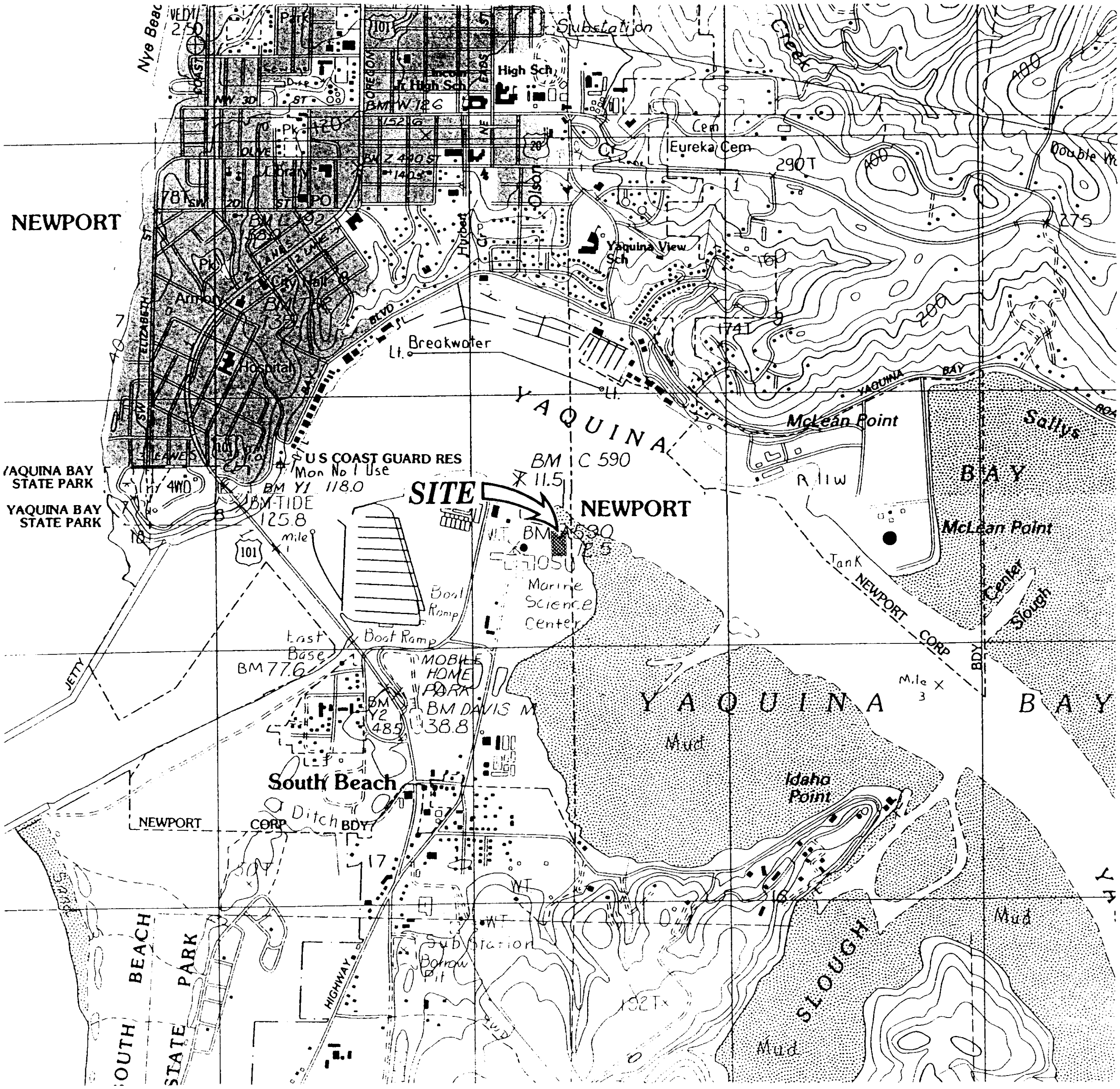
6/01

Dwight J. Hardin, PE
Principal

6/01

Robin L. Warren, PE, PG
Associate

Priest, G.R., December 1995, Explanation of Mapping Methods and Use of the Tsunami Hazard Maps of the Oregon Coast, State of Oregon Department of Geology and Mineral Industries, Open-file Report 0-95-67.



USGS TOPOGRAPHIC MAP
NEWPORT SOUTH, OREG. (1ba) QUAD (1984)

North

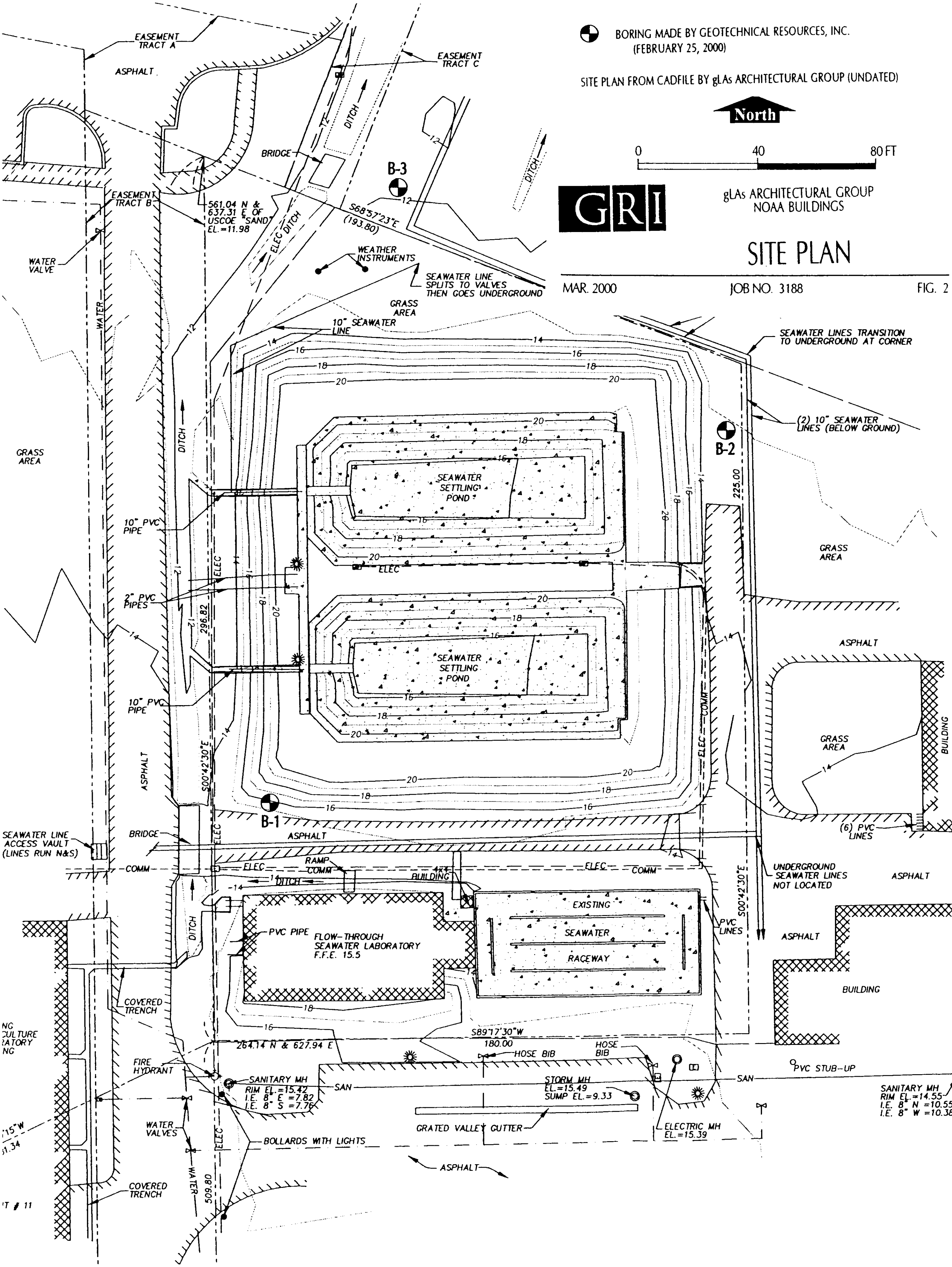
0 1/2 1 MILE

GRI

gLAs ARCHITECTURAL GROUP
NOAA BUILDINGS

VICINITY MAP

MAR. 2000 JOB NO. 3188 FIG. 1


BORING MADE BY GEOTECHNICAL RESOURCES, INC.
(FEBRUARY 25, 2000)
SITE PLAN FROM CADFILE BY gLAs ARCHITECTURAL GROUP (UNDATED)
North
0
40
80 FT
GRI
gLAs ARCHITECTURAL GROUP
NOAA BUILDINGS
SITE PLAN
MAR. 2000
JOB NO. 3188
FIG. 2
EASEMENT TRACT A
EASEMENT TRACT B
EASEMENT TRACT C
ASPHALT
BRIDGE
DITCH
ELEC DITCH
B-1
B-2
B-3
561.04 N & 637.31 E OF USCOE "SAND" EL.=11.98
S68°57'23"E (193.80)
WATER VALVE
WATER
WEATHER INSTRUMENTS
SEAWATER LINE SPLITS TO VALVES THEN GOES UNDERGROUND
GRASS AREA
10" SEAWATER LINE
SEAWATER LINES TRANSITION TO UNDERGROUND AT CORNER
(2) 10" SEAWATER LINES (BELOW GROUND)
225.00
SEAWATER SETTLING POND
10" PVC PIPE
2" PVC PIPES
ELEC
296.82
S00°42'30"E
COMM
BUILDING
(6) PVC LINES
SEAWATER LINE ACCESS VAULT (LINES RUN N&S)
RAMP
4x4 BUILDING
UNDERGROUND SEAWATER LINES NOT LOCATED
EXISTING SEAWATER RACEWAY
PVC PIPE
FLOW-THROUGH SEAWATER LABORATORY F.F.E. 15.5
PVC LINES
COVERED TRENCH
S89°17'30"W
180.00
264.14 N & 627.94 E
HOSE BIB
PVC STUB-UP
FIRE HYDRANT
SANITARY MH RIM EL.=15.42 I.E. 8" E =7.82 I.E. 8" S =7.76
SAN
STORM MH EL.=15.49 SUMP EL.=9.33
SANITARY MH RIM EL.=14.55 I.E. 8" N =10.55 I.E. 8" W =10.38
WATER VALVES
GRATED VALLEY GUTTER
ELECTRIC MH EL.=15.39
BOLLARDS WITH LIGHTS
509.80

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

General

Subsurface conditions and materials were investigated on February 25, 2000, with three borings, designated B-1 through B-3. The borings were drilled using mud-rotary techniques with a truck-mounted CME-75 drill rig provided and operated by Geo-Tech Explorations of Tualatin, Oregon. The approximate locations of the borings are shown on the Site Plan, Figure 2. Available drilling locations were limited due to the steep side slopes of the existing seawater settling pond berms and the presence of numerous utilities including underground power and several seawater lines. Drilling locations were selected as close to the building footprint as site conditions would allow.

All field operations were observed by a geotechnical engineer provided by our firm, who maintained a detailed log of the materials and conditions disclosed during the course of the work. A detailed description of the field explorations completed for this project is provided below.

Borings

The borings were completed to depths of 36.5 to 46.5 ft below the ground surface. Disturbed samples were obtained from the borings at 2.5- to 5-ft intervals of depth 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.

Logs of Borings

Logs of the borings are shown on Figures 1A through 3A. Each log presents a descriptive summary of the various types of material encountered 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. The terms used to describe the materials are defined in Table 1A.

LABORATORY TESTING

General

All samples obtained from the field exploration program were returned to our laboratory for examination and testing. The physical characteristics of the soils were noted and the field classifications were modified where necessary. The laboratory testing program included determinations of natural moisture content and washed sieve analyses.

Natural Moisture Content

Natural moisture content determinations were made in conformance with ASTM 2216. The results are shown on the Boring Logs, Figures 1A through 3A.

Washed Sieve Analysis

The silt and clay content (percent passing the No. 200 sieve) was evaluated for selected samples. Oven-dried samples were placed on the No. 200 sieve, the silt and clay fraction washed through the sieve, and the remaining sample was oven dried and weighed. The results of the tests are summarized below.

SUMMARY OF WASHED SIEVE ANALYSES

Location	Sample	Percent Passing No. 200 Sieve
B-1	S-5	2
	S-8	4
B-2	S-3	4
	S-10	14
	S-12	12
B-3	S-3	4
	S-9	15

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

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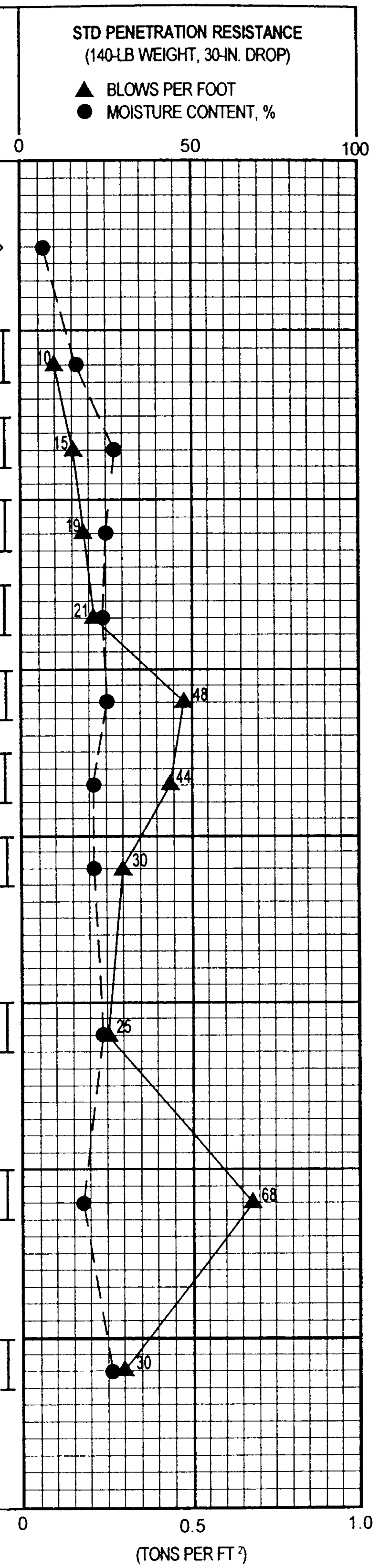
CLASSIFICATION OF MATERIAL

SURFACE ELEVATION 16 ft (±)

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL	DEPTH, FT
0–3.5		FILL (?): Medium dense, brown SAND; fine to medium grained, scattered shell fragments, trace silt, thinly rooted zone at the ground surface (probable fill)	3.5
3.5–15.0		Medium dense, gray SAND; fine to medium grained, trace silt --thin (1/16-in.) seam of organics at 7.5 ft	15.0
15.0–36.5		Dense, gray SAND; fine to medium grained, trace silt --predominately fine grained, scattered shell fragments below 25 ft --grades to very dense below 27 ft	36.5
		(2/25/2000)	

GROUNDWATER: (2/25/00)

SAMPLES: S-1 (G), S-2, S-3, S-4, S-5, S-6, S-7, S-8, S-9, S-10, S-11



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

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