GRI
9725 SW Beaverton-Hillsdale Hwy, Suite 140
Beaverton, OR 97005-3364
p| 503-641-3478 f| 503-644-8034

December 12, 2011 5277GEOTECHNICAL RPT

City of Astoria
1095 Duane Street
Astoria, OR 97013

Attention: Jeff Harrington, PE

SUBJECT: Geotechnical Investigation
Legion Block Amphitheatre and Public Space
Garden of Surging Waves
Astoria, Oregon

At your request, GRI has completed a geotechnical investigation for the proposed amphitheatre and public space on the Legion Block located between 11th and 12th streets and Exchange and Duane streets in Astoria, Oregon. The Vicinity Map, Figure 1, shows the general location of the site. The purpose of our investigation was to evaluate subsurface conditions at the site and develop conclusions and recommendations for construction of the proposed improvements. The investigation included subsurface explorations, laboratory testing, and engineering analyses. This report describes the work accomplished and provides our conclusions and recommendations for use in design and construction of the improvements.

PROJECT DESCRIPTION

The configuration of the existing site is shown on the Site Plan, Figure 2. The first phase of site improvements will include the Garden of Surging Waves, which will be constructed on the northwest corner of the site. As funding becomes available, additional improvements will be constructed on the site, including a shallow amphitheater in the area of the exposed basement on the north portion of the property and a structured pavilion space in the southern portion of the site. We understand the maximum depth of the amphitheater will be about 3 ft. The American Legion Hall located on the southern portion of the site will be integrated into the redevelopment.

Plans for the project are still in the preliminary stage; however, based on our conversations with AAI Engineering, the project structural engineer, we understand the central structure in the Garden of Surging Waves will have column loads of less than 200 kips. Additional ancillary structures are expected to be relatively lightly loaded. Based on our conversation with the design team, we understand spread footings are the preferred foundation type. The majority of the site will likely be surfaced with Portland Cement Concrete (PCC).

SITE DESCRIPTION

Surface Conditions and Topography

The site was previously occupied by a Safeway store, and the main building has been removed, leaving an exposed basement in the center of the northern portion of the site. The eastern half of the site has a structured slab at street grade that is supported on columns and spread footings. The depth to the ground surface beneath the slab varies from about 10 ft on the north side of the site to about 7.5 ft on the south side. The American Legion Hall located near the center of the southern portion of the site will remain. The western half of the site is paved with asphaltic-concrete (AC) pavement that is underlain by fill. The existing ground surface is relatively flat and ranges from about elevation 21 to 23 ft (NAVD 1988).

Geology

Variable fill mantles the site to depths of about 18 to 21 ft and possibly more than 40 ft below the ground surface. The fill is underlain by sand, which is underlain by weathered siltstone of the Astoria Formation.

General

Subsurface materials and conditions at the site were investigated on November 17 and 18, 2011, with four borings, designated B-1 through B-4. The borings were advanced to depths of 31.5 to 41.5 ft below the pavement and slab surfaces at the locations shown on Figure 2. The field and laboratory testing programs completed for this project are described in Appendix A. Logs of the borings are provided on Figures 1A through 4A. The terms used to describe the soils and rock encountered in the borings are defined in Tables 1A and 2A.

Soils

For the purpose of discussion, the materials disclosed by the borings have been grouped into the following major units based on their physical characteristics and engineering properties.

1. **PAVEMENT**
2. **FILL (Silt, Sand, and Gravel)**
3. **SAND (Possible Fill)**
4. **BASALT Fragments**
5. **SILTSTONE**

The following paragraphs provide a detailed description of the soil units and a discussion of the groundwater conditions at the site.

1. PAVEMENT. A 2-in.-thick layer of AC pavement was encountered at the ground surface in borings B-1 and B-3 on the west side of the site. The AC is underlain by about 28 in. of crushed rock base course. Borings B-2 and B-4 were advanced through holes cored in the structured concrete slab, which varies in thickness from about 5 to 6 in. at borings B-2 and B-4, respectively. The ground surface is located about 10 to 7.5 ft below the surface of the slab in borings B-2 and B-4, respectively.

2. FILL (Silt, Sand, and Gravel). Beneath the AC pavement on the west side of the site, and at the ground surface beneath the structured slab on the east side of the site, the site is mantled with variable fill. The thickness of fill ranges from about 12.5 to 21.5 ft and is possibly more than 40 ft thick in boring B-1. The fill typically consists of silt, sand, and gravel. Borings B-1 and B-3 on the west side of the site encountered a 6- to 10-ft-thick layer of clayey silt fill beneath the pavement base course. The clayey silt fill is rust-brown and contains some fine- to coarse-grained sand and scattered gravel and siltstone fragments. Borings B-2 and B-4 on the east side of the site encountered a 1.5- to 2.5-ft-thick layer of dark gray silt sill fill beneath a surficial layer of sand fill. The dark gray silt fill contains a trace to some fine-grained sand and fine organics. N-values of 0 to 7 blows/ft and Torvane shear strength values of 0.3 to 0.9 tsf indicate the relative consistency of the silt fill ranges from very soft to stiff. The natural moisture content of the silt fill ranges from about 35 to 74%.

Beneath the silt fill on the west side of the site and at the ground surface beneath the slab on the east side of the site, the borings encountered sand fill. The thickness of the sand fill ranges from about 7 to 12 ft in borings B-1 and B-3 and about 2.5 to 10 ft in borings B-2 and B-4. The sand is gray, fine grained, and contains varying percentages of silt ranging from a trace of silt to silty, scattered gravel, and fine organics. N-values of 1 to 9 blows/ft indicate the relative density of the sand fill is loose. The natural moisture content of the sand fill ranges from about 29 to 53%. Abundant wood debris, which is likely associated with the December 7, 1922, Astoria fire, was encountered at the base of the sand fill at depths of about 19 to 23 ft below the existing site grade in all of the borings.

Beneath the sand fill in borings B-3 and B-4, the borings encountered a 1.5- to 3-ft thick layer of gravel fill that ranges from rounded to angular and contains a matrix of silt, sand, and clay. N-values of 27 to 55 blows/ft indicate the relative density of the gravel fill is medium dense to very dense.

3. SAND (Possible Fill). The fill described above is underlain by sand that extends to a maximum depth of 39 ft; B-2 was terminated in sand at a depth of 31.5 ft below the surface of the existing concrete slab. The sand is gray, fine grained, and contains varying percentages of silt ranging from a trace of silt to silty. Scattered gravel, siltstone fragments, and fine organics are present within the sand and suggest the material is possibly fill. N-values of 2 to 21 blows/ft indicate the relative density of the sand ranges from loose to medium dense and is typically loose. An N-value of 39 was recorded in the sand in boring B-2 for sample S-8; however, in our opinion, the elevated value is due to a cobble or boulder encountered immediately prior to sampling S-8. The natural moisture content of the sand ranges from about 20 to 42%.

4. BASALT Fragments. Cobble- to boulder-size fragments of dark gray basalt were encountered beneath the sand in borings B-1 and B-3 and are likely fill. The thickness of the unit is about 2 to 2.5 ft in boring B-3; boring B-1 was terminated in basalt fragments at a depth of 41.5 ft below the ground surface. Based on the drill action, we estimate the fragments are angular and the size of cobbles or boulders. An N-value of 59 blows/ft indicates the basalt fragments are very dense.

5. SILTSTONE. Boring B-3 encountered siltstone beneath the basalt fragments, and boring B-4 encountered siltstone beneath the alluvial sand. The siltstone is typically gray brown, is extremely soft (R0), and its relative weathering is fresh. Borings B-3 and B-4 were terminated in siltstone at a depth of 40 and 36.5 ft, respectively.

Groundwater

The borings were advanced using mud-rotary methods, which does not permit the observation of groundwater conditions during drilling. Based on our experience with other nearby projects, we anticipate the groundwater level at the site will fluctuate with seasonal precipitation and the level of the Columbia River. We anticipate the local groundwater level typically occurs at depths of about 10 to 15 ft; however, shallow perched groundwater levels can occur following prolonged, intense rainfall. Shallow groundwater will also occur during flood stages of the Columbia River.

CONCLUSIONS AND RECOMMENDATIONS

General

Subsurface explorations for this investigation indicate the site is underlain by 12.5 to 21.5 ft, and possibly more than 40 ft, of variable fill and wood debris. Beneath the fill and debris, the site is underlain by 13 to 19 ft of loose to medium dense sand, which is underlain by siltstone of the Astoria Formation. We anticipate the local groundwater level with fluctuate with seasonal precipitation and the level of the nearby Columbia River. Our analyses indicate liquefaction of the loose sands below the water table is likely to occur to a depth of about 40 ft.

In our opinion, structural loads for the proposed improvements can be supported by conventional spread footings established on compacted structural fill. The primary geotechnical considerations associated with design and construction of the proposed improvements include the presence of variable, compressible fill and debris and the potential for liquefaction-induced settlement during a design-level earthquake. Some overexcavation of the fill will be necessary to reduce foundation settlement and improve pavement performance. The following sections of this report provide our conclusions and recommendations concerning seismic design considerations, site preparation and grading, structural fill, lateral earth pressure, and design and construction of foundations and pavements.

Seismic Considerations

The potential for liquefaction and liquefaction-induced settlements at the site was evaluated using LiquefyPro, a seismically induced liquefaction and settlement analysis software developed by CivilTech Corporation. Input values for peak ground surface acceleration and earthquake magnitude used in the analysis are consistent with the 2002 U.S. Geological Survey (USGS) seismic hazard deaggregations, which serve as the probabilistic basis for the 2009 IBC. The results of this analysis indicate liquefaction of the loose sands below the water table is likely to occur to a depth of about 40 ft. Based on our studies, we estimate that liquefaction-induced settlements during a design-level earthquake could be on the order of about 10 to 11 in. This estimate should be considered approximate and is likely conservative. We understand no mitigation measures are currently planned for the project.

Due to the potential for liquefaction, the site is identified as a Site Class F, in accordance with the 2009 International Building Code (IBC) and 2010 Oregon Structural Specialty Code. This Site Class designation requires a site-specific seismic hazard study, except for structures with periods of vibration less than or equal to 0.5 seconds. Based on our understanding of the proposed improvements, periods will likely be less than 0.5 seconds, and, consequently, Site Class can be evaluated on the basis of observed soil characteristics, as defined in Section 1613.5.5.1 of the IBC. Based on our review of subsurface conditions at the site, the soil profile at the site is classified as Site Class D. The IBC design methodology uses two spectral response coefficients, S_S and S_1, corresponding to periods of 0.2 and 1.0 seconds to develop the design earthquake spectrum. The spectral response coefficients were obtained from the U.S. Geological Survey (USGS) Uniform Hazard Response Spectra Curves for the coordinates of 46.19° N latitude and 123.83° W longitude. The IBC S_S and S_1 coefficients identified for the site are 1.318 g and 0.650 g, respectively.

Site Preparation and Grading

The ground surface over the area of improvements including walkways and pavement areas should be stripped of existing structures, pavements, and underground utilities. Excavations made for demolition should be backfilled with structural fill. Existing basement walls should be removed to a depth of at least 2 ft below proposed footing, slab, or pavement grades, and existing basement floor slabs should be broken prior to fill placement to allow for drainage. Existing concrete slabs, sidewalks, and basement walls demolished during site stripping may be reduced to fragments less than 2 in. in maximum dimension and incorporated into the amphitheatre fill.

Upon completion of site stripping and excavation to subgrade level, the resulting subgrade should be evaluated by a qualified geotechnical engineer. Any areas of soft or unsuitable material should be overexcavated to firm undisturbed soil and backfilled with structural fill. In this regard, we anticipate overexcavation will be required to establish foundation and pavement subgrades on the west side of the site. Recommendations for foundation and pavement overexcavation are included in the following sections.

Based on the borings, silt fill materials will likely be present at subgrade levels on the west side of the site. The silt soils are easily disturbed and softened by construction traffic and other activities. For this reason, grading will be more straightforward if accomplished during the normally drier summer and fall months. If construction is to proceed during wet conditions, we recommend that construction equipment not traffic the fine-grained subgrade (silt) soils. This will require placing granular fill for a working pad to protect the subgrade. A 12-in.-thick granular work pad placed over a woven geotextile should be sufficient to prevent disturbance of the subgrade by lighter construction equipment. A granular work pad on the order of 18 to 24 in. thick is typically required to protect fine-grained subgrade soils from disturbance by repetitive heavy construction loads. If the subgrade is disturbed during construction, soft disturbed soils should be overexcavated to firm soil and backfilled with granular structural fill.

Structural Fill

It is anticipated that structural fill will be required to establish site grades for walkways and paved areas and to fill the portions of the existing exposed basement to construct the amphitheatre. In our opinion, granular material would be most suitable for construction of the structural fills. Granular material, such as sand, sandy gravel, or fragmental rock with a maximum size of about 1 1/2 in. would be suitable structural fill material. Granular fill placed during wet conditions should be relatively clean and have less than about 5% passing the No. 200 sieve (washed analysis). Granular fill should be placed in 12-in.-thick (loose) lifts and compacted to at least 95% of the maximum dry density as determined by ASTM D 698, or until well keyed with a vibratory roller. Fill placed in landscaped areas should be compacted to a minimum of about 90% of ASTM D 698.

All backfill placed in utility trench excavations within the limits of the buildings, walkways, and paved areas should consist of granular structural fill. Trench backfill can consist of sand, sandy gravel, or crushed rock of up to 1 1/2-in. maximum size and having less than about 5% passing the No. 200 sieve (washed analysis). The granular backfill should be compacted 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.

Lateral Earth Pressures

Design lateral earth pressures for embedded walls depend on the drainage condition behind the wall and the ability of the wall to yield. Assuming the embedded walls of the amphitheater will be fully drained and restrained by the surrounding pavement, i.e., a rigid non-yielding wall, we recommend designing the walls on the basis of an equivalent fluid having a unit weight of 55 pcf. Additional lateral loading due to surcharge loads can be evaluated using the criteria shown on Figure 3.

Horizontal pressures due to seismic loads may be estimated on the basis of an equivalent fluid having a unit weight of 20 pcf. The resultant of the seismic force acts at a distance of 0.6H above the base of the wall, where H is the height of the wall. The lateral force induced by an earthquake is in addition to the lateral earth pressures acting on the wall during static conditions.

Permanent drainage should be provided for all embedded walls, as shown on Figure 4. Free-draining wall backfill should consist of clean, granular structural fill material compacted to about 92% of the maximum dry density determined by ASTM D 698. A 1-ft-wide blanket of open-graded drain rock with less than about 2% passing the No. 200 sieve (washed analysis) should be placed against the wall. Crushed drain rock of $^{3}/_{4}$- to $1^{1}/_{2}$-in. size is suitable for this purpose. Overcompaction of backfill behind walls should be avoided. Heavy compactors and large pieces of construction equipment should not operate within 5 ft of any embedded wall to avoid the buildup of excessive lateral pressures. Compaction close to the walls should be accomplished with hand-operated vibratory plate compactors. Overcompaction of backfill could significantly increase lateral earth pressures behind walls.

Foundation Support

Subsurface explorations disclosed 6 to 10 ft of compressible silt fill beneath the pavement base course on the west side of the site. To reduce the anticipated total and differential settlements, the existing silt fill below the pavement base course and within the foundation excavation for the new structures should be overexcavated a minimum of 2 ft and replaced with granular structural fill to the limits indicated on Figure 5. We recommend all footing excavations and subgrades be evaluated by a geotechnical engineer as the work progresses.

In our opinion, following preparation of the foundation subgrade as described above, foundation support for the improvements can be provided by conventional spread footings. Footings should be established in compacted structural fill at a minimum depth of 2 ft 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.

Spread footings established in accordance with the above criteria can be designed to impose an allowable soil bearing pressure of up to 1,500 psf. This value applies to the total of real loads, i.e., dead load plus frequently and/or permanently applied live loads. The allowable bearing pressure can be increased by one-third for the total of all loads; including dead, live, and wind or seismic forces. We estimate the total settlement of spread footings supporting column loads of up to 200 kips will be on the order of 1 in. Differential settlements between adjacent comparably loaded footings should be less than half the total settlement.

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 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 0.35 for the coefficient of friction for footings cast on fine-grained soils. 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 250 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 placed as granular structural fill.

Subdrainage and Floor Support

Groundwater levels can be expected to rise to near the existing ground surface following prolonged, intense rainfall or during flood stages of the Columbia River. Therefore, structures embedded below existing site grades, such as the amphitheater, should be provided with a subdrainage system to reduce hydrostatic pressure and the risk of groundwater entering through embedded walls and floor slabs. Typical subdrainage details for embedded structures are shown on Figure 4. The figure shows peripheral subdrains to drain embedded walls and a granular drainage blanket beneath the concrete floor slab, which is drained by a system of subslab drainage pipes. All groundwater collected should be drained by gravity or pumped from sumps into the storm sewer system.

To provide more uniform support and facilitate drainage, we recommend placing a minimum 8 in. of free-draining, clean, angular rock beneath embedded floor slabs. This material should consist of angular rock such as 1 1/2- to 3/4-in.-size crushed rock, with less than 2% passing the No. 200 sieve (washed analysis), and should be placed in one lift and compacted to at least 95% of the maximum dry density according to ASTM D 698 or until well-keyed. In our opinion, it is appropriate to assume a coefficient of subgrade reaction, k, of 175 pci to characterize the subgrade support with 8 in. of compacted crushed rock beneath the slab.

PCC Pavement Design

Our observations indicate the pavement on the west side of the site has not performed well and has numerous cracks and distressed areas. Due to the poor condition of the existing pavements and the compressible fill disclosed in the borings, we recommend paved areas within the compressible fill be overexcavated 12 in. The subgrade should be blanketed with a woven geotextile before backfilling with granular structural fill. In areas where a granular work pad has been previously placed, the work pad may be incorporated into the recommended overexcavated section.

We anticipate the open areas around the improvements will be subjected to primarily pedestrian traffic with occasion light truck traffic for maintenance. We anticipate the majority of the site will be paved with PCC pavement. Based on our experience with similar projects and subgrade materials, we recommend a minimum pavement thickness of 6 in., underlain by 12 in. of base course. In areas where compressible silt fill has been overexcavated and replaced with granular structural fill, the granular fill can be considered as base course.

The recommended pavement section is based on the assumption that pavement construction will be accomplished during the dry season and after construction of larger project elements. If wet-weather pavement construction is considered, it will likely be necessary to increase the thickness of crushed rock base course to support construction equipment and protect the subgrade from disturbance. The recommended pavement section is not intended to support extensive construction traffic, such as dump trucks and concrete trucks. Pavements subject to construction traffic may require repair.

For the recommended pavement section, drainage is an essential aspect of pavement performance. We recommend all paved areas be provided with positive drainage to remove surface water and water within the base course. This will be particularly important in cut sections or at low points within the paved areas, such as at catch basins. Effective methods to prevent saturation of the base course materials include providing weep holes in the sidewalls of catch basins, subdrains in conjunction with utility excavations, and separate trench drain systems. To provide quality materials and construction practices, we recommend the pavement work conform to Oregon Department of Transportation standards.

Prior to placing base course materials, all pavement areas should be proof rolled with a fully loaded 10-cy dump truck. Any soft areas detected by the proof rolling should be overexcavated to firm ground and backfilled with compacted structural fill.

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. In addition, 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 our 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 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 earthwork and design and construction of floor support, foundations, and pavements. In the event that any changes in the design and location of the 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 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 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.

Submitted for GRI,

Michael W. Reed, PE, GE
Principal

Tamara G. Kimball, PE, GE
Project Engineer

This document has been submitted electronically.

APPENDIX A

Field Explorations and Laboratory Testing

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

Subsurface materials and conditions at the site were investigated on November 17 and 18, 2011, with four borings, designated B-1 through B-4. The borings were advanced to depths of about 31.5 to 41.5 ft at the locations shown on Figure 2. The borings were drilled using mud-rotary drilling techniques with a track-mounted drill rig provided and operated by Western States Soil Conservation of Hubbard, Oregon. The field exploration work was coordinated and documented by an experienced geotechnical engineer from GRI, who maintained a detailed log of the materials and conditions disclosed during the course of the work.

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.

Relatively undisturbed 3.0-in.-O.D. Shelby tube samples were obtained by pushing the tubes into undisturbed soil using the hydraulic rams on the drill rig. The soils exposed in the ends of the Shelby tubes were examined and classified in the field. The ends of the tubes were sealed with rubber caps and returned to our laboratory for further examination and physical testing.

Logs of the borings are provided on Figures 1A through 4A. 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, percent passing the No. 200 sieve, and Torvane shear strengths. The terms used to describe the soil and rock are defined in Tables 1A and 2A.

The following sections provide a detailed description of the laboratory testing completed for this project.

LABORATORY TESTING

General

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

Natural Moisture Content

Natural moisture content determinations were made in conformance with ASTM 2216. The results are provided on Figures 1A through 4A.

Grain Size Analysis (Washed Sieve)

Washed sieve analyses were performed on representative samples of the soils to assist in their classification and evaluation of liquefaction potential. The test is performed by taking a sample of known dry weight and washing it over a No. 200 sieve. The material retained on the No. 200 sieve is oven-dried and re-weighed, and the percentage of material (by weight) that passed the No. 200 sieve is calculated. The test results are tabulated below.

Boring	Sample	Depth, ft	% Passing No. 200 Sieve	Classification
B-1	S-7	20.0	48	Silty SAND (Possible Fill)
B-2	S-4	10.0	42	FILL: Silty SAND
	S-9	30.0	37	Silty SAND (Possible Fill)
B-3	S-5	12.5	7	FILL: SAND; trace silt
B-4	S-5	12.5	32	FILL: Silty SAND
	S-8	25.0	46	Silty SAND (Possible Fill)

Undisturbed Unit Weight

The unit weight, or density, of two undisturbed soil samples was determined in the laboratory in substantial conformance with ASTM D 2937. The unit weight determinations are summarized in the following table.

SUMMARY OF UNIT WEIGHT DETERMINATIONS

Boring	Sample	Depth, ft	Dry Unit Weight, pcf	Natural Moisture Content, %	Soil Type
B-3	S-3	7.5	77	42	FILL: Clayey SILT
B-4	S-4	11.0	54	74	FILL: SILT

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 as the instrument is rotated is measured using a calibrated spring. The results of the Torvane shear tests are shown on Figures 3A and 4A.

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

GUIDELINES FOR CLASSIFICATION OF ROCK

RELATIVE ROCK WEATHERING SCALE:

Term	Field Identification
Fresh	Crystals are bright. Discontinuities may show some minor surface staining. No discoloration in rock fabric.
Slightly Weathered	Rock mass is generally fresh. Discontinuities are stained and may contain clay. Some discoloration in rock fabric. Decomposition extends up to 1 in. into rock.
Moderately Weathered	Rock mass is decomposed 50% or less. Significant portions of rock show discoloration and weathering effects. Crystals are dull and show visible chemical alteration. Discontinuities are stained and may contain secondary mineral deposits.
Predominantly Decomposed	Rock mass is more than 50% decomposed. Rock can be excavated with geologist's pick. All discontinuities exhibit secondary mineralization. Complete discoloration of rock fabric. Surface of core is friable and usually pitted due to washing out of highly altered minerals by drilling water.
Decomposed	Rock mass is completely decomposed. Original rock "fabric" may be evident. May be reduced to soil with hand pressure.

RELATIVE ROCK HARDNESS SCALE:

Term	Hardness Designation	Field Identification	Approximate Unconfined Compressive Strength
Extremely Soft	R0	Can be indented with difficulty by thumbnail. May be moldable or friable with finger pressure.	< 100 psi
Very Soft	R1	Crumbles under firm blows with point of a geology pick. Can be peeled by a pocket knife and scratched with fingernail.	100 - 1,000 psi
Soft	R2	Can be peeled by a pocket knife with difficulty. Cannot be scratched with fingernail. Shallow indentation made by firm blow of geology pick.	1,000 - 4,000 psi
Medium Hard	R3	Can be scratched by knife or pick. Specimen can be fractured with a single firm blow of hammer/geology pick.	4,000 - 8,000 psi
Hard	R4	Can be scratched with knife or pick only with difficulty. Several hard hammer blows required to fracture specimen.	8,000 - 16,000 psi
Very Hard	R5	Cannot be scratched by knife or sharp pick. Specimen requires many blows of hammer to fracture or chip. Hammer rebounds after impact.	> 16,000 psi

RQD AND ROCK QUALITY:

Relation of RQD and Rock Quality		Terminology for Planar Surface		
RQD (Rock Quality Designation), %	Description of Rock Quality	Bedding	Joints and Fractures	Spacing
0 - 25	Very Poor	Laminated	Very Close	< 2 in.
25 - 50	Poor	Thin	Close	2 in. – 12 in.
50 - 75	Fair	Medium	Moderately Close	12 in. – 36 in.
75 - 90	Good	Thick	Wide	36 in. – 10 ft
90 - 100	Excellent	Massive	Very Wide	> 10 ft