# Geotechnical Engineering Report
# Make Beacon Hill Public
Spokane, WA

*Prepared for:*

Craig Andersen, PLA, LEED AP
AHBL, Inc.
601 W. Main Avenue, Suite 305
Spokane, WA 99201

*Prepared by:*

Budinger & Associates, Inc.
1101 N. Fancher Road
Spokane Valley, WA 99212

JOHN EDWARD FINNEGAN
STATE OF WASHINGTON
0033932
REGISTERED
PROFESSIONAL ENGINEER

8/25/2023

John Finnegan, PE, LHG
Geotechnical Engineer, Principal

Jason Pritzl, LG
Lead Geologist

Budinger
& Associates

BAI Project Number: S23325

August 25, 2023

## *EMBEDDED TABLES*

## *ATTACHED FIGURES*

# *CONTEXT*

This geotechnical engineering report (GER) presents the results of geotechnical exploration and analysis for the proposed improvements. These services were contracted and coordinated with AHBL, Inc., represented by Craig Andersen, PLA, LEED AP.

## *Project Considerations*

We understand improvements to 2 existing trailheads at the base of Beacon Hill are proposed. The locations include John H. Shields Park and Camp Sekani Park on E. Upriver Drive. Improvements include new paved parking areas and bike paths, stormwater drainage swales, portable restrooms, playground areas, and various interpretive signs and kiosks. Vehicles using the new paved areas will consist primarily of passenger cars and trucks.

## *Location*

Shields Park is in the SE ¼ of the SE ¼ of Section 2 and Sekani Park is in the SW ¼ of the NE ¼ of Section 1, Township 25 North, Range 43 East, Willamette Meridian. Spokane County lists the parcels as 35011.9002 for Sekani Park and 35024.9036 and .0001 for Shields Park; assigned addresses are 6707 and 5625 E. Upriver Drive, respectively. The locations are illustrated in the attached *Vicinity Map* and *Site Plans*.

## *Scope*

This geotechnical study involved interpretation of subsurface conditions to provide conclusions addressing the suitability of the site to support proposed structures and provide geotechnical parameters required for others to design and construct. We endeavored to conduct these services in accordance with generally accepted geotechnical engineering practices as outlined in proposal S23325, dated May 1, 2023.

The following scope was completed:

- Explored subsurface conditions with 5 test borings advanced at Shields Park and 3 test borings advanced at Camp Sekani Park (a total of 8 borings) to a maximum depth of 27 feet.
- Characterized the encountered subsurface conditions;
- Completed laboratory testing on representative soil samples;
- Prepared calculations of stormwater infiltration and pavement section thickness; and,
- Prepared this report presenting the exploration and laboratory results, as well as conclusions and recommendations.

# *ENCOUNTERED CONDITIONS*

## *Physical Setting*

Geologic mapping of the area shows the sites are underlain by Quarternary alluvium (*Qal*) and glacial flood-channel deposits (*Qfcg*). Cretaceous Newman Lake gneiss ($Kog_n$) is mapped along the northern edge of Shields Park. During the last ice age, repeated catastrophic flood events resulting from rupturing of the ice dams that retained Glacial Lake Missoula, inundated much of the Spokane area, and scoured pre-existing rock and sedimentary formations. The floods deposited coarse-grained sediment in consequentially developed and localized channels and ultimately lead to the formation of the Spokane Valley – Rathdrum Prairie (SVRP) aquifer. *Kog$_n$* consists of medium to coarse-grained granitic rock. *Qal* is described as *"silt, sand, and gravel deposits in the present-day stream channels and flood plains… consisting of reworked glacial-flood deposits, loess, and volcanic ash"* (WSDNR, OFR 99-6). *Qfcg* consist of *"thick bedded to massive mixtures of boulders, cobbles, pebbles, and sand."*

## *Surface Conditions*

### Camp Sekani

The ground surface generally sloped down to the south at approximately 12 percent towards the Spokane River; total relief was approximately 30 feet. The site was partially developed with gravel parking areas, a portable restroom shelter, and two small buildings supported by concrete foundations. Undeveloped areas were moderately populated by mature conifers among low-growing grasses and weeds. A seasonal drainage channel orientated north/south was observed to the east of the site and led to an approximately 24-inch-diameter culvert buried beneath Upriver Drive. The channel appeared to be dry during the time field observations were made.

### Shields Park

The site was primarily undeveloped. The ground surface was generally level and void of vegetation in most areas. A large, approximately 6-foot-wide by 3-foot-tall boulder or possible rock outcrop was observed in the east-central portion of the proposed new parking area. Protuberant outcroppings of granitic rock were observed on a south-facing hillside to the north. A small restroom building and paved parking area were observed to the east of the site.

## *Subsurface Conditions*

Conditions encountered in the explorations are described in the *Logs* in accordance with methods described in *Field Exploration*. The subsurface materials were differentiated based on characteristics relevant to this project.

<u>Existing fill</u>

*Existing fill* consisting generally of silty gravel and sand was encountered in Boring 2 (B-2), B-3, B-5, B-6, and B-7 beginning at the ground surface and extending to depths ranging from 2 to 4.5 feet below ground surface (BGS). The condition varied from very loose to dense. Small amounts of brick and plastic debris were observed in *existing fill*.

<u>Fine-grained soil</u>

Sandy lean clay and silt were encountered in B-1 and B-8 beginning at the ground surface and extending to depths of 8 and 4 feet BGS, respectively. B-7 and B-8 encountered silty sand zones at 2 to 6 and 4 to 6.5 feet BGS, respectively. Fines content (percent, by weight, passing the US #200 sieve) ranged from 43 to 65 percent.

<u>Gravel</u>

With the exception of B-4 and B-8, permeable *gravel* with silt, sand, cobbles, and boulders was encountered beginning beneath *fine-grained soil* and *existing fill* and extended to depths greater than 27 feet BGS. The condition was generally medium dense, and the presence of cobbles and boulders tended to interfere with penetration resistance tests resulting in artificially high observed blow counts at some intervals.

<u>Rock</u>

Metamorphosed granitic rock (gneiss) was encountered in B-4 and B-8 beginning at depths of 2 and 6.5 feet BGS. *Rock* was fresh and strong. Uniaxial unconfined compressive strength tests yielded results of 13,642 and 15,083 pounds per square inch (psi) for 2 representative samples tested.

### *Surface and Groundwater Hydrology*

Surface water was not observed on the sites. Surface water was observed in the Spokane River channel to the south of the sites. The surface of the river was approximately 30 feet lower in elevation than the low points of the sites. #

Groundwater was not encountered in the borings. The normal pool elevation of the Spokane River behind Upriver Dam is 1,910 feet. Groundwater is anticipated to begin at elevations beneath the sites that are consistent with the surface elevation of the river at any given point in time.

## *CONCLUSIONS*

*Existing fill* poses a settlement risk to pavements.

*Gravel* is a suitable target soil for subsurface infiltration of stormwater. The aperture of sampling equipment limited the size of soil particles that could be recovered and exclusion of large soil particles (large gravel and cobbles) from representative samples tested may have resulted in slightly overstated fines percentages.

Representative samples of surficial soils were tested for pH, organic content, and CEC. Results are presented in the *Laboratory Summary*. The Spokane Regional Stormwater Manual (SRSM) lists criteria for bio-infiltration swale design in Chapter 6.7.1, Table 6-1. The samples were not composite samples and as such represent values for informational purposes to determine initial suitability. Amendments to the encountered soils may be necessary in order to meet SRSM swale design criteria if reuse of them as treatment soil is desired.

Boulders are present and should be expected to impede excavation and add to earthwork costs.

*Rock* was encountered in strong, fresh condition on the north side of the proposed parking lot at Shields Park and may require heavy ripping, hammering, and/or blasting in order to meet subgrade elevations. Variability should be expected in uniaxial compressive strengths. While tested specimens ranged in strength from 13.6 to 15.1 kips per square inch (ksi), a range of 10 to 28 ksi should be expected throughout the site.

## *RECOMMENDATIONS*

The recommendations presented throughout this chapter are intended to provide economically feasible criteria at normally accepted risk levels. More conservative design parameters can be used if lower risks are preferred. Specifically, the design should incorporate the following recommendations concerning flexible pavement and stormwater drainage.

### *Seismic Considerations*

The recommended seismic site class designation is Site Class D “stiff soil.” Spectral response acceleration parameters, adjusted for Site Class D, were calculated using USGS, U.S. Seismic Design Web Services through the Applied Technology Council website (ATC, 2023). The values of predicted earthquake ground motion for short period structural elements (0.2 second spectral response acceleration, Ss) and for long period structural elements (1.0 second spectral response acceleration, S1) are provided in the table below. The design parameters (SDS and SD1) are equal to ⅔ of the maximum earthquake spectral response accelerations (SMS and SM1).

**Table 1. Seismic design parameters**

| Site | Site Class | Latitude | Longitude | PGA | Ss | $S_1$ | $S_{DS}$ | $S_{D1}$ |
|---|---|---|---|---|---|---|---|---|
| Sekani | D | 47.694 N | -117.314 W | 0.139g | 0.311g | 0.112g | 0.322g | 0.177g |
| Shields | D | 47.687 N | -117.328 W | 0.139g | 0.31g | 0.112g | 0.321g | 0.177g |

*Code Reference: International Building Code (ASCE 7-16)

Due to the lack of shallow groundwater, relatively dense condition of the encountered soils, and low probability of high ground acceleration, the liquefaction triggering potential is considered very low.

## *Earthwork*

**Site preparation.** Strip the ground surface of vegetation and other deleterious items in construction areas only so that mineral soil lacking concentrated organics is exposed in the subgrade.

As stated in *Conclusions*, *existing fill* poses a settlement risk. Risk can likely be mostly mitigated by compacting with a large (16 ton or larger) vibrating, pad-footed roller. Compaction should include scarifying, such as by ripping with a dozer; moisture-conditioning, if necessary, to within approximately 2 percent of maximum dry unit weight; and compacting with a minimum of 6 passes.

**Temporary slopes.** Due to varying construction methods and conditions, temporary cuts should be the responsibility of the contractor. The overburden soils are consistent with Type C materials per WISHA excavation criteria. WISHA specifies a maximum inclination of 1 ½ horizontal to 1 vertical (1 ½ H:1V) in the temporary condition for Type C.

**Permanent slopes.** Maximum permanent cut and fill slope angles of 2H:1V are recommended except where potentially submerged in drainage basins, where the slopes should be no steeper than 3H:1V. Protect completed surfaces as soon as possible with mechanical or bio-technical erosion control. Permanent cuts in *rock* are anticipated to be stable at slope angles of 0.5H:1V. Steeper rock cuts may be feasible provided the rock structure is evaluated by the geotechnical engineer during excavation.

**Preparation of surfaces to receive fill.** Compact surfaces to receive fill to at least 92 percent maximum dry unit weight. Determine maximum dry unit weight and optimum moisture contents for fill material in accordance with the modified Proctor test (ASTM D1557 - MP).

**Protection of subgrade.** Following compaction of subgrade, protect surfaces from degradation during inclement weather. Protection measures include erosion control maintenance, preventing tracking soil and rock offsite, and preventing driving on wet subgrade soil. Reduce frost penetration in freezing weather by leaving surfaces of soil un-compacted if left for an extended duration. Prevent frost penetration in freezing weather by covering soils, such as placing a temporary loose, insulating layer of soil on top.

**Fill material.** *Gravel* and *existing fill* are considered suitable for re-use as structural fill provided that deleterious items (anthropogenic debris, organics, over-sized materials, etc.), if encountered, are removed prior to the re-use. *Fine-grained soil* and *existing fill* encountered in B-2 and B-7 should not be reused as structural fill; they are considered moisture sensitive and will be difficult to work with in wet conditions. If imported fill is needed, a material such as *Gravel Borrow* (WSDOT SS[1] Section 9-03.14(1)) is recommended. Contact us to review alternative material selections.

**Fill Placement.** On sloping ground which will receive fill, bench surfaces to no steeper than 8 percent before placing and compacting structural fill. Place fill in lifts of thickness suited to the compaction equipment but no more than 12 inches. Compact structural fill to at least 92 percent of maximum dry unit weight for footing subgrades; compact to 92 percent of MP also for slabs and pavement subgrades, except within the top 12 inches of final grade where compaction should be increased to 95 percent. Do not place fill in a frozen condition or on un-compacted frozen subgrade.

**Verification and application.** These earthwork recommendations apply to structural fill, backfill against footings, and backfill of utility trenches. Retain a qualified earthwork technician present during fill and backfill operations to observe and test each lift of fill. Frequency of testing should be 2 tests per 2,000 square feet or fraction thereof per lift. A representative of the Geotechnical Engineer is best suited to provide such testing.

## *Flexible Pavement*

Pavement thickness was determined in general accordance with 1993 AASHTO *Guide for Design of Pavement Structures.* Traffic loads and other parameters required for designing pavement sections were not provided. As such, certain assumptions were made in order to determine the recommended pavement section thicknesses. If the parameters listed below are to be altered, we must be contacted to re-evaluate these recommendations.

Pavement design input criteria included the following:

- Design life: 20 years
- Reliability: 75 percent
- Initial serviceability 4.2
- Terminal serviceability: 2.00
- Standard deviation: 0.45
- Subgrade resilient modulus ($M_R$): 5,000 psi
- Average daily traffic (ADT) 340
- Truck percentage 1.5
- Truck Factor (ESALs/truck) 1.6
- Total design ESALs: 29,784

The recommended minimum flexible pavement section for the proposed parking lots is 3 inches of asphalt over 6 inches of base over compacted subgrade. Where pavement subgrade consists of *fine-grained soil*, we recommend the use of a nonwoven geotextile separation fabric between base materials and subgrade soil.

---

[1] Washington State Department of Transportation Standard Specifications

**Table 2: Pavement Compaction and Materials Summary**

| Material | Compaction | Recommended Material Specification |
|---|---|---|
| Asphalt – 3 inches | 92% TM | *WSDOT SS Section 9-03.8(6).* |
| Base – 6 inches | 95% MP | *WSDOT SS Section 9-03.9(3)* |
| Separation geotextile fabric | | *WSDOT SS 9-33.2(1), Table 3* |
| Subgrade, top 12 inches | 95% MP | Encountered soils or embankment fill, improved by compaction |

TM = Theoretical Maximum Unit Weight
MP = Modified Proctor *(AASHTO T-180)*

We recommend grading surfaces so parking lot runoff will be collected and disposed of such that water is not allowed to accumulate near the pavements. We recommend crack maintenance regularly to reduce surface water infiltration. Surface and subgrade drainage are critical to the performance of the pavement section.

These pavement recommendations are made based on the parameters listed above. These recommendations should be considered preliminary until vehicle types and configurations, structural coefficients, ESAL classifications, and future traffic growth rate can be confirmed.

### *Stormwater Drainage*

Gradation analysis was used to estimate permeability based on fines percentages from representative soil samples in accordance with the SRSM*, Appendix 4A – Spokane 200 Method.* The results are summarized in the table below.

**Table 3. Drywell Design Outflow Rate Analysis**

| Exploration ID | Sample Depth (ft) | Fines (%) | Hydraulic Conductivity (in/hr)[1] | Normalized Outflow Rate (cfs/ft)[2] | Safety Factor[3] | Factored Outflow Rate (cfs)[4] | | Infiltration Feasibility |
|---|---|---|---|---|---|---|---|---|
| | | | | | | Single Depth H=6 | Double Depth H=10 | |
| 1 | 10 – 12 | 8.1 | 18 | 0.036 | 2.3 | 0.092 | 0.16 | fair |
| 1 | 15 – 17 | 5.6 | 36 | 0.065 | 1.5 | 0.26 | 0.44 | good |
| 5 | 5 – 7 | 5.5 | 37 | 0.067 | 1.5 | 0.27 | 0.45 | good |
| 5 | 10 – 12 | 3.5 | 86 | 0.14 | 1.3 | 0.65 | 1.1 | excellent |
| 7 | 10 – 12 | 6.6 | 26 | 0.050 | 2.0 | 0.15 | 0.25 | fair |

1. in/hr - inches per hour ($in^3/in^2/hr$)
2. cfs/ft - cubic feet per second per foot
3. Safety Factors from SRSM Table 4A-1
4. cfs - cubic feet per second

We recommend maximum design outflow rates of 0.1 and 0.2 cubic feet per second for single and double-depth drywells, respectively, at Camp Sekani. At Shields Park, 0.3 and 0.5 cubic feet per second are the recommended outflow rates for single and double-depth drywells, respectively. If higher outflow rates are desirable, we recommend full-scale testing of new drywells to determine more accurate design outflow rates.

Infiltration structures can "silt-up" over time and operation and maintenance guidelines in the stormwater manual should be followed. We recommend setting aside sufficient area for eventual replacement.

### *Additional Services*

Effective geotechnical services involve cooperation with the owner, designer, and constructor as follows:

1. Preliminary study to assist in planning and to economically adapt the project to its geologic environment;
2. Soil exploration and analysis to characterize subsurface conditions and recommend design criteria;
3. Consultation with the designer to adapt the specific design to the site in accordance with the recommendations;
4. Construction observation to verify the conditions encountered and to make recommendations for modifications, as necessary; and,
5. Construction material testing, quality control, and special inspection.

This report satisfies Item 2 of the 5-phase endeavor. We are eager to provide assistance with design and construction as appropriate to assist in completing a safe and economical project.

## *FIELD EXPLORATION*

The fieldwork was conducted by lead geologist Jason Pritzl, LG, staff geologist Josh Hudgins, GIT, and supervised by geotechnical engineer John Finnegan, PE, beginning July 25th and concluding July 26th, 2023. The field activities generally consisted of the following:

- Reconnaissance of the sites and surrounding areas;
- Logging subsurface conditions in 8 test borings
- Advancing 3 Kessler® DCP soundings; and,
- Obtaining split-spoon samples of the encountered soils.

Results are presented in *Figures*.

### *Test Borings*

Borings were advanced with a track-mounted Geoprobe 7822 drill rig equipped with an automatic standard penetration test (SPT) hammer utilizing a 4.5-inch outside diameter air-rotary overburden system.

### *Soil Samples*

Samples were obtained by driving split-spoon samplers through the end of the temporary drill casing.

**Standard Penetration Tests (SPTs) - ASTM D 1586.** SPTs were conducted by driving a 2-inch outside diameter split-spoon sampler with a hydraulicly operated automatic drop hammer using a 140-pound driving mass which free falls 30 inches. The resulting blow count for each foot of sampler advancement, represents an uncorrected N-value, that is presented in the *Boring Logs*. The energy ratio (ER) is much higher with the automatic drop hammer compared to the reference cathead/rope system.

**3-Inch Split Spoon Samples (3" SS) - ASTM D 3550.** Split-spoon samples were obtained with a 3.0-inch outside by 2.4-inch inside diameter split-spoon sampler similar to the SPT sampler. Blow counts with the 3" SS do not represent SPT N-values since the end area of the 3" SS is approximately twice that of the standard sampler. A correction factor of 0.56 was applied to blow counts to estimate the representative SPT N-value presented in the *Boring Logs*.

## *DCP Testing*

**Kessler® DCP Testing – ASTM D6951.** Soil strength was estimated with a series of DCP tests using a Kessler® DCP which consists of a 10.1-pound slide hammer and rods with 2-inch graduations. The hammer is manually lifted and allowed to fall from a fixed height. DCP test results can be correlated to CBR values for estimating relative soil strength for pavement design. The results of DCP penetration per 1-inch intervals are presented in *Figures.*

## *Soil and Rock Classification*

Field descriptions of soils and rock were completed in accordance with the current version of the Washington State Department of Transportation, *Geotechnical Design Manual* (GDM), M 46-03, except that fines (silt and clay) were described in accordance with ASTM D 2487. *Whereas, the GDM uses the terms 'silty' and 'clayey' to describe a very broad range of fines from 10 to 49 percent; ASTM D 2487 uses those terms for percentages greater than 12 and the term 'with' for fines ranging from 5 to 12 percent, which is typically necessary to describe variations relevant to soil permeability per the SRSM.* A key to the descriptions is provided in *Guide to Soil and Rock Descriptions.*

## *Location*

**Horizontal & vertical control.** The *Site Plan* was reproduced from plans provided by the client and is based on measured offsets from existing site features at the time of exploration.

Elevations presented in the *Boring Logs* were correlated from contour lines illustrated on plans provided by the client. Horizontal and vertical locations can be considered accurate to within 5-foot and 1-foot respectively, relative to the information provided.

# *LABORATORY ANALYSIS*

Laboratory testing was performed on representative samples of the soils encountered to provide data used in our assessment of soil characteristics.

Tests were conducted, where practical, in accordance with nationally recognized standards (ASTM, AASHTO, etc.), which are intended to model in-situ soil conditions and behavior. The results are presented in *Figures*.

### *Index Parameters*

**Moisture content – ASTM D2216.** Moisture contents were determined by direct weight proportion (weight of water/weight of dry soil) determined by drying soil samples in an oven until reaching constant weight.

**Gradation – ASTM D6913.** Gradation analysis was performed by the mechanical sieve method. The mechanical sieve method is utilized to determine particle size distribution based upon the dry weight of sample passing through sieves of varying mesh sizes. The results of gradation are provided in *Grain Size Distribution Results*.

### *Chemical Parameters*

**pH – AASHTO T289.** The quantified measurement of soil pH (acidity = pH <7) and minimum resistivity are useful variables in determining the potential corrosivity of the soil. Certain clayey soils exhibit excess acidity that attacks concrete, iron, and buried utilities.

**Cation Exchange Capacity (CEC) – EPA 9081.** Method 9081 is applicable to most soils, including calcareous and non-calcareous soils. The method of determining cation-exchange capacity by summation should be employed for distinctly acid soils. The soil sample is mixed with an excess of sodium acetate solution, resulting in an exchange of the added sodium cations for the matrix cations. The concentration of displaced sodium is then determined by atomic absorption, emission spectroscopy, or an equivalent means. The results are presented as milliequivalents per 100 grams (meq/100g).

**Organic Content – ASTM D2974.** Organic content is determined by measuring weight loss after subjecting an appropriate mass of soil to burning off organic matter in an ignition muffle furnace. The loss is recorded as a percentage of the dry soil content.

## *LIMITATIONS*

The conclusions and recommendations presented herein are based upon the results of field explorations and laboratory testing results. They are predicated upon our understanding of the project, its design, and its location as defined by the client. We endeavored to conduct this study in accordance with generally accepted geotechnical engineering practices in this area.

This GER presents our professional interpretation of exploration data developed, which we believe meet the standards of the geotechnical profession in this area; we make no other warranties, express or implied. Attached is a document titled "*Important Information About Your Geotechnical Engineering Report,*" which we recommend you review carefully to better understand the context within which these services were completed.

Unless test locations are specified by others or limited by accessibility, the scope of analysis is intended to develop data from a representative portion of the site. However, the areas tested are discreet. Interpolation between these discreet locations is made for illustrative purposes only but should be expected to vary. If a greater level of detail is desired, the client should request an increased scope of exploration.

## *REFERENCES*

AASHTO, 1993, Guide for Design of Pavement Structures.

American Society of Civil Engineers, 2017, ASCE Standard 7-16.

ASTM International, 2011, Standard Practice for Classification of Soils for Engineering Purposes, D 2487-11.

City of Spokane, 2020, Map Spokane, https://spokane.maps.arcgis.com/apps/webappviewer/

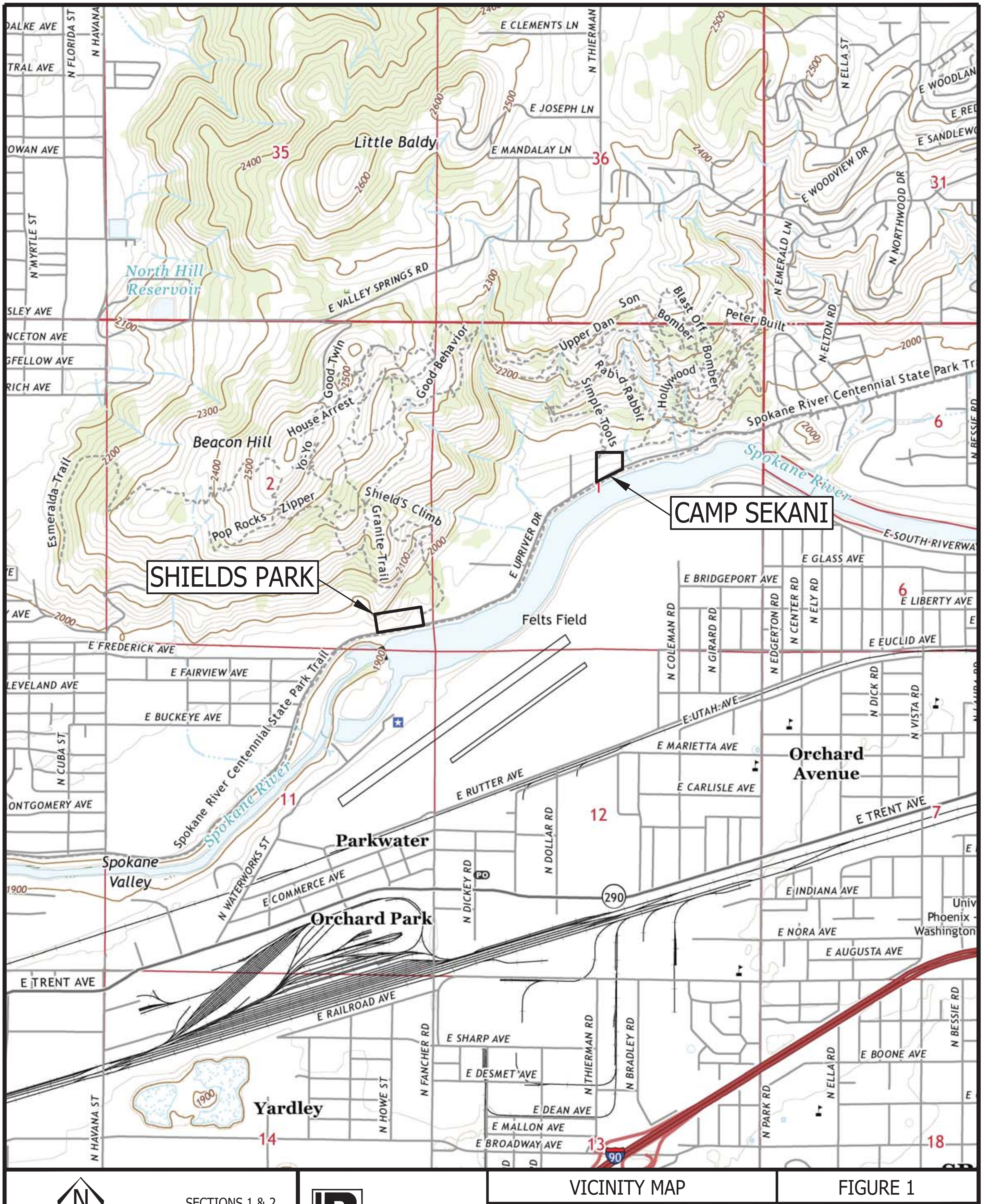
Spokane Regional Stormwater Manual (SRSM), 2008.

Washington State Department of Ecology, Stormwater Management Manual for Eastern Washington, February 2019.

Washington State Department of Natural Resources, (WSDNR), Preliminary Geologic Map of the Spokane NE 7.5-Minute Quadrangle, Spokane County, Washington, OFR 99-6.

Washington State Department of Transportation, 2022, Geotechnical Design Manual (WSDOT GDM).

Washington State Department of Transportation, 2023, Standard Specifications for Road, Bridge, and Municipal Construction (WSDOT SS).



N

SCALE: 1"=2000'

0 1000 2000

SECTIONS 1 & 2
T 25 N R 43 E
USGS 2020

Budinger
& Associates

VICINITY MAP

MAKE BEACON HILL PUBLIC - PHASE 2
SHIELDS & CAMP SEKANI PARKS
SPOKANE, WASHINGTON

FIGURE 1

PROJECT NUMBER S23325

DATE: 7/2023

◐ TEST BORING LOCATION (B-1)

N
SCALE: 1"=80'
0 40 80

Budinger
& Associates

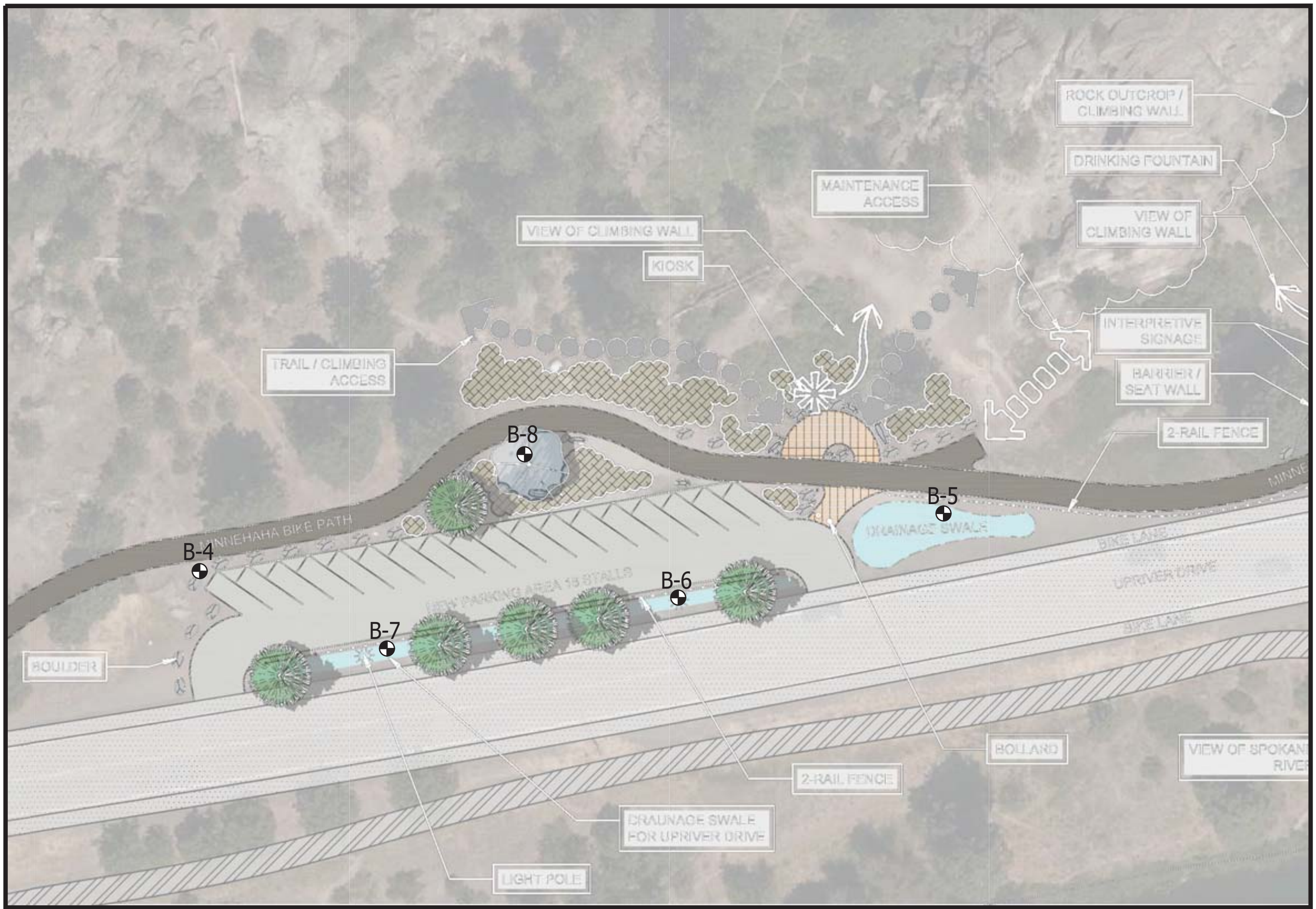
SITE PLAN

MAKE BEACON HILL PUBLIC - PHASE 2
CAMP SEKANI PARK
SPOKANE, WASHINGTON

FIGURE 2-1

PROJECT NUMBER S23325

DATE: 7/2023



● TEST BORING LOCATION (B-4)

N
SCALE: 1"=50'
0 25 50

Budinger
& Associates

## SITE PLAN

MAKE BEACON HILL PUBLIC - PHASE 2
SHIELDS PARK
SPOKANE, WASHINGTON

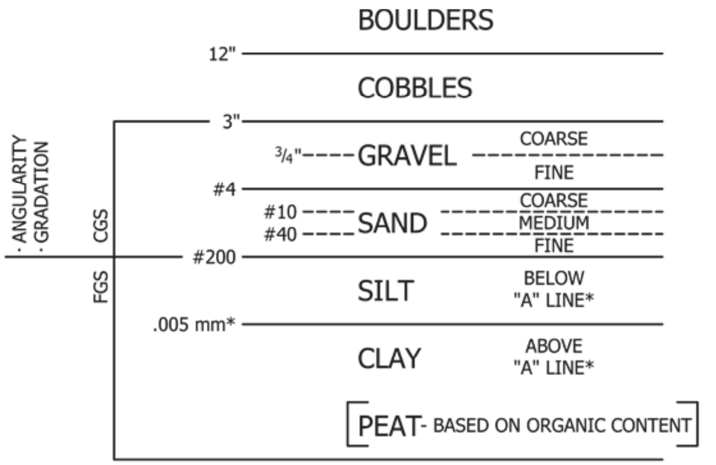
FIGURE 2-2

PROJECT NUMBER S23325

DATE: 7/2023

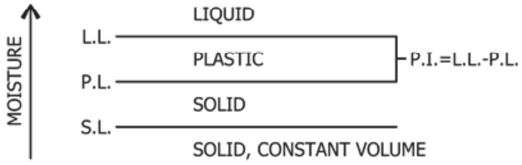
# GUIDE TO SOIL & ROCK DESCRIPTIONS

## SOIL CLASSIFICATION

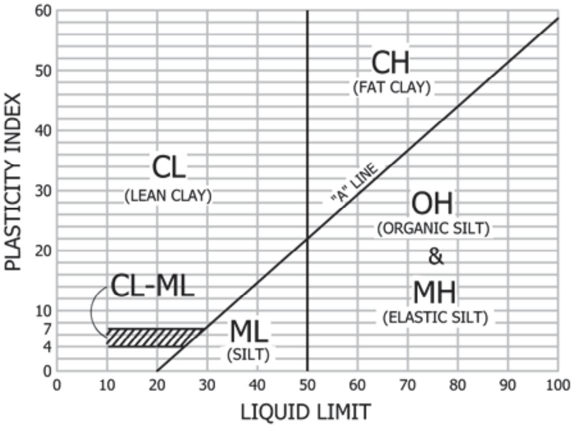


** SEE PLASTICITY CHART*
*CGS - COARSE GRAINED SOIL - MORE THAN 50% RETAINED ON A #200 SIEVE*
*FGS - FINE GRAINED SOIL - 50% MORE PASSES, #200 SIEVE*
*FINES - PORTION FINER THAN #200 SIEVE*

## ATTERBERG LIMITS



## PLASTICITY CHART



*NOTE - CHART APPLIES TO FGS AND MINUS #40 SIEVE FRACTION OF CGS*

## GUIDE TO SOIL DESCRIPTION MODIFIERS, MOISTURE, AND CONDITION PRESENTED ON LOGS

| MODIFIER | ESTIMATED PERCENTAGE OF MATERIAL |
|---|---|
| SUFFIX "LY" OR "Y" | 30% OR MORE FOR COARSE PARTS IN FGS<br>GREATER THAN 12% FOR FINES IN CGS |
| WITH | 15% - 29% FOR COARSE PARTS IN FGS<br>5% - 12% FOR FINES IN CGS |

*NOTE - VISUAL ESTIMATES OF MATERIAL PERCENTAGES TYPICALLY VARY 0 TO 10% FROM THOSE DETERMINED BY LABORATORY TESTING.*

MOISTURE

- DRY
- MOIST
- SATURATED OR WET

SOIL CONDITION

- CGS:
  - VERY LOOSE
  - LOOSE
  - MEDIUM DENSE
  - DENSE
  - VERY DENSE
- FGS:
  - VERY SOFT
  - SOFT
  - MEDIUM STIFF
  - STIFF
  - VERY STIFF
  - HARD

SAMPLES

| | |
|---|---|
| ◩ | STANDARD 2" PENETRATION TEST SAMPLER WITH BLOWS PER FOOT |
| ■ | 3" SPLIT SPOON SAMPLER WITH BLOWS PER FOOT |
| □ | DRILL CUTTING SAMPLE |
| ▨ | BULK SAMPLE |
| ⧖ | THIN-WALLED TUBE SAMPLE |
| I | DIAMOND CORE RUN WITH % RECOVERY & ROCK QUALITY DESIGNATION |
| ⊠ | 2.5" SPLIT SPOON SAMPLER WITH BLOWS PER FOOT |
| ▨ | CONTINUOUS SOIL SAMPLE |
| R | REFUSAL OF SAMPLE (50+ BLOWS PER 6") |

ROCK WEATHERING

- FRESH
- SLIGHTLY WEATHERED
- MODERATELY WEATHERED
- HIGHLY WEATHERED
- COMPLETELY WEATHERED
- RESIDUAL SOIL

ROCK CONDITION

- EXTREMELY WEAK
- VERY WEAK
- MODERATELY WEAK
- MODERATELY STRONG
- STRONG
- VERY STRONG

Budinger
& Associates

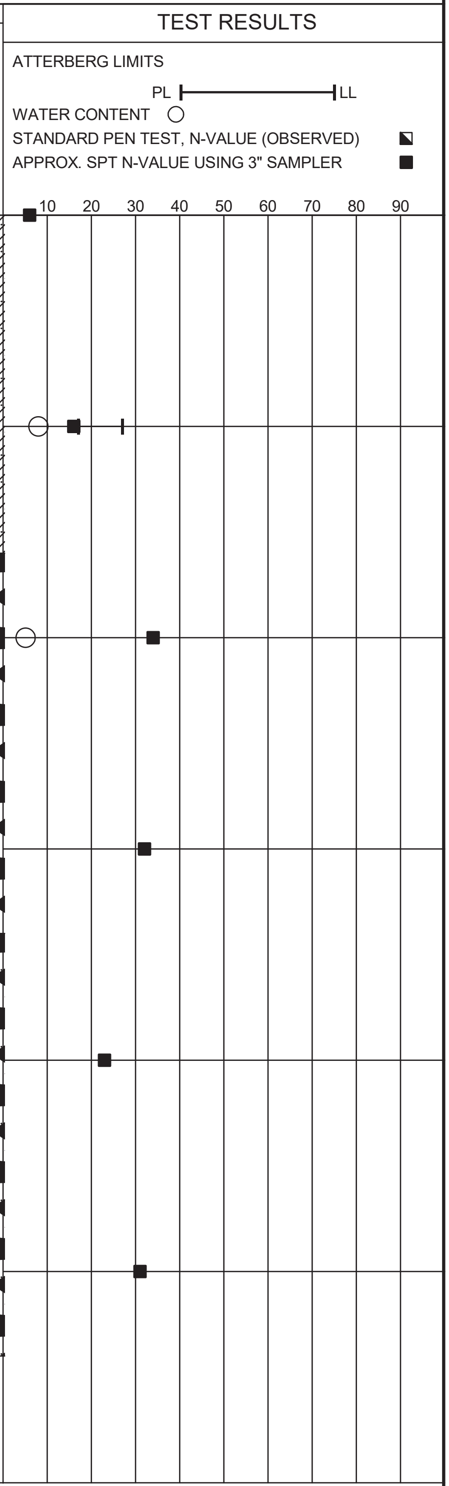
FIGURE 3

# TEST BORING 1

**Date of Boring:** 7-25-23
**Driller:** Budinger & Assoc., Inc.
**Type of Drill:** Geoprobe 7822DT Drill, automatic SPT hammer
**Location:** South side of site - north side of central proposed swale
**Surface:** duff and topsoil

**Elevation:** 1939 ft
**Logged by:** J. Pritzl
**Size of hole:** air rotary overburden system, 4.5 in O.D. casing

| DEPTH | SAMPLES | RQD, SPT N (% RECOVERY) (Blows per 6") | MOISTURE, COLOR, CONDITION | DESCRIPTION | SOIL LOG |
|---|---|---|---|---|---|
| 0 | ▮ | 6 (83%) | dry, light brown, medium stiff | SANDY LEAN CLAY with organics, small rootlets | ▨ |
| 5 | ▮ | 16 (63%) | very stiff | | |
| | | | dry, olive brown, medium dense to dense | GRAVEL with Silt, Sand, Cobbles and Boulders, coarse to fine, subangular to subrounded, micaceous, stratified with sandy zones | |
| 10 | ▮ | 34 (83%) | | | |
| 15 | ▮ | 32 (88%) | | | |
| | | | dry, olive brown, medium dense to dense | GRAVEL with Sand, Cobbles and Boulders, coarse to fine, subangular to subrounded, micaceous, stratified with sandy zones | |
| 20 | ▮ | 23 (79%) | | | |
| 25 | ▮ | 31 (79%) | | | |
| | | | no free groundwater observed | End of Boring @ 27 ft | |
| 30 | | | | | |



LOGS WITHOUT WELL WITH TESTS S23325.GPJ GINT STD US.GDT 8/18/23

Budinger
& Associates
1101 North Fancher Road
Spokane Valley, WA 99212

**BORING LOGS** **FIGURE 4-1**

Project: Make Beacon Hill Public - Phase 2

Location: Spokane Valley, WA

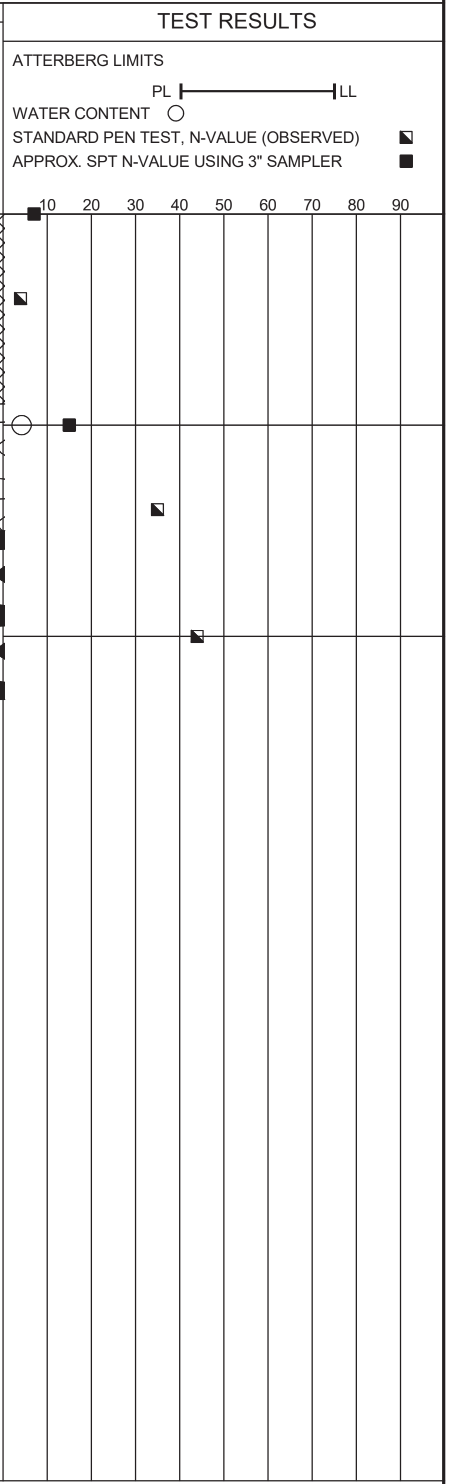
Number: S23325

# TEST BORING 2

**Date of Boring:** 7-25-23
**Driller:** Budinger & Assoc., Inc.
**Type of Drill:** Geoprobe 7822DT Drill, automatic SPT hammer
**Location:** Northeast proposed parking area
**Surface:** bare

**Elevation:** 1945 ft
**Logged by:** J. Pritzl
**Size of hole:** air rotary overburden system, 4.5 in O.D. casing

| DEPTH | SAMPLES | RQD, SPT N (% RECOVERY) (Blows per 6") | MOISTURE, COLOR, CONDITION | DESCRIPTION |
|---|---|---|---|---|
| 0 | ■ | 7 (42%) | dry, light brown, loose to very loose | SILT with Sand and Gravel, occasional Cobbles (existing fill) |
|  | ◩ | 4 *(4-2-2-6)* (58%) |  |  |
| 5 |  |  | dry, moderate brown, medium dense | SILTY GRAVEL with Sand and Cobbles, coarse to fine, subangular to subrounded |
|  | ■ | 15 (54%) |  |  |
|  | ◩ | 35 *(5-15-20)* (28%) | dry, grayish brown, dense | GRAVEL with Silt, Sand, Cobbles and Boulders, coarse to fine, subangular to subrounded, micaceous |
| 10 | ◩ | 44 *(17-23-21)* (66%) |  |  |
|  |  |  | no free groundwater observed | End of Boring @ 11.5 ft |
| 15 |  |  |  |  |
| 20 |  |  |  |  |
| 25 |  |  |  |  |
| 30 |  |  |  |  |



Budinger
& Associates
1101 North Fancher Road
Spokane Valley, WA 99212

## BORING LOGS

## FIGURE 4-2

Project: Make Beacon Hill Public - Phase 2

Location: Spokane Valley, WA

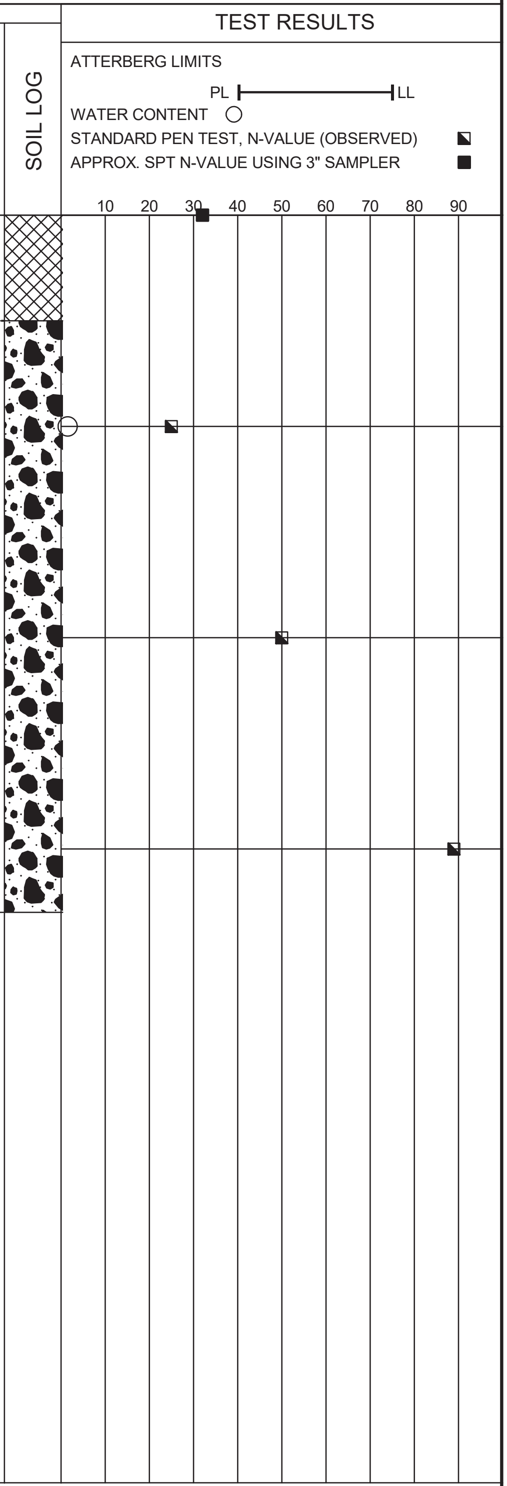
Number: S23325

# TEST BORING 3

**Date of Boring:** 7-25-23
**Driller:** Budinger & Assoc., Inc.
**Type of Drill:** Geoprobe 7822DT Drill, automatic SPT hammer
**Location:** Northwest proposed parking area
**Surface:** gravel

**Elevation:** 1944 ft
**Logged by:** J. Pritzl
**Size of hole:** air rotary overburden system, 4.5 in O.D. casing

| DEPTH | SAMPLES | RQD, SPT N (% RECOVERY) (Blows per 6") | MOISTURE, COLOR, CONDITION | DESCRIPTION |
|---|---|---|---|---|
| 0 | ■ | 32 (89%) | dry, moderate brown, dense | SILTY GRAVEL with Sand, Cobbles and Boulders, coarse to fine, subangular to subrounded (existing fill) |
| | | | dry, grayish brown, medium dense to dense | GRAVEL with Sand, Cobbles and Boulders, coarse to fine, subangular to subrounded, micaceous |
| 5 | ◪ | 25 *(11-12-13)* (56%) | | |
| 10 | ◪ | 50 *(21-22-38)* (56%) | | |
| 15 | ◪ | 89 *(23-39-50)* (66%) | | |
| | | | no free groundwater observed | End of Boring @ 16.5 ft |
| 20 | | | | |
| 25 | | | | |
| 30 | | | | |



**Budinger & Associates**
1101 North Fancher Road
Spokane Valley, WA 99212

## BORING LOGS

## FIGURE 4-3

Project: Make Beacon Hill Public - Phase 2
Location: Spokane Valley, WA
Number: S23325

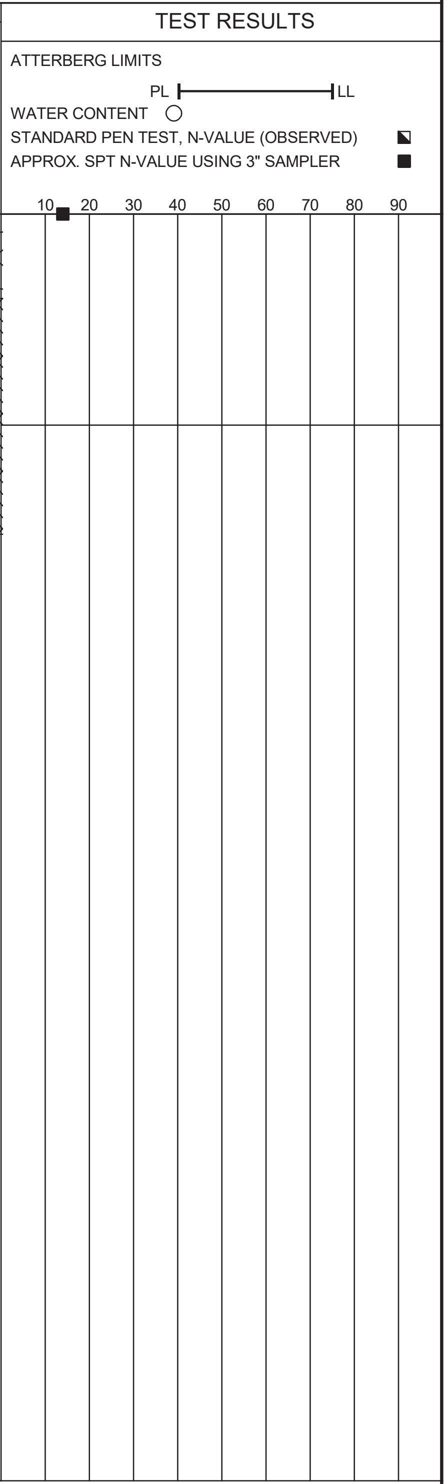
LOGS WITHOUT WELL WITH TESTS S23325.GPJ GINT STD US.GDT 8/18/23

# TEST BORING 4

**Date of Boring:** 7-25-23
**Driller:** Budinger & Assoc., Inc.
**Type of Drill:** Geoprobe 7822DT Drill, automatic SPT hammer
**Location:** Northwest corner of proposed new parking area
**Surface:** bare

**Elevation:** 1947 ft
**Logged by:** J. Pritzl
**Size of hole:** air rotary overburden system, 4.5 in O.D. casing

| DEPTH | SAMPLES | RQD, SPT N | (% RECOVERY) (Blows per 6") | MOISTURE, COLOR, CONDITION | DESCRIPTION |
|---|---|---|---|---|---|
| 0 | | 14 | (56%) | dry, dark to moderate brown, medium dense | SILTY GRAVEL with Sand, Cobbles and Boulders, coarse, angular to subangular |
| | | 100 | (100%) | light to moderate gray | Gneiss, coarse grained (pegmatitic), fresh, strong to very strong rock (R4 to R5), widely spaced discontinuities in good condition<br>Fracture Frequency = 1 |
| 5 | | 100 | (100%) | | Fracture Frequency = 0 |
| | | | | no free groundwater observed | End of Boring @ 7.58 ft |
| 10 | | | | | |
| 15 | | | | | |
| 20 | | | | | |
| 25 | | | | | |
| 30 | | | | | |



**Budinger & Associates**
1101 North Fancher Road
Spokane Valley, WA 99212

## BORING LOGS

## FIGURE 4-4

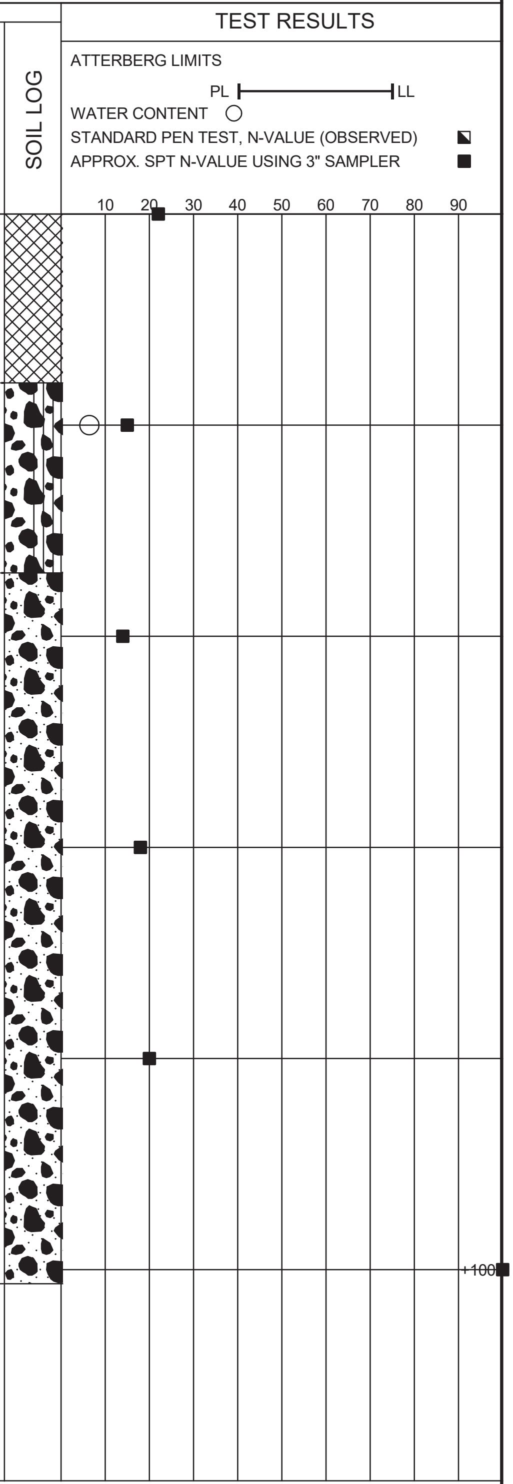
Project: Make Beacon Hill Public - Phase 2
Location: Spokane Valley, WA
Number: S23325

# TEST BORING 5

**Date of Boring:** 7-26-23
**Driller:** Budinger & Assoc., Inc.
**Type of Drill:** Geoprobe 7822DT Drill, automatic SPT hammer
**Location:** Eastern proposed drainage swale
**Surface:** gravel

**Elevation:** 1939 ft
**Logged by:** J. Pritzl
**Size of hole:** air rotary overburden system, 4.5 in O.D. casing

| DEPTH | SAMPLES | RQD, SPT N | (% RECOVERY) (Blows per 6") | MOISTURE, COLOR, CONDITION | DESCRIPTION |
|---|---|---|---|---|---|
| 0 | ▮ | 22 | (67%) | dry, dark brown, medium dense | SILTY GRAVEL with Sand and Cobbles, coarse to fine, angular to subrounded, trace of brick debris (existing fill) |
| 5 | ▮ | 15 | (79%) | dry, olive brown, medium dense | GRAVEL with Silt, Sand, Cobbles and Boulders, coarse to fine, subangular to subrounded, micaceous, stratified with sandy zones |
| 10 | ▮ | 14 | (79%) | dry, olive brown, medium dense | GRAVEL with Sand, Cobbles and Boulders, coarse to fine, subangular to subrounded, micaceous, stratified with sandy zones |
| 15 | ▮ | 18 | (75%) | | |
| 20 | ▮ | 20 | (63%) | | |
| 25 | ▮ | R<br>*(65/4")* | (0%) | no free groundwater observed | End of Boring @ 25.33 ft |
| 30 | | | | | |



LOGS WITHOUT WELL WITH TESTS S23325.GPJ GINT STD US.GDT 8/18/23

Budinger
& Associates
1101 North Fancher Road
Spokane Valley, WA 99212

## BORING LOGS

## FIGURE 4-5

Project: Make Beacon Hill Public - Phase 2

Location: Spokane Valley, WA

Number: S23325

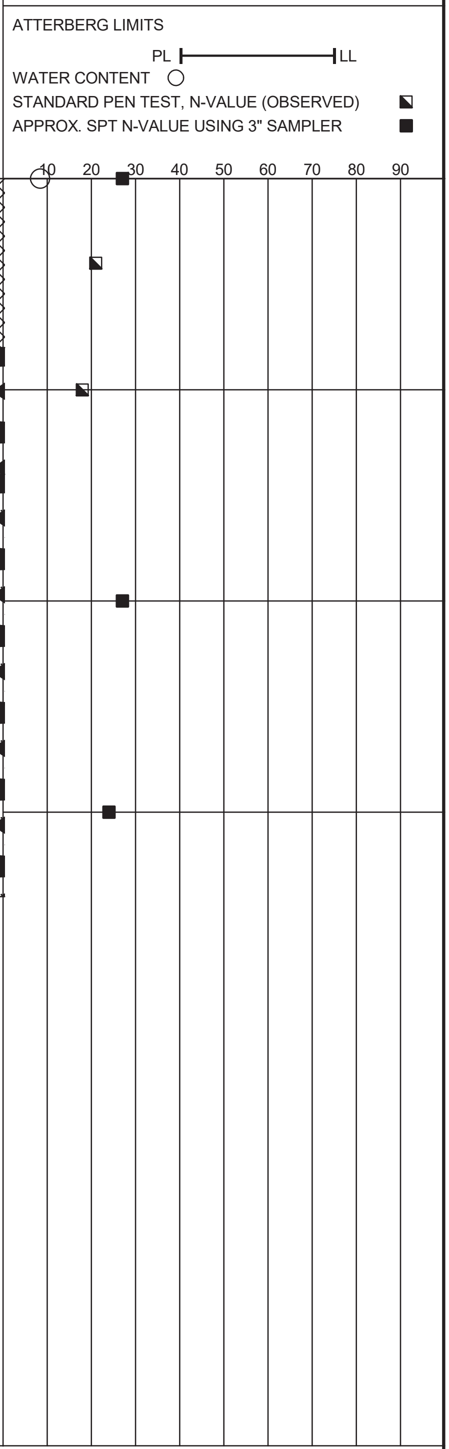
# TEST BORING 6

**Date of Boring:** 7-26-23
**Driller:** Budinger & Assoc., Inc.
**Type of Drill:** Geoprobe 7822DT Drill, automatic SPT hammer
**Location:** East end of proposed street-side drainage swale
**Surface:** gravel

**Elevation:** 1940 ft
**Logged by:** J. Pritzl
**Size of hole:** air rotary overburden system, 4.5 in O.D. casing

| DEPTH | SAMPLES | RQD, SPT N (% RECOVERY) (Blows per 6") | MOISTURE, COLOR, CONDITION | DESCRIPTION | SOIL LOG |
|---|---|---|---|---|---|
| 0 | ▮ | 27 (75%) | dry, dark brown, medium dense to dense | SILTY GRAVEL with Sand and Cobbles, coarse to fine, angular to subrounded, some brick debris (existing fill) | ▨ |
|  | ◪ | 21 (10-9-12) (0%) |  |  |  |
| 5 | ◪ | 18 (14-10-8) (56%) | dry, olive brown, medium dense | GRAVEL with Silt, Sand, Cobbles and Boulders, coarse to fine, subangular to subrounded, micaceous, stratified with sandy zones |  |
|  |  |  | dry, olive brown, medium dense to dense | GRAVEL with Sand, Cobbles and Boulders, coarse to fine, subangular to subrounded, micaceous, stratified with sandy zones<br>(poor air return to surface from 8 to 11.5 feet) |  |
| 10 | ▮ | 27 (75%) |  |  |  |
| 15 |  |  |  | (poor air return to surface from 14 to 15 feet) |  |
|  | ▮ | 24 (78%) |  |  |  |
|  |  |  | no free groundwater observed | End of Boring @ 17 ft |  |
| 20 |  |  |  |  |  |
| 25 |  |  |  |  |  |
| 30 |  |  |  |  |  |

TEST RESULTS



Budinger
& Associates
1101 North Fancher Road
Spokane Valley, WA 99212

## BORING LOGS

## FIGURE 4-6

Project: Make Beacon Hill Public - Phase 2

Location: Spokane Valley, WA

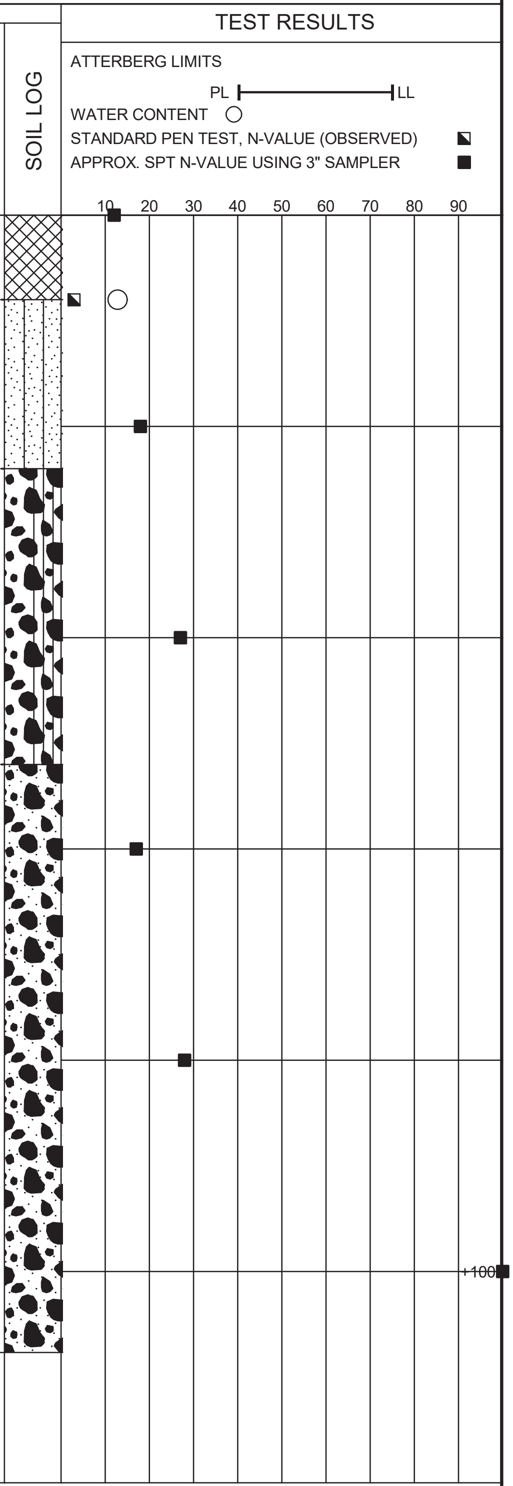
Number: S23325

# TEST BORING 7

**Date of Boring:** 7-26-23
**Driller:** Budinger & Assoc., Inc.
**Type of Drill:** Geoprobe 7822DT Drill, automatic SPT hammer
**Location:** West end of proposed street-side drainage swale
**Surface:** gravel

**Elevation:** 1940 ft
**Logged by:** J. Pritzl
**Size of hole:** air rotary overburden system, 4.5 in O.D. casing

| DEPTH | SAMPLES | RQD, SPT N | (% RECOVERY) (Blows per 6") | MOISTURE, COLOR, CONDITION | DESCRIPTION |
|---|---|---|---|---|---|
| 0 | ■ | 12 | (67%) | dry, dark brown, medium dense | SILTY SAND with Gravel, some plastic debris (existing fill) |
|  | ◪ | 3<br>(2-1-2-3) | (54%) | dry, moderate brown, very loose | SILTY SAND with Gravel, medium to fine, slightly micaceous |
| 5 | ■ | 18 | (71%) |  |  |
|  |  |  |  | dry, olive brown, medium dense to dense | GRAVEL with Silt, Sand, Cobbles and Boulders, coarse to fine, subangular to subrounded, micaceous, stratified with sandy zones |
| 10 | ■ | 27 | (92%) |  |  |
|  |  |  |  | dry, olive brown, medium dense to dense | GRAVEL with Sand, Cobbles and Boulders, coarse to fine, subangular to subrounded, micaceous, stratified with sandy zones |
| 15 | ■ | 17 | (79%) |  |  |
| 20 | ■ | 28 | (83%) |  |  |
| 25 | ■ | R | (78%) |  |  |
|  |  |  |  | no free groundwater observed | End of Boring @ 26.92 ft |
| 30 |  |  |  |  |  |



## BORING LOGS

## FIGURE 4-7

Project: Make Beacon Hill Public - Phase 2
Location: Spokane Valley, WA
Number: S23325

Budinger
& Associates
1101 North Fancher Road
Spokane Valley, WA 99212

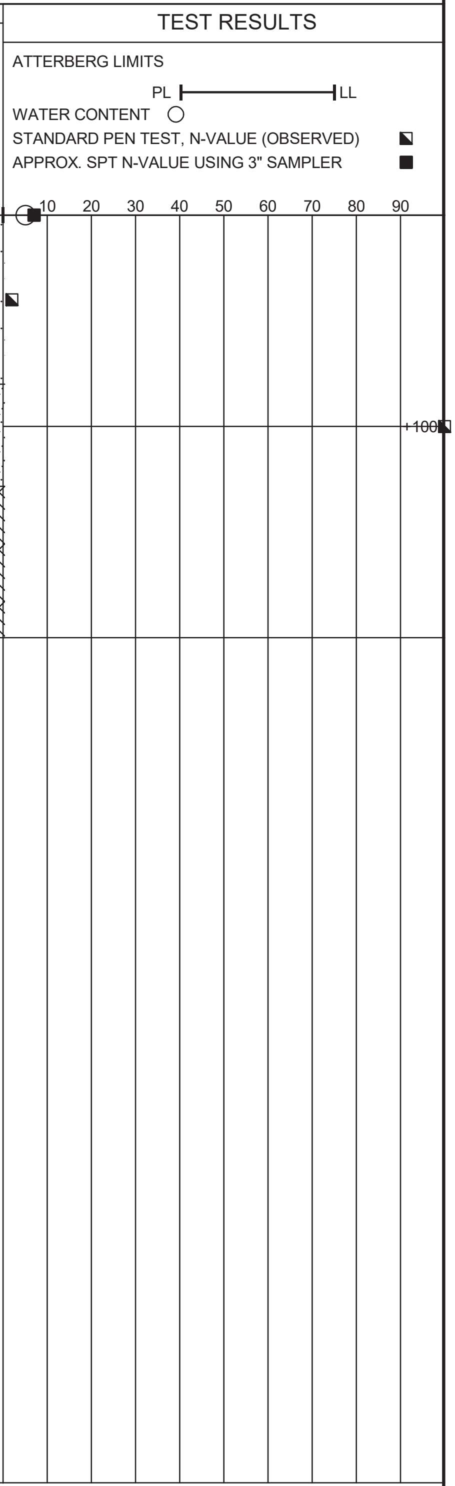
# TEST BORING 8

**Date of Boring:** 7-26-23
**Driller:** Budinger & Assoc., Inc.
**Type of Drill:** Geoprobe 7822DT Drill, automatic SPT hammer
**Location:** North side of proposed new parking area
**Surface:** bare

**Elevation:** 1945 ft
**Logged by:** J. Pritzl
**Size of hole:** air rotary overburden system, 4.5 in O.D. casing

| DEPTH | SAMPLES | RQD, SPT N (% RECOVERY) (Blows per 6") | MOISTURE, COLOR, CONDITION | DESCRIPTION | SOIL LOG |
|---|---|---|---|---|---|
| 0 | | 7 (71%) | dry, light brown, loose to very loose | SANDY SILT with small rootlets | |
| | | 2 *(1-1-1-5)* (67%) | | | |
| 5 | | R *(4-5-50/5")* (76%) | dry, light to moderate brown, loose | SILTY SAND, slightly micaceous | |
| | | | light to moderate gray | Gneiss, coarse grained (pegmatitic), fresh, strong to very strong rock (R4 to R5) | |
| 10 | | | no free groundwater observed | End of Boring @ 10 ft | |
| 15 | | | | | |
| 20 | | | | | |
| 25 | | | | | |
| 30 | | | | | |

## TEST RESULTS



# BORING LOGS

# FIGURE 4-8

Project: Make Beacon Hill Public - Phase 2

Location: Spokane Valley, WA

Number: S23325

Budinger
& Associates
1101 North Fancher Road
Spokane Valley, WA 99212

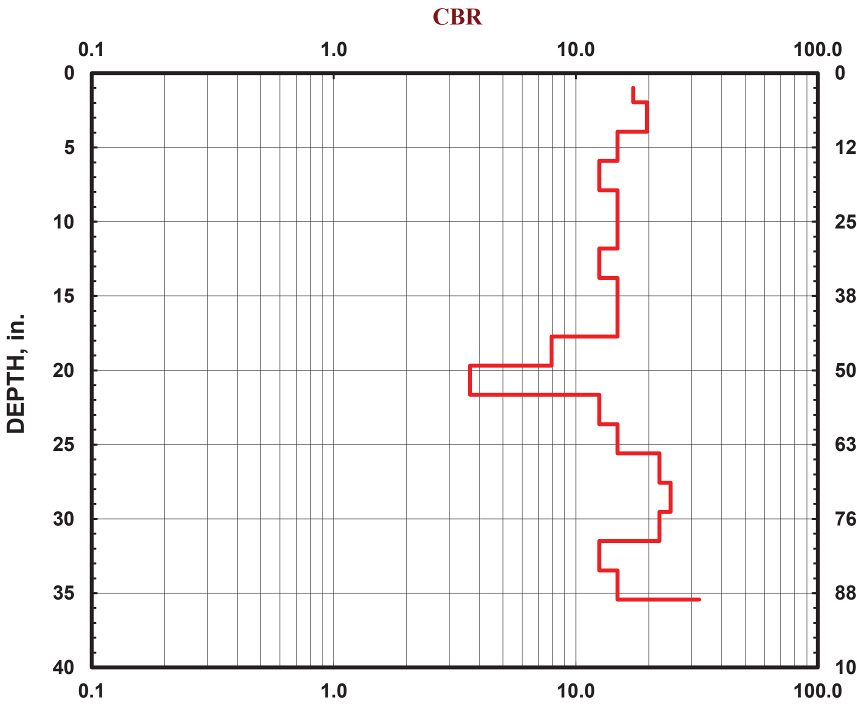
# DCP TEST DATA

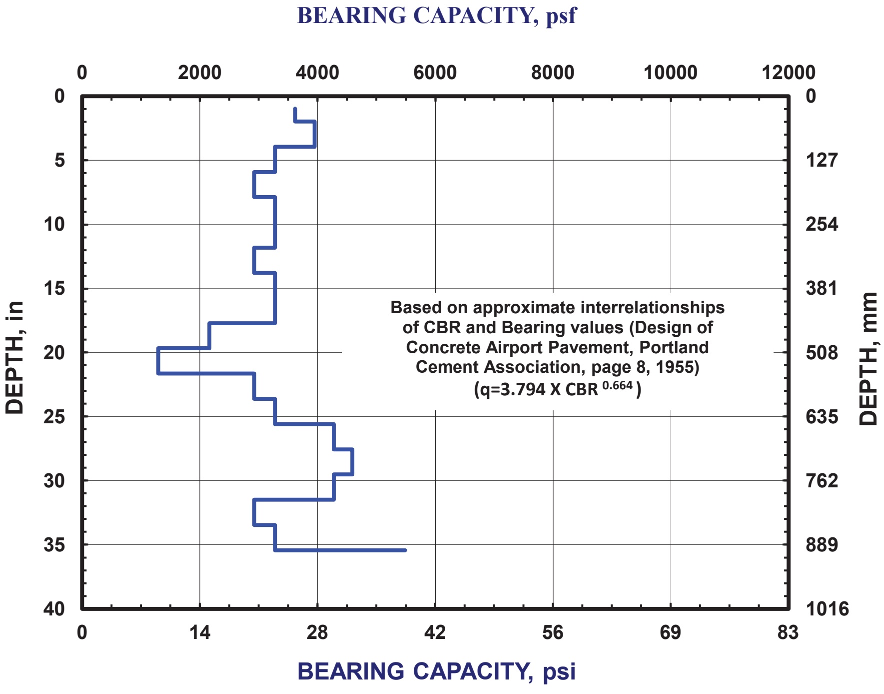
**Project:** *Make Beacon Hill Public - Phase 2*
**Date:** 25-Jul-23
**Location:** B-2
**Soil Type(s):** Silt

Hammer
● 10.1 lbs.
○ 17.6 lbs.
○ Both hammers used

Soil Type
○ CH
○ CL
● All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 4 | 25 | 2 |
| 4 | 50 | 2 |
| 4.5 | 75 | 2 |
| 4.5 | 100 | 2 |
| 3.5 | 125 | 2 |
| 3.5 | 150 | 2 |
| 3 | 175 | 2 |
| 3 | 200 | 2 |
| 3.5 | 225 | 2 |
| 3.5 | 250 | 2 |
| 3.5 | 275 | 2 |
| 3.5 | 300 | 2 |
| 3 | 325 | 2 |
| 3 | 350 | 2 |
| 3.5 | 375 | 2 |
| 3.5 | 400 | 2 |
| 3.5 | 425 | 2 |
| 3.5 | 450 | 2 |
| 2 | 475 | 2 |
| 2 | 500 | 2 |
| 1 | 525 | 2 |
| 1 | 550 | 2 |
| 3 | 575 | 2 |
| 3 | 600 | 2 |
| 3.5 | 625 | 2 |
| 3.5 | 650 | 2 |
| 5 | 675 | 2 |
| 5 | 700 | 2 |
| 5.5 | 725 | 2 |
| 5.5 | 750 | 2 |
| 5 | 775 | 2 |
| 5 | 800 | 2 |
| 3 | 825 | 2 |
| 3 | 850 | 2 |
| 3.5 | 875 | 2 |
| 3.5 | 900 | 2 |
|  |  |  |
|  |  |  |
|  |  |  |
|  |  |  |
|  |  |  |
|  |  |  |
|  |  |  |


CBR


BEARING CAPACITY, psf

Based on approximate interrelationships of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955) ($q=3.794 \times CBR^{0.664}$)

**Figure 5-1**

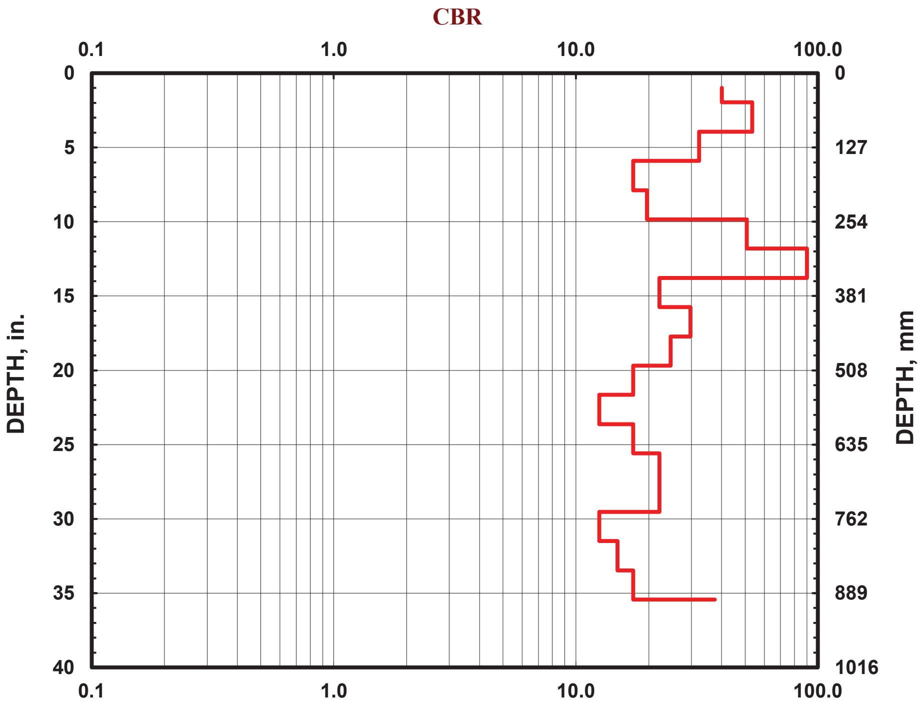
# DCP TEST DATA

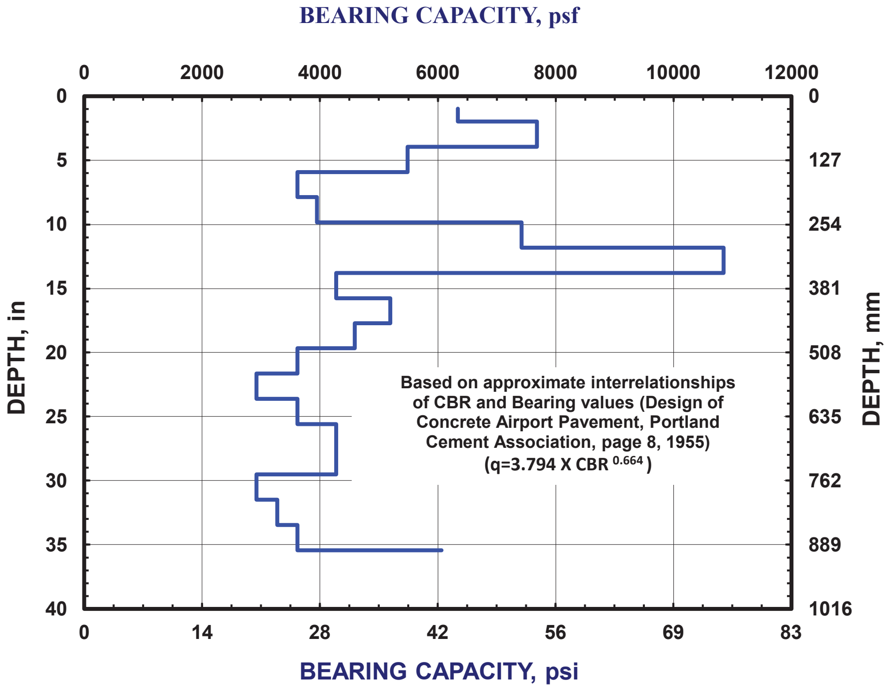
**Project:** *Make Beacon Hill Public - Phase 2*
**Date:** 25-Jul-23
**Location:** B-3
**Soil Type(s):** Gravel

Hammer
● 10.1 lbs.
○ 17.6 lbs.
○ Both hammers used

Soil Type
○ CH
○ CL
● All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 8.5 | 25 | 2 |
| 8.5 | 50 | 2 |
| 11 | 75 | 2 |
| 11 | 100 | 2 |
| 7 | 125 | 2 |
| 7 | 150 | 2 |
| 4 | 175 | 2 |
| 4 | 200 | 2 |
| 4.5 | 225 | 2 |
| 4.5 | 250 | 2 |
| 10.5 | 275 | 2 |
| 10.5 | 300 | 2 |
| 17.5 | 325 | 2 |
| 17.5 | 350 | 2 |
| 5 | 375 | 2 |
| 5 | 400 | 2 |
| 6.5 | 425 | 2 |
| 6.5 | 450 | 2 |
| 5.5 | 475 | 2 |
| 5.5 | 500 | 2 |
| 4 | 525 | 2 |
| 4 | 550 | 2 |
| 3 | 575 | 2 |
| 3 | 600 | 2 |
| 4 | 625 | 2 |
| 4 | 650 | 2 |
| 5 | 675 | 2 |
| 5 | 700 | 2 |
| 5 | 725 | 2 |
| 5 | 750 | 2 |
| 3 | 775 | 2 |
| 3 | 800 | 2 |
| 3.5 | 825 | 2 |
| 3.5 | 850 | 2 |
| 4 | 875 | 2 |
| 4 | 900 | 2 |





Based on approximate interrelationships of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955) ($q=3.794 \times CBR^{0.664}$)

**Figure 5-2**

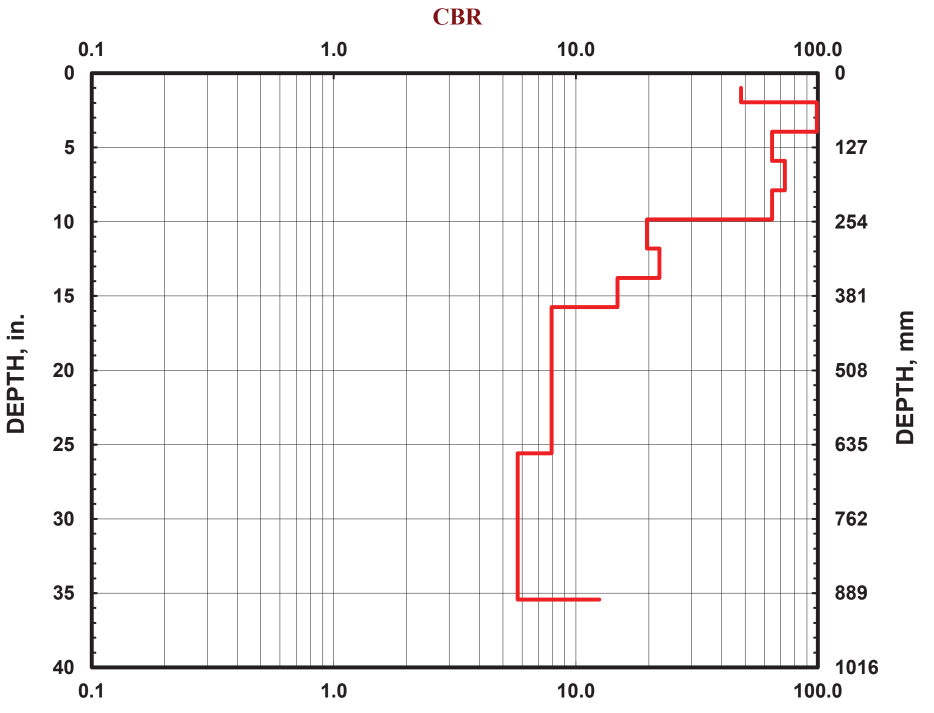
# DCP TEST DATA

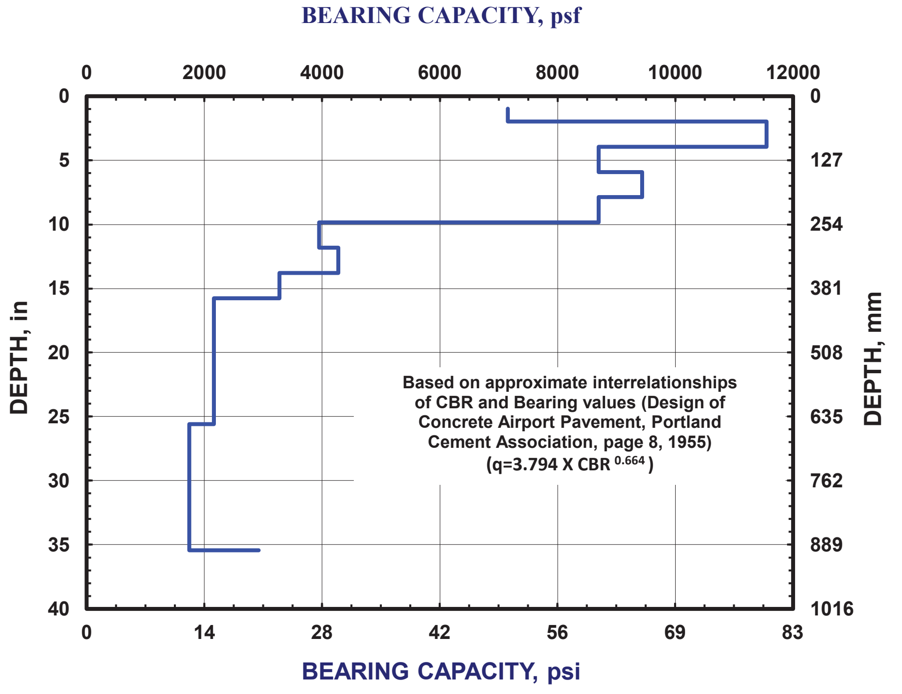
**Project:** *Make Beacon Hill Public - Phase 2*
**Date:** 26-Jul-23
**Location:** B-7
**Soil Type(s):** Sand

Hammer
● 10.1 lbs.
○ 17.6 lbs.
○ Both hammers used

Soil Type
○ CH
○ CL
● All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 10 | 25 | 2 |
| 10 | 50 | 2 |
| 32.5 | 75 | 2 |
| 32.5 | 100 | 2 |
| 13 | 125 | 2 |
| 13 | 150 | 2 |
| 14.5 | 175 | 2 |
| 14.5 | 200 | 2 |
| 13 | 225 | 2 |
| 13 | 250 | 2 |
| 4.5 | 275 | 2 |
| 4.5 | 300 | 2 |
| 5 | 325 | 2 |
| 5 | 350 | 2 |
| 3.5 | 375 | 2 |
| 3.5 | 400 | 2 |
| 2 | 425 | 2 |
| 2 | 450 | 2 |
| 2 | 475 | 2 |
| 2 | 500 | 2 |
| 2 | 525 | 2 |
| 2 | 550 | 2 |
| 2 | 575 | 2 |
| 2 | 600 | 2 |
| 2 | 625 | 2 |
| 2 | 650 | 2 |
| 1.5 | 675 | 2 |
| 1.5 | 700 | 2 |
| 1.5 | 725 | 2 |
| 1.5 | 750 | 2 |
| 1.5 | 775 | 2 |
| 1.5 | 800 | 2 |
| 1.5 | 825 | 2 |
| 1.5 | 850 | 2 |
| 1.5 | 875 | 2 |
| 1.5 | 900 | 2 |





Based on approximate interrelationships of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955) ($q=3.794 \times CBR^{0.664}$)

**Figure 5-3**

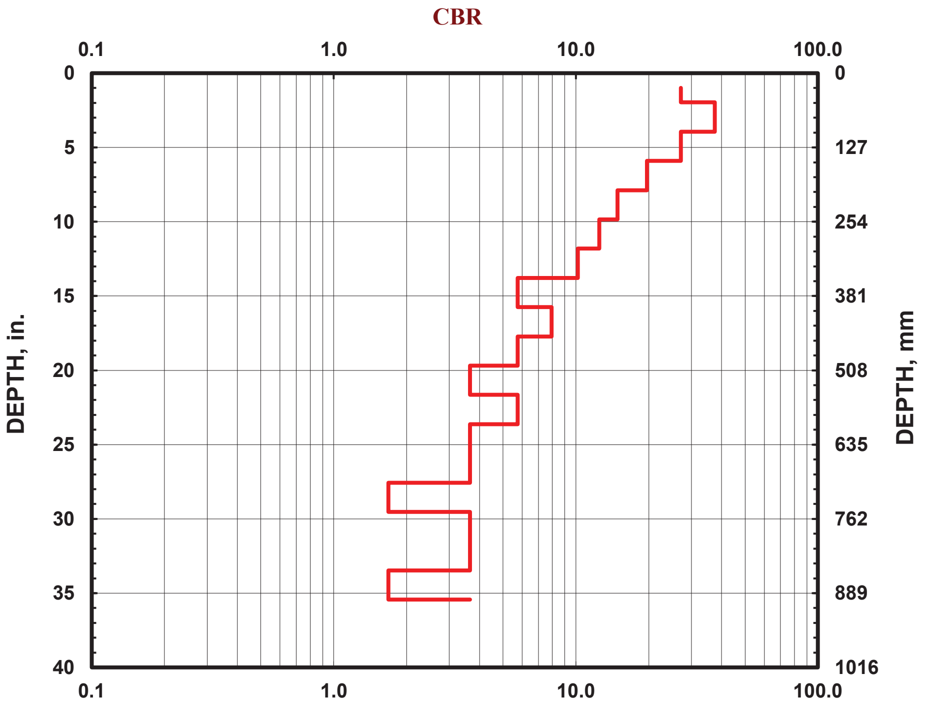
# DCP TEST DATA

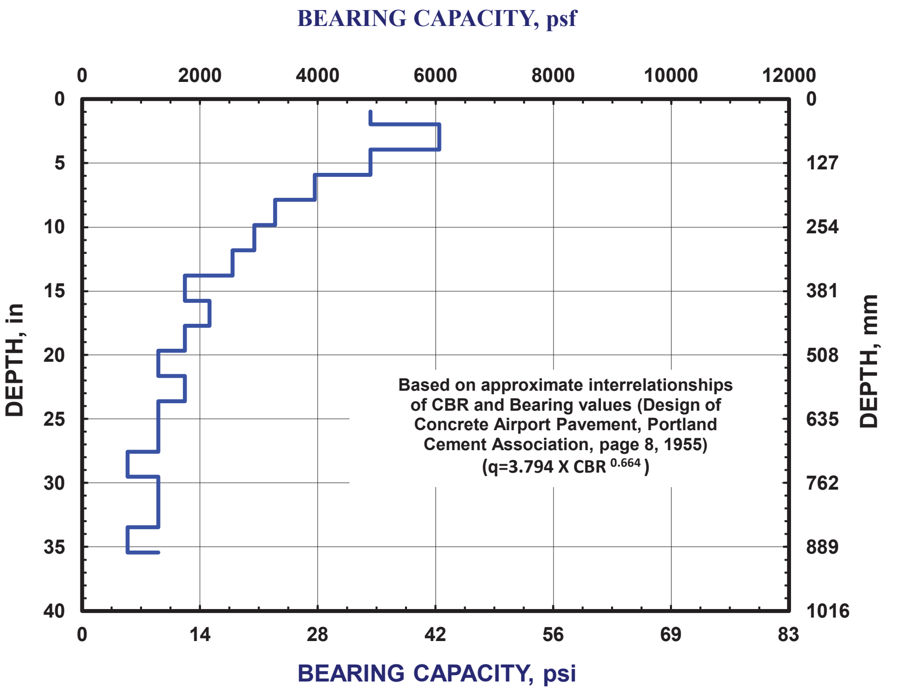
**Project:** *Make Beacon Hill Public - Phase 2*
**Location:** B-8
**Date:** 26-Jul-23
**Soil Type(s):** Silt

Hammer
● 10.1 lbs.
○ 17.6 lbs.
○ Both hammers used

Soil Type
○ CH
○ CL
● All other soils

| No. of Blows | Accumulative Penetration (mm) | Type of Hammer |
|---|---|---|
| 6 | 25 | 2 |
| 6 | 50 | 2 |
| 8 | 75 | 2 |
| 8 | 100 | 2 |
| 6 | 125 | 2 |
| 6 | 150 | 2 |
| 4.5 | 175 | 2 |
| 4.5 | 200 | 2 |
| 3.5 | 225 | 2 |
| 3.5 | 250 | 2 |
| 3 | 275 | 2 |
| 3 | 300 | 2 |
| 2.5 | 325 | 2 |
| 2.5 | 350 | 2 |
| 1.5 | 375 | 2 |
| 1.5 | 400 | 2 |
| 2 | 425 | 2 |
| 2 | 450 | 2 |
| 1.5 | 475 | 2 |
| 1.5 | 500 | 2 |
| 1 | 525 | 2 |
| 1 | 550 | 2 |
| 1.5 | 575 | 2 |
| 1.5 | 600 | 2 |
| 1 | 625 | 2 |
| 1 | 650 | 2 |
| 1 | 675 | 2 |
| 1 | 700 | 2 |
| 0.5 | 725 | 2 |
| 0.5 | 750 | 2 |
| 1 | 775 | 2 |
| 1 | 800 | 2 |
| 1 | 825 | 2 |
| 1 | 850 | 2 |
| 0.5 | 875 | 2 |
| 0.5 | 900 | 2 |





Based on approximate interrelationships of CBR and Bearing values (Design of Concrete Airport Pavement, Portland Cement Association, page 8, 1955) ($q=3.794 \times CBR^{0.664}$)

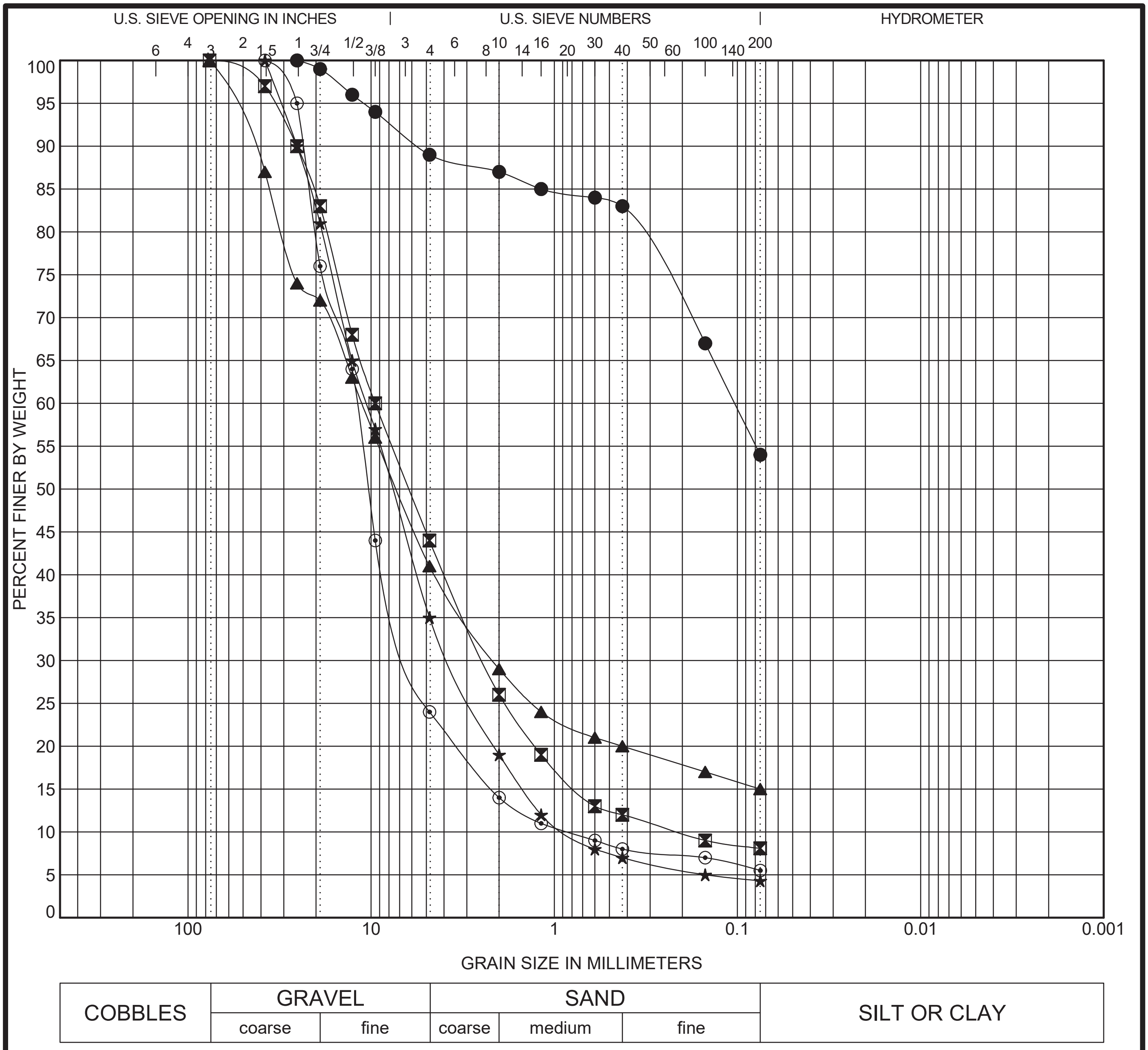
**Figure 5-4**

***SOIL MECHANICS***
***LABORATORY SUMMARY***

| | | Units | Test Methods | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| LABORATORY NUMBER | | | | 23-5397 | 23-5387 | 23-5388 | 23-5398 | 23-5399 | 23-5389 | 23-5390 | 23-5391 | 23-5392 | 23-5395 | 23-5396 | 23-5400 | 23-5393 | 23-5394 |
| TEST BORING NUMBER | | | | 6 | 1 | 1 | 7 | 8 | 1 | 1 | 2 | 3 | 5 | 5 | 7 | 4 | 4 |
| DEPTH | TOP | feet | | 0 | 0 | 5 | 2 | 0 | 10 | 15 | 5 | 5 | 5 | 10 | 10 | 3 | 7 |
| | BOTTOM | feet | | 2 | 2 | 7 | 4 | 2 | 12 | 17 | 7 | 6 1/2 | 7 | 12 | 12 | 3 1/2 | 7 1/2 |
| STRATUM | | | | *existing fill* | *fine-grained soil* | | | | *gravel* | | | | | | | *rock* | |
| MOISTURE CONTENT | | % | ASTM D2216 | 8.4 | | 8 | 12.8 | 5.1 | 5.1 | | 4.2 | 1.5 | 6.4 | | | | |
| CATION EXCHANGE CAPACITY | | meq/100g | EPA 9081 | | 11.2 | | 12.1 | | | | | | | | | | |
| pH | | | AASHTO T289 | | 5.3 | | 8 | | | | | | | | | | |
| ORGANIC CONTENT | | % | ASTM D2974 | | 4.2 | | 4.2 | | | | | | | | | | |
| LIQUID LIMIT | | % | ASTM D4318 | | | 27 | | | | | | | | | | | |
| PLASTIC LIMIT | | % | | | | 17 | | | | | | | | | | | |
| PLASTICITY INDEX | | % | | | | 10 | | *NP | | | | | | | | | |
| UNCONFINED COMPRESSIVE STRENGTH | | psi | ASTM D7012 | | | | | | | | | | | | | 13,642 | 15,083 |
| UNIFIED CLASSIFICATION | | | ASTM D2487 | | | CL | | ML | | | | GW | | | | | |
| SIEVE ANALYSIS | | | ASTM D6913 | | | | | | | | | | | | | | |
| | 6" | | | | | | | | | | | | | | | | |
| | 3" | | | | | | | | 100 | | 100 | | | | | | |
| | 1 1/2" | | | 100 | | | | 100 | 97 | | 87 | 100 | 100 | | | | |
| S | 1" | % | | 93 | | 100 | | 98 | 90 | | 74 | 90 | 95 | | | | |
| I | 3/4" | | | 89 | | 99 | 100 | 97 | 83 | | 72 | 81 | 76 | | | | |
| E | 1/2" | P | | 75 | | 96 | 93 | 96 | 68 | | 63 | 65 | 64 | | | | |
| V | 3/8" | A | | 65 | | 94 | 89 | 95 | 60 | | 56 | 57 | 44 | | | | |
| E | #4 | S | | 44 | | 89 | 77 | 92 | 44 | | 41 | 35 | 24 | | | | |
| | #10 | S | | 37 | | 87 | 70 | 89 | 26 | | 29 | 19 | 14 | | | | |
| S | #16 | I | | 33 | | 85 | 67 | 87 | 19 | | 24 | 12 | 11 | | | | |
| I | #30 | N | | 29 | | 84 | 63 | 83 | 13 | | 21 | 8 | 9 | | | | |
| Z | #40 | G | | 28 | | 83 | 60 | 81 | 12 | | 20 | 7 | 8 | | | | |
| E | #100 | | | 23 | | 67 | 49 | 76 | 9 | | 17 | 5 | 7 | | | | |
| | #200 | | ASTM D1140 | 21 | | 54 | 43 | 65 | 8.1 | 5.6 | 15 | 4.3 | 5.5 | 3.5 | 6.6 | | |

*NP = Non Plastic

**Figure 6**



| Specimen Identification | | | Classification | LL | PL | PI | Cc | Cu |
|---|---|---|---|---|---|---|---|---|
| ● | 1 | 5.0 | **SANDY LEAN CLAY(CL)** | 27 | 17 | 10 | | |
| ☒ | 1 | 10.0 | | | | | 2.93 | 44.76 |
| ▲ | 2 | 5.0 | | | | | | |
| ★ | 3 | 5.0 | **WELL-GRADED GRAVEL with SAND(GW)** | | | | 1.50 | 12.59 |
| ⊙ | 5 | 5.0 | | | | | 3.44 | 14.24 |

| Specimen Identification | | | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | 1 | 5.0 | 25.4 | 0.103 | | | 11.0 | 35.0 | 54.0 | |
| ☒ | 1 | 10.0 | 76.2 | 9.5 | 2.43 | 0.212 | 56.1 | 35.7 | 8.1 | |
| ▲ | 2 | 5.0 | 76.2 | 11.214 | 2.151 | | 58.8 | 25.9 | 15.0 | |
| ★ | 3 | 5.0 | 38 | 10.593 | 3.651 | 0.841 | 65.2 | 30.5 | 4.3 | |
| ⊙ | 5 | 5.0 | 38 | 11.984 | 5.891 | 0.841 | 76.1 | 18.4 | 5.5 | |

Budinger
& Associates
1101 North Fancher Road
Spokane Valley, WA 99212

# GRAIN SIZE DISTRIBUTION

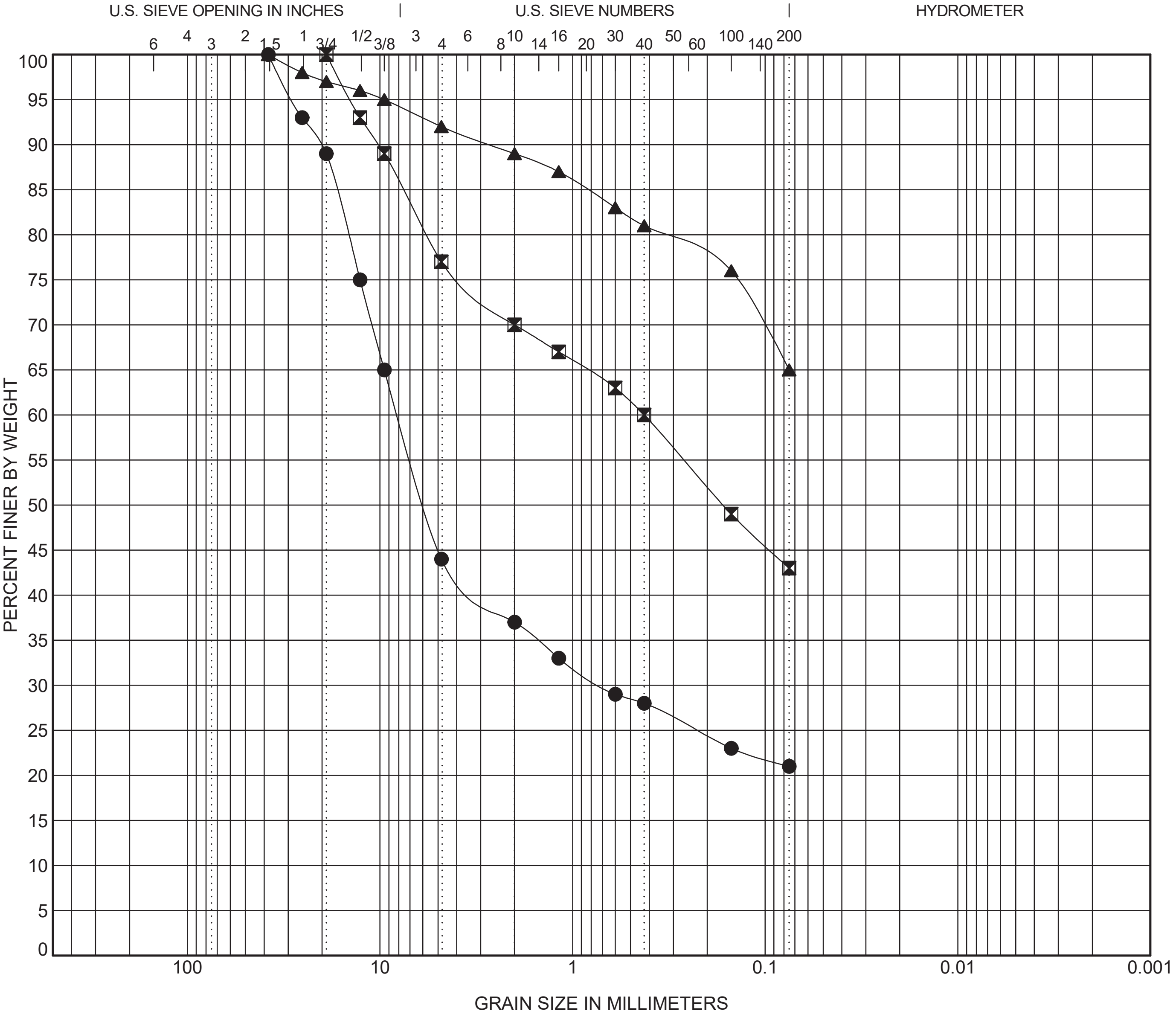
Project: Make Beacon Hill Public - Phase 2

Location: Spokane Valley, WA

Number: S23325

**Figure 7-1**

GRAINSIZE WO FIGURE # S23325.GPJ GINT STD US.GDT 8/18/23



| COBBLES | GRAVEL | | SAND | | | SILT OR CLAY |
|---|---|---|---|---|---|---|
| | coarse | fine | coarse | medium | fine | |

| Specimen Identification | | | Classification | LL | PL | PI | Cc | Cu |
|---|---|---|---|---|---|---|---|---|
| ● | 6 | 0.0 | | | | | | |
| ☒ | 7 | 2.0 | | | | | | |
| ▲ | 8 | 0.0 | **SANDY SILT(ML)** | **NP** | **NP** | **NP** | | |
| | | | | | | | | |
| | | | | | | | | |

| Specimen Identification | | | D100 | D60 | D30 | D10 | %Gravel | %Sand | %Silt | %Clay |
|---|---|---|---|---|---|---|---|---|---|---|
| ● | **6** | **0.0** | **38** | **8.075** | **0.711** | | **56.1** | **22.9** | **21.0** | |
| ☒ | **7** | **2.0** | **19** | **0.425** | | | **23.1** | **33.9** | **43.0** | |
| ▲ | **8** | **0.0** | **38** | | | | **8.0** | **27.0** | **65.0** | |
| | | | | | | | | | | |
| | | | | | | | | | | |

Budinger
& Associates
1101 North Fancher Road
Spokane Valley, WA 99212

# GRAIN SIZE DISTRIBUTION

Project: Make Beacon Hill Public - Phase 2
Location: Spokane Valley, WA
Number: S23325

**Figure 7-2**

# Important Information about This

# Geotechnical-Engineering Report

**Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.**

**While you cannot eliminate all such risks, you can manage them. The following information is provided to help.**

## Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a constructor — a construction contractor — or even another civil engineer. Because each geotechnical- engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply this report for any purpose or project except the one originally contemplated.

## Read the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

## Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical-engineering report that was:

- not prepared for you;
- not prepared for your project;
- not prepared for the specific site explored; or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

## Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the geotechnical engineer performed the study. *Do not rely on a geotechnical-engineering report whose adequacy may have been affected by*: the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Contact the geotechnical engineer before applying this report to determine if it is still reliable.* A minor amount of additional testing or analysis could prevent major problems.

## Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ — sometimes significantly — from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide geotechnical-construction observation is the most effective method of managing the risks associated with unanticipated conditions.

## A Report's Recommendations Are Not Final

Do not overrely on the confirmation-dependent recommendations included in your report. *Confirmation-dependent recommendations are not final*, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

## A Geotechnical-Engineering Report Is Subject to Misinterpretation

Other design-team members' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical construction observation.

### Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical-engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

### Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure constructors have sufficient time* to perform additional study. Only then might you be in a position to give constructors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

### Read Responsibility Provisions Closely

Some clients, design professionals, and constructors fail to recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

### Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. *Do not rely on an environmental report prepared for someone else.*

### Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold- prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical- engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; *none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.*

### Rely, on Your GBC-Member Geotechnical Engineer for Additional Assistance

Membership in the Geotechnical Business Council of the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you GBC-Member geotechnical engineer for more information.

GEOTECHNICAL
BUSINESS COUNCIL
*of the Geoprofessional Business Association*

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@geoprofessional.org www.geoprofessional.org