



***Report of Geotechnical Engineering
Services
Veterans Affairs Primary Care Facility
E. Fourth Plain Boulevard and
St. Johns Boulevard
Vancouver, WA***



***Prepared for
HDJ Design Group and
Anderson Dabrowski Architects***



***April 30, 2013
15949-01***



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**REPORT OF GEOTECHNICAL ENGINEERING SERVICES
VETERANS AFFAIRS PRIMARY CARE FACILITY
E. FOURTH PLAIN BOULEVARD AND ST. JOHNS BOULEVARD
VANCOUVER, WASHINGTON**

1 INTRODUCTION

Hart Crowser, Inc. (Hart Crowser) is pleased to submit our report of geotechnical engineering services for the proposed Veterans Affairs (VA) primary care facility located on the VA complex southwest of the intersection of E. Fourth Plain Boulevard and St. Johns Boulevard in Vancouver, Washington. Our work was completed in general accordance with our proposal dated September 10, 2012 and our Consultant Agreement dated February 26, 2013.

We understand the proposed development work includes construction of an approximately 20,000-square-foot, one-story, primary care facility in the southwest portion of the site. A second story will eventually be added to the building.

Based on our correspondence with the design team, we understand the building will be supported by a series of columns founded on isolated spread footings. The typical structural load for the final two-story building will be approximately 100 to 175 kips per column. Based on the level nature of the site, we anticipate that site grading will be minor with mass cuts and fills less than approximately 2 feet deep/thick. New asphalt paving will be required to provide a parking lot and drive aisles. Concrete pads may be used at trash enclosures and walkways. Stormwater will be treated on site and then discharged via infiltration systems.

The location of the site is shown on Figure 1. The existing site layout is shown on Figure 2.

2 SCOPE OF SERVICES

The purpose of our work was to evaluate subsurface conditions for the proposed development and to provide geotechnical engineering services for design of specific project elements. Our complete scope of work was described in our September 10, 2012 proposal and is summarized below:

- Reviewed existing published information and maps for the site.

- Drilled four borings using a truck-mounted drill rig.
- Logged the borings and collected representative soil samples.
- Conducted field infiltration testing.
- Completed laboratory tests to evaluate soil engineering properties.
- Completed engineering analysis to evaluate infiltration characteristics and for foundation, seismic, and pavement design.
- Prepared this report summarizing our findings, conclusions, and recommendations, including information relating to earthwork, foundation design, seismic design, stormwater infiltration, and pavements.

3 SITE CONDITIONS

3.1 *Geologic and Soil Mapping*

The geology of the site is mapped in the Washington Department of Geology and Earth Resources (DGER) *Geologic Map of the Vancouver Quadrangle, Washington and Oregon* (Phillips, 1987). As mapped by Phillips (1987), the site is underlain by gravel-sized flood deposits consisting of “well-rounded, foreset-bedded, well-sorted and stratified pebble and cobble gravel...with a sandy matrix...” Our subsurface investigation suggests that the site geology is more similar to Phillips’ (1987) description of sand-sized flood deposits—“very coarse to fine sand...” with a thin surface layer of fine-grained flood deposits—“very fine sand, silt, and clay...” than to pebble and cobble gravel, as described for the site.

The soils at the site are mapped by the U.S. Department of Agriculture in the *Soil Survey of Clark County, Washington* (McGee 1972) as mantled by Lauren gravelly loam, 0 to 8 percent. Lauren soils are described as deep and somewhat excessively drained, with moderately rapid to rapid permeability, slow runoff, and slight erosion potential. The upper 33 inches of the soil is estimated to have a hydraulic conductivity (permeability) of 0.6 to 2 inches per hour increasing to 2 to 6 inches per hour from 33 to 44 inches below ground surface (bgs). Below 44 inches the estimated value further increases to 6.0 to 20 inches per hour.

Groundwater mapping prepared by Clark County (Swanson and McCarley 1995) shows the approximate depth to groundwater at 190 to 200 feet bgs.

3.2 Geologic and Seismic Hazards

Overall seismic hazards for this area have been mapped by the Washington Division of Geology and Earth Resources Open File Report 2004-20 by Palmer, et al (2004a). This map series includes hazard maps for liquefaction and ground shaking amplification (e.g., Site Class). These and other seismic hazards are discussed in the following paragraphs.

Refer to *Section 6.3 - Seismic Design* of this report for the recommended International Building Code (IBC) seismic design parameters.

3.2.1 Liquefaction

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles, resulting in the sudden loss of shear strength in the soil. Granular soils, which rely on interparticle friction for strength, are susceptible to liquefaction until the excess pore pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water. In general, loose, saturated sand soils with low silt and clay contents are the most susceptible to liquefaction. Silty soils with low plasticity are moderately susceptible to liquefaction under relatively higher levels of ground shaking. For any soil type, the soil must be saturated for liquefaction to occur.

Palmer, et al (2004a) indicates the site has a “very low” potential for liquefaction susceptibility. Based on the lack of shallow groundwater at the site, the very low liquefaction potential designation appears appropriate for the site.

3.2.2 Site Classification

Palmer, et al (2004b) identifies the site as having a National Earthquake Hazards Reduction Program (NEHRP) “Site Class C” designation. The “Site Class” is a classification based on the properties of the upper 100 feet of the soil and bedrock materials at a site. Site Class C indicates a relatively low to moderate potential for ground motion amplification.

Our site-specific investigation generally revealed loose to medium dense granular flood deposits. Analysis based on the relative density of the soils encountered indicates a Site Class D is warranted. See *Section 6.3 - Seismic Design* of this report for additional discussion.

3.2.3 Earthquake Induced Landsliding

Based on the flat site slopes, it is our opinion the potential for earthquake-induced landsliding is low.

3.2.4 Summary

Based on our research, subsurface exploration, and our experience with similar soils at nearby sites, we agree that the potential for the site to be affected by liquefaction and landsliding is very low. However, our analysis indicates that the Site Class mapping slightly underestimates the risk for ground shaking amplification. Refer to *Section 6.3 - Seismic Design* of this report for the recommended seismic design parameters.

3.3 Surface Conditions

The project site is located within the VA complex generally located south of E. Fourth Plain Boulevard and west of St Johns Boulevard. The project site occupies approximately 2.8 acres in the north-central portion of the VA complex. The site is bound by B Street to the south, E. Fourth Plain Boulevard to the north and east and a paved parking lot to the west, which serves the Clark County Center for Community Health building.

The majority of the project site is comprised of a grassy field. The southern portion of the site is comprised of a C-TRAN transit stop, with concrete sidewalks surrounding a three-lane paved bus turnaround area. An approximately 8,000-square-foot gravel parking area is present in the northwest corner of the project site.

The site is generally level and at the grade of the adjacent streets to the north, east, and west. The south border of the site gently slopes down to the elevation of the adjacent road. Site elevations range from 201 feet above mean sea level (MSL) in the northern portion to 191 feet MSL at the southwest corner edge of the project site.

3.4 Subsurface Conditions

3.4.1 General

We explored subsurface soil and groundwater conditions at the site by drilling four borings on March 18, 2013. Infiltration testing was conducted at two of the boring locations. All explorations were performed by a truck-mounted drill rig. The exploration locations are shown on Figure 2. Appendix A summarizes our exploration and infiltration methods and presents our exploration logs.

Laboratory test results are provided on the exploration logs and described in Appendix B.

Soils encountered in our explorations include a relatively thin layer of low-plasticity silt overlying sand with variable amounts of gravel to the depths explored. The sands generally ranged from silty medium-grained sand near the ground surface to well-graded sand and gravel with depth. These soils are discussed in detail in the paragraphs below.

3.4.2 Silt Deposits

Below an approximately 6-inch layer of organic topsoil, native silt was encountered to depths of 3.5 to 7 feet bgs. In general, the silt is brown, moist, medium stiff, low to medium plasticity, and with trace to some sand and gravel.

Standard Penetration Test (SPT) blow counts (“N-values”) in the silt were between 5 blows per foot (bpf) and 15 bpf, averaging approximately 8 bpf. Two moisture content tests of the silt returned values of 20 and 21 percent.

3.4.3 Sand and Gravel Deposits

Below the silt, the site is underlain to the depths explored by sand and gravel flood deposit soils ranging from silty medium-grained sand to well-graded sand with gravel at depth. In general, the upper 5 feet of the deposits consists of loose to medium dense, poorly graded sand with trace to some silt and trace to some sub-rounded gravel. Below this, the deposits generally became medium dense, fines content decreased, and gravel content increased.

The upper sand deposits were typically loose to medium dense, with SPT blow counts (“N-values”) between 8 blows per foot (bpf) and 11 bpf, averaging approximately 10 bpf. The deeper sand deposits were typically medium dense with N-values between 11 and 26 bpf, averaging approximately 16 bpf. One sample in boring B-1 at a depth between 13 and 14 feet bgs returned a blow count of 32 as a result of locally increased gravel content.

Measured moisture contents of the granular deposits ranged from 7 percent to 12 percent based on 12 tests. Six fines content tests returned values between 6 and 18 percent fines (percent of silt- and clay-size particles). Three gradation (sieve) tests yielded the same classification of well graded sand with trace to some silt and gravel (Unified Soil Classification System [USCS] classification SW). These test results are shown on the exploration logs in Appendix A and on the figures in Appendix B.

3.4.4 Groundwater

No groundwater was encountered in our explorations. As noted previously, groundwater mapping prepared by Clark County (Swanson and McCarley 1995) shows the approximate depth to groundwater at 190 to 200 feet bgs.

3.4.5 Infiltration Testing

We performed two infiltration tests at the site. Infiltration test holes were advanced adjacent to borings B-3 and B-4 at depths of 6 and 8 feet bgs, respectively.

The infiltration tests consisted of falling head encased borehole tests, conducted in general accordance with the procedures referenced in Clark County's *Stormwater Manual* (Clark County 2009) and as described in Appendix A of this report.

A summary of the field testing results is provided in Table 1.

Table 1 - Infiltration Results

Exploration / Infiltration Test ID	Depth (feet)	Soil Description	Measured Drop Rate (inches/hour)	Calculated Coefficient of Permeability (inches/hour)
B-3 / IT-1	6.0	Well graded sand with silt (SW)	50	20
B-4 / IT-2	8.0	Well graded sand with silt (SW)	25	7

Note: The drop rates and coefficients of permeability reported in Table 1 are unfactored values. Correction factors need to be applied to these rates.

The designer should refer to *Section 7 - Drainage Design Recommendations* of this report for further discussion on the infiltration results.

4 CONCLUSIONS

Based on our explorations, testing, and analyses, it is our opinion that the site is suitable for the proposed use, provided the recommendations in this report are included in design and construction. We offer the following general summary of our conclusions:

- Site soils generally consist of 3.5 to 7.5 feet of fine-grained silt overlying sandier soils. The soils should be readily excavatable by conventional

equipment. However, the near-surface silts may be easily disturbed by construction activities when moist or wet.

- The proposed building may be supported by a conventional spread footing foundation system; however, the use of gravel pads beneath the footings may be required to reduce potential settlement, depending upon foundation loads.
- The sand deposits encountered below a 3.5 to 7.5-foot layer of near-surface silt appear to be suitable for infiltration of stormwater.

The following sections present our specific recommendations for earthworks and structural components of the project.

5 EARTHWORKS RECOMMENDATIONS

5.1 General

Based on available information, we estimate the mass grading for the site will be minimal with cuts and fills on the order of 2 feet deep/thick. However, deeper excavations will be required for installation of utilities.

All earthwork activities should be conducted in accordance with Washington State Department of Transportation's (WSDOT) *Standard Specifications for Road, Bridge, and Municipal Construction* [WSS] (WSDOT 2012).

5.2 Site Preparation

5.2.1 Clearing and Grubbing

Initial site preparation and earthwork operations will include clearing and grubbing, stripping, and grading to establish subgrade elevation for improvements. Most of the site is covered with grass lawn and planter areas. We estimate that the average depth of material to be stripped is 6 inches. However, the actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal or stockpiled for use in landscaped areas.

Trees and their root balls should be grubbed out to the depth of the roots, which could exceed 3 feet bgs. Depending upon the methods used to remove the root balls, considerable disturbance and loosening of the subgrade could occur during site grubbing. We recommend that soil disturbed during grubbing

operations be removed to expose firm, undisturbed subgrade. The resulting excavations should be backfilled with compacted structural fill.

5.2.2 Demolition

Demolition should include complete removal of existing site improvements within areas to receive new pavements, buildings, or engineered fills. Underground utility lines or vaults encountered in areas of new improvements should be completely removed or grouted full if left in place.

Voids resulting from removal of pavements, sidewalks, etc. or loose soil in utility lines should be backfilled with compacted structural fill, as discussed in *Section 5.5 - Structural Fills and Backfills* of this report. The base of such excavations should be completed to a firm subgrade before filling, and their sides sloped at a minimum of 1 horizontal to 1.5 vertical (1H:1.5V) gradient to allow for more uniform compaction at the edges of the excavations.

Materials generated during demolition of existing improvements should be transported off site for disposal or stockpiled in areas designated by the owner. In general, these materials will not be suitable for reuse as engineered fill. However, asphalt, concrete, and base rock materials may be crushed and recycled for use as general fill. Such recycled materials should meet the specifications for imported granular material, as described in *Section 5.5 - Structural Fills and Backfills* of this report.

5.2.3 Subgrade Preparation and Evaluation

Following demolition, site preparation, and rough grading, the suitability of the subgrade should be evaluated by proofrolling with a fully loaded dump truck or similar heavy rubber-tired construction equipment to identify any remaining soft, loose, or unsuitable areas. The proofroll should be conducted prior to placing new fill. The proofrolling should be observed by a representative of Hart Crowser who would evaluate the suitability of the subgrade and identify areas of yielding that are indicative of soft or loose soil. During wet weather or when the exposed subgrade is wet or unsuitable for proofrolling, the prepared subgrade should be evaluated by observing excavation activity and probing with a steel foundation probe. Observations and probing should be performed by Hart Crowser.

If soft or loose zones are identified during proofrolling or probing, these areas should be excavated to the extent indicated by Hart Crowser and replaced with structural fill.

If site preparation activities cause excessive subgrade disturbance, replacement with imported structural fill may be necessary. Disturbance to the subgrade should be expected if site preparation and earthwork are conducted during periods of excessive wet weather and/or when the moisture content of the surficial soil exceeds optimum.

5.3 Wet Soil/Wet Weather Construction

The near-surface silty sands may be easily disturbed when they are wet or heavily trafficked. If not carefully executed, site preparation, utility trench work, and pavement construction can create extensive soft areas and significant repair costs can result. Earthwork planning should include considerations for minimizing subgrade disturbance.

One method for minimizing subgrade disturbance during construction is through the use of granular haul roads and staging areas. Based on our experience, between 12 and 18 inches of imported granular material is generally required in staging and haul roads areas. However, the actual thickness will depend on the contractor's means and methods, and accordingly, should be the contractor's responsibility. Additionally, a geotextile fabric should be placed as a barrier between the subgrade and imported granular material in areas of repeated construction traffic. The imported granular material and the geotextile fabric should meet the specifications in *Section 5.5 - Structural Fills and Backfills* of this report.

We note that the "design aggregate base" thickness for pavement areas is intended to support post-construction design traffic loads and should not be used to support construction traffic when the subgrade soils are wet. Accordingly, if staging areas or haul roads are proposed, the "design thickness" of the base rock should not be relied upon and additional thicknesses of base rock should be placed.

5.4 Excavation

5.4.1 General Excavations

Site soils are generally medium stiff and loose to medium dense within expected excavation depths. It is our opinion that conventional earthmoving equipment in proper working condition should be capable of making necessary general excavations for utilities, footings, and other earthwork. Also, occasional cobbles and small boulders may be encountered in the granular native site soils, particularly in the deeper granular soil unit. The earthwork contractor should be

responsible for providing equipment and following procedures as needed to excavate the site soils as described in this report.

5.4.2 Trench Excavations

Temporary trench excavation sidewalls should stand vertical to a depth of approximately 4 feet, provided surface water is directed away from the excavations. Due to the granular nature of the deeper site soils, excavations that extend below approximately 4 feet below existing grades will have a high susceptibility to caving and running of the sidewalls. Open excavation techniques may be used to excavate trenches with depths between 4 and 10 feet bgs, provided the walls of the excavation are cut at a slope of 1H:1V or flatter. In lieu of large open cuts, approved temporary shoring may be used for excavation support. A wide variety of shoring systems are available; consequently, we recommend that the contractor be responsible for selecting the appropriate system. All trench excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. Site soils are generally OSHA Type C.

We note that box shoring is a safety feature used to protect workers and does not prevent caving. If the excavations are left open for extended periods of time, then caving of the sidewalls may occur. The presence of caved material will limit the ability to properly backfill and compact the trenches. The voids between the box shoring and the sidewalls of the trenches should be properly filled with sand or gravel before caving occurs.

5.4.3 Dewatering

Groundwater is not expected within the depths of excavations; however, the project contractor should be made responsible for temporary drainage of surface water to prevent standing water and/or erosion at the working surface or in excavations.

5.5 Structural Fills and Backfills

Structural fills should be considered to include subgrade soils beneath buildings, foundations, slabs, and pavements; backfill behind retaining walls for a horizontal distance equal to the wall height; fills on slopes greater than 5H:1V; and other areas intended to support structures or within the influence zone of structures.

Fills should only be placed over a subgrade that has been prepared in conformance with the prior sections of this report. A variety of material may be

used as structural fill at the site. However, all material used as structural fill should be free of organic matter or other unsuitable materials and should meet specifications provided in the WSS (WSDOT 2012). A brief characterization of some of the acceptable materials and our recommendations for their use as structural fill are provided below.

5.5.1 On-Site Soils

On-site, near-surface soils that might be used for on-site fill generally consist of silt and well graded sand to sand with silt and gravel. This material may be used as structural fill if the material is properly moisture conditioned; free of debris, organic materials, and particles over 6 inches in diameter; and meets the specifications provided in WSS 9-03.14(3) – Common Borrow. We note that the near-surface silts are sensitive to moisture and may require moisture conditioning before they can be used. If used, the on-site soils should be placed and compacted in lifts with maximum uncompacted thicknesses and relative densities as recommended in the tables that follow.

5.5.2 Imported Select Structural Fill

Imported granular material used as structural fill should be pit or quarry run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in WSS 9-03.9(1) – Ballast, WSS 9-03.14(1) – Gravel Borrow, or WSS 9-03.14(2) – Select Borrow. However, the imported granular material should also have a maximum size of 2 inches and be angular, fairly well graded between coarse and fine material, have less than 5 percent by dry weight passing the U.S. Standard No. 200 Sieve, and have at least two mechanically fractured faces. The material should be placed and compacted in lifts with maximum uncompacted thicknesses and relative densities as recommended in the tables that follow.

5.5.3 Aggregate Bases

Imported granular material used as aggregate base (base rock) beneath pavements or the building should be clean, crushed rock or crushed gravel and sand that is fairly well graded between coarse and fine. The base aggregate should meet the specifications provided in WSS 9-03.9 – Aggregates for Ballast and Crushed Surfacing, depending upon application, with the exception that the aggregate have less than 5 percent by dry weight passing a U.S. Standard No. 200 Sieve and have at least two mechanically fractured faces.

For use beneath pavements or footings, the aggregate base should have a maximum particle size of 1 or 1.5 inches, while for use beneath building or sidewalk slabs should have a maximum particle size of 0.75 or 1 inch. For use

beneath buildings, the base rock should also meet the gradation of WSS - 9-03.9(3) Crushed Surfacing for "Base Course," though should have less than 5 percent by dry weight passing a U.S. Standard No. 200 Sieve.

The aggregate base material should be placed and compacted in lifts with maximum uncompacted thicknesses and relative densities as recommended in the tables that follow.

5.5.4 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should consist of well graded granular material with a maximum particle size of 1 inch and should meet the specifications provided in WSS 9-03.12(3) – Gravel Backfill for Pipe Zone Bedding.

Within pavement and slab subgrades the remainder of the trench backfill up to the subgrade elevation can consist of the above 1-inch material or of granular material with a maximum particle size of 2.5 inches, less than 10 percent by dry weight passing the U.S. Standard No. 200 Sieve, and meeting the specifications provided in WSS 9-03.19 – Bank Run Gravel for Trench Backfill.

In landscape areas, trench backfill placed above the pipe zone may consist of general fill materials that are free of organics and materials over 3 inches in diameter, and meet the specifications provided in WSS 9-03.14(3) – Common Borrow and WSS 9-03.15 – Native Material for Trench Backfill, as appropriate.

The material should be placed and compacted in lifts with maximum uncompacted thicknesses and relative densities as recommended in the tables that follow.

5.5.5 Recycled Materials

Portland cement concrete (PCC), asphaltic concrete (AC), and base rock rubble may be used as structural fill, provided there is no contamination, the material meets the specifications provided in WSS 9-03.21 – Recycled Material, and that the recommendations below are followed.

5.5.5.1 Processed Fill Materials

Recycled material may be used as select structural fill, provided it is processed by crushing and screening, grinding in place, or other methods to meet the structural fill recommendations in this report. This recycled fill may be used as structural fill, except as base rock for pavements or buildings or within utility trenches (unless approved by the pipe manufacturer).

5.5.5.2 Unprocessed Fill Materials

PCC and AC rubble, which has a maximum particle size of 4 inches in nominal diameter, may be mixed with other imported or on-site fill to create a uniform, well graded material and used in pavement areas. We recommend that at least 1 foot of other processed or imported structural fill overlie the unprocessed fill material blend.

5.6 Fill Placement and Compaction

Structural fill should be placed and compacted in accordance with the following guidelines:

- Place fill and backfill on a prepared subgrade that consists of firm, inorganic native soils or approved structural fill.
- Place fill or backfill in uniform horizontal lifts with a thickness appropriate for the material type and compaction equipment. Table 2 provides general guidance for lift thicknesses.

Table 2 - Guidelines for Uncompacted Lift Thickness

Compaction Equipment	Guidelines for Uncompacted Lift Thickness (inches)		
	On Site Soil	Granular and Crushed Rock Maximum Particle Size $\leq 1\frac{1}{2}$ inch	Crushed Rock Maximum Particle Size $> 1\frac{1}{2}$ inch
Plate Compactors and Jumping Jacks	4 – 6	4 – 8	Not Recommended
Rubber-tire Equipment	6 – 8	10 – 12	6 – 8
Light Roller	8 – 10	10 – 12	8 – 10
Heavy Roller	10 – 12	12 – 18	12 – 16
Hoe Pack Equipment	12 – 16	18 – 24	12 – 16

Note: The above table is based on our experience and is intended to serve as a guideline. The information provided in this table should not be included in the project specifications.

- Use appropriate operating procedures to attain uniform coverage of the area being compacted.
- Place fill at a moisture content within approximately 3 percent of optimum as determined in accordance with ASTM International (ASTM) Test Method D 1557. Moisture condition fill soil to achieve uniform moisture content within the specified range before compacting. Compact fill to the percent of maximum dry densities as noted in Table 3.

- Do not place, spread, or compact fill soils during freezing or unfavorable weather conditions. Frozen or disturbed lifts should be removed or properly recompacted prior to placement of subsequent lifts of fill soils.

Table 3 - Fill Compaction Criteria

Fill Type	Percent of Maximum Dry Density Determined in Accordance with ASTM D 1557		
	0 – 2 Feet Below Subgrade	>2 Feet Below Subgrade	Pipe Bedding and Pipe Zone
Mass Fill (fine-grained soils)	92	92	-----
Mass Fill (granular materials)	95	90	-----
Aggregate Bases	95	95	-----
Trench Backfill	95	92	90
Nonstructural Trench Backfill	90	88	-----
Nonstructural Zones	90	88	90

Note: "Nonstructural" areas are only located in landscaping zones, where the potential for localized trench settlement is acceptable to the owner.

During structural fill placement and compaction, a sufficient number of in-place density tests should be completed by Hart Crowser to verify that the specified degree of compaction is being achieved. For structural fill with more than 30 percent retained on the 3/4-inch sieve, Hart Crowser should visually verify proper compaction with a proofroll or other methods.

6 STRUCTURAL DESIGN RECOMMENDATIONS

6.1 Foundation Support Recommendations

6.1.1 General

Based on the results of our investigation, it is our opinion the proposed structure can be supported on conventional spread footings bearing on native soil or new structural fill constructed in accordance with the recommendations in this report. Due to the relatively soft nature of the upper silts, the use of gravel pads beneath the footings may be required, depending upon building loads.

The following recommendations are based on the assumption that maximum structural loads will be between approximately 100 and 175 kips for column footings. If structural loads are greater, then we should be contacted to verify our recommendations are appropriate.

6.1.2 Dimensions and Design Parameters

We understand that no continuous footings are proposed, just isolated footings for columns. The isolated spread footings should have a minimum width of 3 feet. The bottom of all footings should be at least 2 feet below existing grades and 1 foot below the lowest adjacent finished grade or base of slab, whichever is deeper.

Isolated column footings with loads less than 125 kips, may be designed using an allowable bearing pressure of 2,500 pounds per square foot (psf). If the footings are underlain by a gravel pad (as described in *Section 6.1.3 - Foundation Subgrade Preparation* below), then the allowable bearing pressure may be increased to 3,500 psf. If isolated column footings have loads larger than 125 kips, then they shall be underlain by a gravel pad and may be designed using the 3,500 psf bearing capacity.

The bearing values provided above represent net bearing pressures; the weight of the footings and overlying backfill can be ignored in calculating footing sizes. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads and may be increased by one-third for short-term loads, such as wind or seismic forces.

Lateral loads on footings can be resisted by passive earth pressures on the sides of footings and by friction on the bearing surface. We recommend that passive earth pressures be calculated using an equivalent fluid weight of 325 pounds per cubic foot (pcf). We recommend using a friction coefficient of 0.35 for a silt subgrade, and 0.45 for a gravel pad subgrade. The passive earth pressure and friction components may be combined, provided that the passive component does not exceed two-thirds of the total. The lateral resistance values do not include safety factors.

We estimate that total post-construction settlements should be less than 1 inch, with differential settlement of less than 1/2 inch between columns. Construction of the second story is expected to induce less than an additional 1/2 inch of settlement.

6.1.3 Foundation Subgrade Preparation

As noted above, if isolated column footings have loads greater than 125 kips or are designed using a bearing pressure of 3,500 psf, then gravel pads are required below the footings. The gravel pad shall consist of aggregate base compacted to at least 95 percent of its maximum dry density, as discussed in *Section 5 - Earthworks Recommendations* of this report.

For isolated column footings, the gravel pads should have a thickness equal to 2 feet or 1/3 of the footings' length, whichever is greater. The gravel should extend laterally at least 1 foot beyond all edges of the footings. For example, a 4- by 8-foot isolated column footing would be underlain by a 2.7-foot thick gravel pad that is 6- by 10-foot in plan dimension.

Prior to the installation of the gravel pad (if needed) and placement of reinforcing steel in the footing excavations, all loose or disturbed soils should be removed and the footing subgrade lightly compacted with a small vibratory plate compactor. If water infiltrates and pools in the excavation, the water, along with any disturbed soil should be removed before placing the reinforcing steel. If construction is undertaken during periods of rain, we recommend that imported granular material be placed over the base of footing excavations. The granular material reduces subgrade disturbance from standing water and from foot traffic during forming and tying of reinforcing steel. Typically, 3 to 6 inches of granular material that is lightly compacted until well keyed provides sufficient protection from disturbance.

We recommend that Hart Crowser observe all foundation excavations before placement of aggregate base to determine that bearing surfaces have been adequately prepared and that the soil conditions are consistent with those observed during our explorations.

6.2 Floor Slabs

Satisfactory subgrade support for concrete slabs supporting up to 125 psf areal loading can be obtained from the new structural fills or reworked subgrade prepared in accordance with the previous recommendations presented in this report. A minimum 6-inch thick layer of crushed rock should be placed over the prepared subgrade to assist as a capillary break. Base rock material placed directly below the slab should be 3/4- to 1-inch maximum size.

Slabs should be reinforced according to their proposed use and per the structural engineer's recommendations. Load-bearing concrete slabs may be designed assuming a modulus of subgrade reaction, k , of 125 pounds per square inch per inch, provided the site is prepared as recommended in this report.

Flooring manufacturers often require vapor barriers to protect flooring and flooring adhesives. Many flooring manufacturers will warrant their product only if a vapor barrier is installed according to their recommendations. Selection and design of an appropriate vapor barrier, if needed, should be based on discussions among members of the design team.

We recommend that Hart Crowser observe slab subgrade preparation before placement of aggregate base to determine that subgrade has been adequately prepared and that the soil conditions are consistent with those observed during our explorations. We should also evaluate the compacted aggregate base to verify required compaction levels have been achieved.

6.3 Seismic Design

We anticipate that seismic design will be in accordance with the 2009 International Building Code (IBC) (International Code Council 2009). The parameters provided in Table 4 are appropriate for code-level seismic design.

Table 4 - Seismic Design Parameters

Parameter	Value
Site Class	D
Spectral Response Acceleration S_s	0.909 g
Spectral Response Acceleration S_1	0.322 g
Site Coefficient, F_a	1.136
Site Coefficient, F_v	1.756
Spectral Response Acceleration (Short Period), S_{DS}	0.688 g
Spectral Response Acceleration (1-Second Period), S_{D1}	0.377 g

7 DRAINAGE DESIGN RECOMMENDATIONS

7.1 Temporary Drainage

During mass grading at the site, the contractor should be made responsible for temporary drainage of surface water as necessary to prevent standing water and/or erosion at the working surface. During rough and finished grading of the building site, the contractor should keep all footing excavations and building pads free of water.

7.2 Surface Drainage

The finished ground surface around buildings should be sloped away from their foundations at a minimum 2 percent gradient for a distance of at least 5 feet. Downspouts or roof scuppers should discharge into a storm drain system that carries the collected water to an appropriate stormwater system. Trapped planter areas should not be created adjacent to the building without providing means for positive drainage (i.e., swales or catch basins).

7.3 Subsurface Drainage

Based on our findings, it does not appear that subsurface drainage (e.g., footing drains) will be required around building perimeters. However, if trapped planters or adverse grades are created adjacent to buildings, then the use of footing drains should be considered. The need for perimeter footing drains should be evaluated once the grading plan has been developed.

7.4 Infiltration Systems

The results of our field infiltration testing are described in *Section 3.4.5 - Infiltration Testing* of this report. We calculated unfactored coefficients of permeability of 7 and 20 inches/hour. We noted no visible differences in the tested soils that would account for the different rates; therefore, we recommend that the lower value be used for design. However, because of this conservative recommendation, we do not recommend that an additional “Soils Correction Factor” be applied in addition to the “Base Correction Factor” of 2 required by Table 6-1 of Clark County’s *Stormwater Manual* (Clark County 2009).

We did not test the upper silt layer, and infiltration into this unit should not be attempted. The bases of all infiltration systems should penetrate at least 3 feet in the sand deposits, which were encountered 3.5 to 7 feet below grade.

In summary, we recommend that infiltration system be designed to discharge stormwater into the sand deposits, and that an unfactored coefficient of permeability of 7 inches/hour be used for design. As noted above, a minimum Base Correction Factor of 2 should be applied to this value. Additional System Correction Factors may also need to be applied.

The presence of suitable soils should be confirmed by Hart Crowser during construction.

Groundwater was not encountered in our explorations and, based on regional mapping, is expected to be approximately 190 to 20 feet below grade; therefore, it should not have a significant effect on design of stormwater disposal systems.

8 PAVEMENT DESIGN RECOMMENDATIONS

8.1 General

Our pavement design recommendations include options for flexible AC and rigid PCC pavement. Our design thicknesses assume that site development occurs during a period of dry weather when the subgrade is moisture conditioned and compacted per *Section 5 - Earthworks Recommendations* of this report.

We were not provided specific traffic counts for the project, so we assumed some traffic loading criteria based on our experience with similar projects. If these and other assumptions in the following section do not appear valid, please contact our office so that updated recommendations can be developed.

8.2 Assumptions

We made the following assumptions regarding the design of the pavement:

- A 20-year design life with equivalent single-axle loads (ESALs) and heavy truck traffic, as follows:
 - Parking Lot – 20,000 ESALs (up to 250 cars per day; no heavy trucks)
 - Drive Aisles/Delivery Routes – 50,000 ESALs (up to 5 trucks per day)
- A resilient modulus of 4,500 pounds per square inch (psi) was assumed for a subgrade that has been moisture conditioned and compacted in conformance with *Section 5 - Earthworks Recommendations* of this report.
- A resilient modulus of 25,000 psi was estimated for the base rock.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Reliability and standard deviation of 85 percent and 0.45, respectively.
- Structural coefficients of 0.45 and 0.10 for the asphalt and base rock layers, respectively.
- Minimum moduli of rupture and elasticity of 550 and 3,600,000 psi, respectively, for PCC.
- Minimum compressive strength of 4,000 psi for PCC.

If these assumptions appear incorrect, then we should be contacted to re-evaluate our recommendations.

Also, construction traffic should be limited to non-building, unpaved portions of the site or haul roads. Construction traffic should not be allowed on new pavements. If construction traffic is to be allowed on newly constructed road sections, an allowance for additional traffic will need to be made in the design pavement section.

8.3 Pavement Sections

The pavement sections in Table 5 are minimum recommended material thicknesses.

Table 5 - Pavement Sections

Pavement Type/Location	AC Thickness (inches)	Aggregate Base Thickness (inches)
AC – Parking Stalls	3.0	8.0
AC – Drive Aisles	3.0	11.5
PCC – All Locations	5.0	4.0

8.4 Pavement Materials

The asphalt cement binder should be PG 64-22 Performance Grade Asphalt Cement according to WSS 9-02.1(4) – Performance Graded Asphalt Binder. The AC should be 1/2-inch hot mix asphalt with a minimum lift thickness of 1.5 inches. The AC should conform to the specifications provided in WSS 5-04 – Hot Mix Asphalt and WSS 9-03.8 – Aggregates for Hot Mix Asphalt and be compacted to 91 percent of Rice Density of the mix, as determined in accordance with ASTM D 2041.

Rigid PCC pavement should meet the specifications provided in WSS 5-05 – Cement Concrete Pavement. The PCC shall have a minimum compressive strength of 4,000 psi and nominal maximum aggregate size of 1.5 inches. The PCC should be constructed with a maximum joint spacing of 15 feet. The slabs shall be interlocked at contraction joints (e.g., continuous slab with no dowels). However, dowels should be used at construction and expansion joints.

Imported granular material used as base aggregate (base rock) should meet the criteria specified in *Section 5.5 - Structural Fills and Backfills* of this report. The base aggregate should be compacted to not less than 95 percent of the maximum dry density as determined by ASTM D 1557.

9 CONSTRUCTION OBSERVATIONS

Satisfactory foundation and earthwork performance depends to a large degree on quality of construction. Sufficient monitoring of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during subsurface explorations. Recognition of changed conditions often requires experience; therefore, Hart Crowser or their representative should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

We recommend that Hart Crowser be retained to monitor construction at the site to confirm that subsurface conditions are consistent with the site explorations and to confirm that the intent of project plans and specifications relating to earthwork and foundation construction are being met. In particular, we recommend that the foundation, building, infiltration system, and pavement subgrades; and compaction of structural fills and aggregate bases be observed and/or tested by Hart Crowser.

10 LIMITATIONS

We have prepared this report for the exclusive use of HDJ Design Group, Anderson Dabrowski Architects, and their authorized agents for the proposed VA primary care facility in Vancouver, Washington. Our work was completed in general accordance with our proposal dated September 10, 2012, and our Consultant Agreement dated February 26, 2013. Our report is intended to provide our opinion of geotechnical parameters for design and construction of the proposed project based on exploration locations that are believed to be representative of site conditions. However, conditions can vary significantly between exploration locations and our conclusions should not be construed as a warranty or guarantee of subsurface conditions or future site performance.

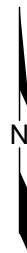
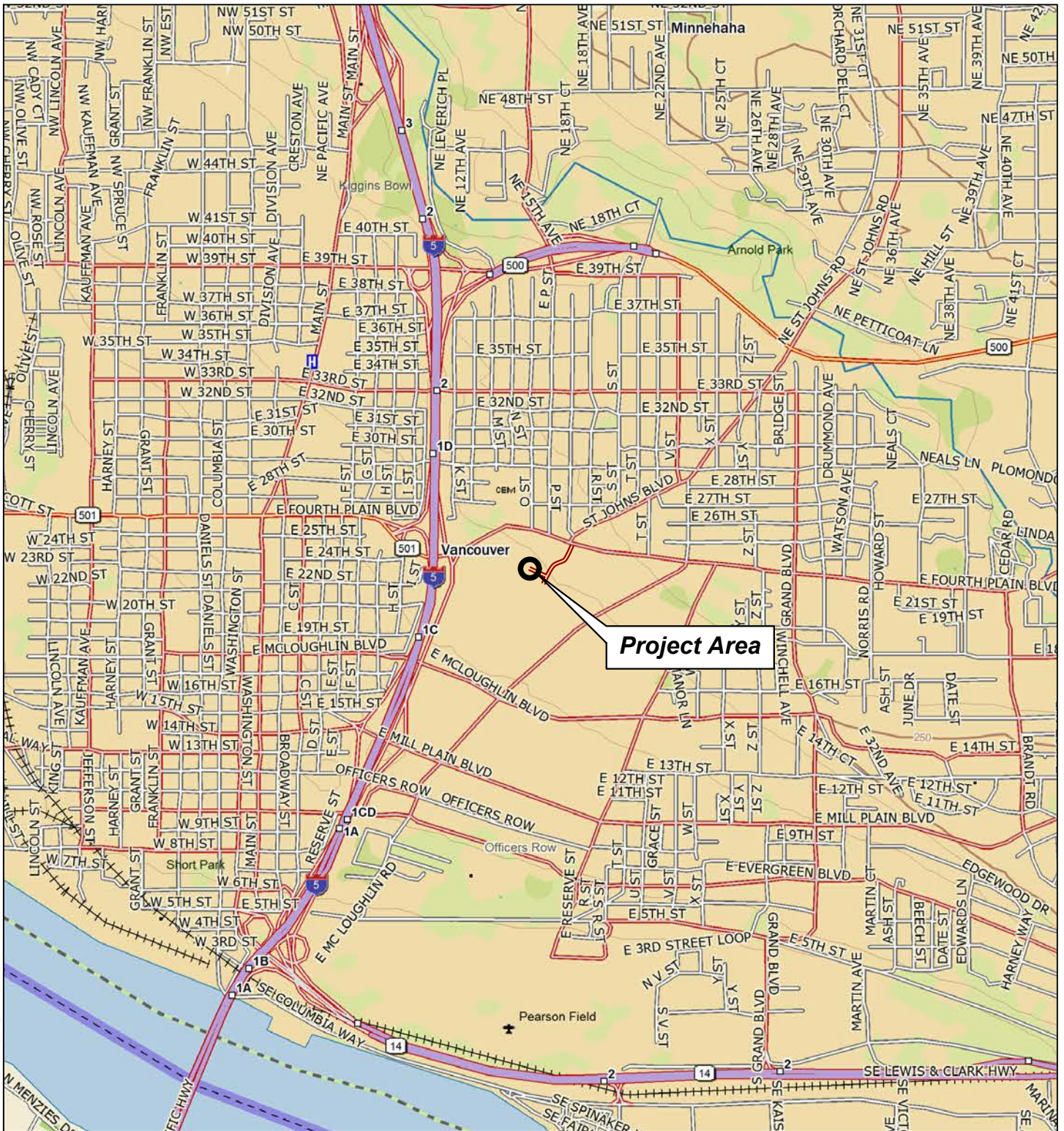
Within the limitations of scope, schedule, and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty, express or implied, should be understood.

Any electronic form, facsimile, or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by Hart Crowser and will serve as the official document of record.


12 REFERENCES

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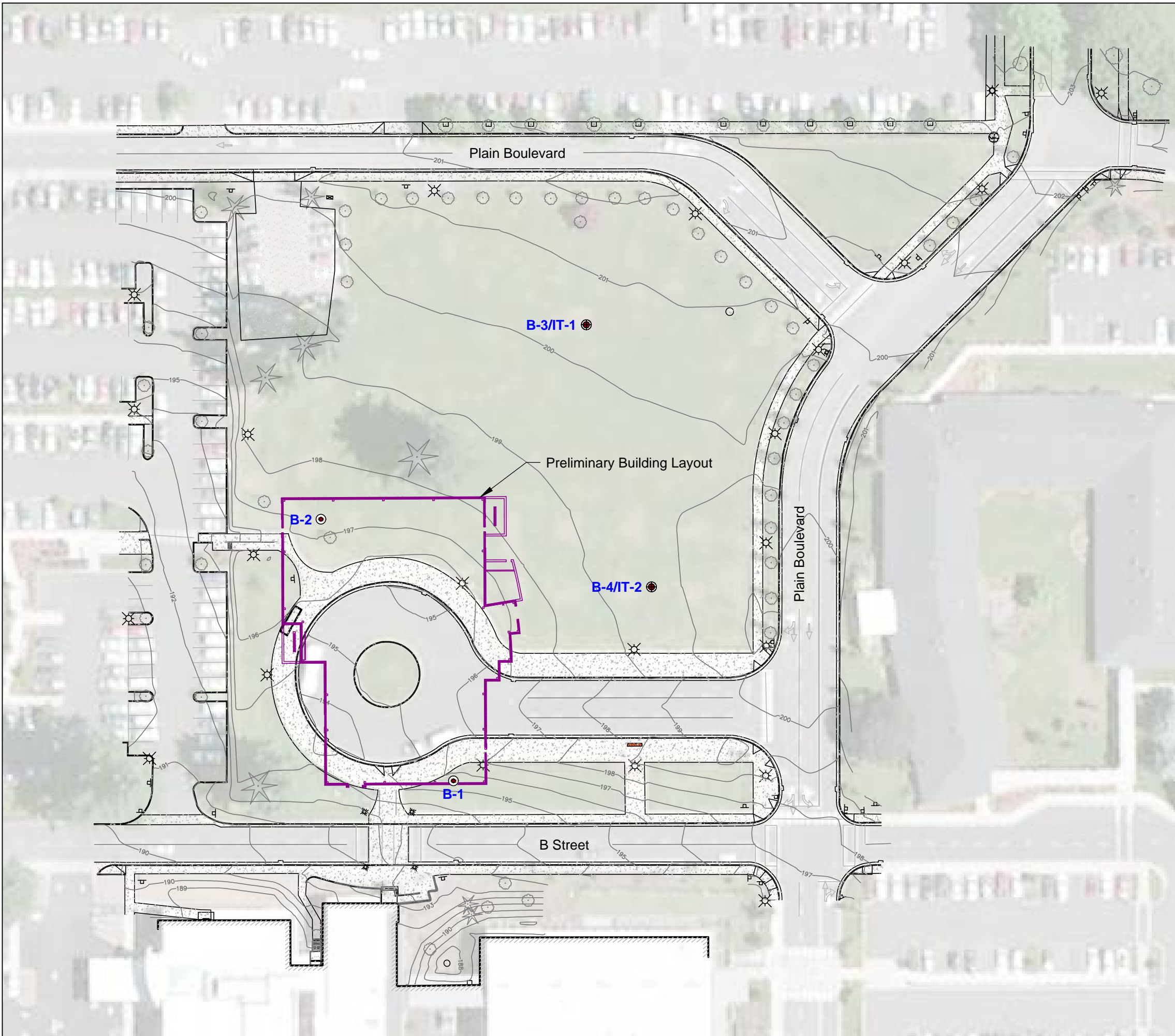
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0 2,000 4,000
Scale in Feet

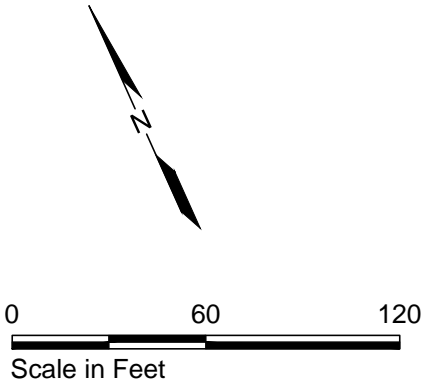
Veterans Affairs Primary Care Facility E 4th Plain Blvd and St. Johns Blvd, Vancouver, Washington	
Vicinity Map	
15949-01	4/13
	Figure 1

JAB 04/30/13 F:\Data\Jobs\15949 HDJ Design Group\01 VA Primary Care Bldg\Figures - CAD\1594901-002 (Exist Site Plan).dwg




Exploration Location and Number:

- B-2** ● Soil Boring
- B-4/IT-2** ● Soil Boring with Infiltration Test



Source: Topographic Survey provided by HDJ Design Group, dated 12/21/12.

Veterans Affairs Primary Care Facility E 4th Plain Blvd and St. Johns Blvd, Vancouver, Washington	
Site Plan	
15949-01	4/13
	Figure 2

APPENDIX A FIELD EXPLORATIONS

APPENDIX A FIELD EXPLORATIONS

General

We evaluated subsurface soil and groundwater conditions at the site by drilling four borings on March 18, 2013. The borings were advanced using a rubber-tired, truck-mounted drill rig equipped with a hollow-stem auger system and operated by Western States Drilling of Hubbard, Oregon.

The locations of the explorations are shown on Figure 2. The exploration locations were staked by Hart Crowser in the field and then surveyed by HDJ Design Group.

The field explorations were coordinated by an engineering staff member, who located the explorations, classified the various soil units encountered, obtained representative soil samples for geotechnical testing, observed and recorded groundwater conditions, and maintained a detailed log of each boring. Exploration logs are included in this appendix. Results of the laboratory testing are indicated on the exploration logs and are included in Appendix B.

Soil Sampling and Classification

Materials encountered in the borings were classified in the field in general accordance with ASTM Standard Practice D 2488 "Standard Practice for the Classification of Soils (Visual-Manual Procedure)." Soil classifications and sampling intervals are shown in the exploration logs in this appendix.

Sampling using a SPT sampler was completed in general conformance with ASTM Test Method D 1586 "Standard Method for Penetration Test and Split-Barrel Sampling of Soils." The sampler was driven with a 140-pound auto-trip hammer falling 30 inches. The N-value, or number of blows required to drive the sampler 1 foot, or as otherwise indicated, into the soils is shown adjacent to the sample symbols on the boring logs. Disturbed samples were obtained from the sampler for subsequent classification and testing.

Infiltration Test Procedures

Infiltration tests IT-1 and IT-2 consisted of falling head, encased borehole tests conducted in general accordance with the procedures referenced in Clark County's *Stormwater Manual* (Clark County 2009) and as briefly described below.



















- In IT-1 and IT-2 the infiltration test boreholes were advanced to the test depth, and the auger was withdrawn. A 3-inch PVC pipe was embedded approximately 6 inches into the base of the hole.
- The pipe was filled with several feet of water for initial saturation. In IT-1 water did not infiltrate quickly; therefore, we maintained water in the pipe for several hours to ensure saturation of the underlying soils (“pre-soaking”) prior to testing. Eventually, the base of the borehole of IT-1 was found to be disturbed with fines build-up, and the borehole was shifted several feet. In IT-2 and the relocated IT-1 the initial saturation water drained in less than 1 minute. Therefore, testing was begun immediately.
- The infiltration testing consisted of filling the pipe with several feet of water and measuring the time for the water level to drop.
- The test procedure was repeated until we obtained consistent readings.
- The collected data were used to calculate the coefficient of permeability for each test location.

Refer to *Section 3.4.5 - Infiltration Testing* and *Section 7.4 - Infiltration Systems* sections of this report for a discussion of our findings and recommendations regarding the design of infiltration systems.

KEY TO EXPLORATION LOGS



SOIL CLASSIFICATION CHART

MATERIAL TYPES	MAJOR DIVISIONS		GROUP SYMBOL	SOIL GROUP NAMES & LEGEND		<div>OTHER MATERIAL SYMBOLS</div> <div><div>Concrete</div><div>Asphalt</div><div>Topsoil</div></div>
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	GW	WELL-GRADED GRAVEL		
			GP	POORLY-GRADED GRAVEL		
		GRAVELS WITH FINES, >12% FINES	GM	SILTY GRAVEL		
			GC	CLAYEY GRAVEL		
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	SW	WELL-GRADED SAND		
			SP	POORLY-GRADED SAND		
		SANDS AND FINES >12% FINES	SM	SILTY SAND		
			SC	CLAYEY SAND		
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	CL	LEAN CLAY		
			ML	SILT		
		ORGANIC	OL	ORGANIC CLAY OR SILT		
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	CH	FAT CLAY		
			MH	ELASTIC SILT		
		ORGANIC	OH	ORGANIC CLAY OR SILT		
	HIGHLY ORGANIC SOILS			PT	PEAT	

Note: Multiple symbols are used to indicate borderline or dual classifications

MOISTURE MODIFIERS

Dry - Absence of moisture, dusty, dry to the touch
Moist - Damp, but no visible water
Wet - Visible free water or saturated, usually soil is obtained from below the water table

SEEPAGE MODIFIERS

None -
Slow - < 1 gpm
Moderate - 1-3 gpm
Heavy - > 3 gpm

CAVING MODIFIERS

None -
Minor - isolated
Moderate - frequent
Severe - general

MINOR CONSTITUENTS

Trace - < 5% (silt/clay)
Occasional - < 15% (sand/gravel)
With - 5-15% (silt/clay) in sand or gravel
15-30% (sand/gravel) in silt or clay

SAMPLE TYPES



Dames & Moore



Standard Penetration Test (SPT)



Shelby Tube



Bulk or Grab

LABORATORY/ FIELD TESTS

ATT - Atterberg Limits
CP - Laboratory Compaction Test
CA - Chemical Analysis (Corrosivity)
CN - Consolidation
DD - Dry Density
DS - Direct Shear
HA - Hydrometer Analysis
OC - Organic Content
PP - Pocket Penetrometer (TSF)
P200 - Percent Passing No. 200 Sieve
SA - Sieve Analysis
SW - Swell Test
TV - Torvane Shear
UC - Unconfined Compression

GROUNDWATER SYMBOLS



Water Level (at time of drilling)



Water Level (at end of drilling)



Water Level (after drilling)

STRATIGRAPHIC CONTACT



Distinct contact between soil strata or geologic units



Gradual or approximate change between soil strata or geologic units

Notes:

Blowcount (N) is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted) per ASTM D-1586. See exploration log for hammer weight and drop.

When the Dames & Moore (D&M) sampler was driven with a 140-pound hammer (denoted on logs as D+M 140), the field blow counts (N-value) shown on the logs have been reduced by 50% to approximate SPT N-values.

Refer to the report text and exploration logs for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the exploration locations at the time the explorations were made. The logs are not warranted to be representative of the subsurface conditions at other locations or times.

KEY TO EXPLORATION LOGS - GINT STD US LAB.GDT - 10/22/12 13:15 - F:\DATA\JOBS\17893 CHEHALIS RIVER OVERFLOW BRIDGE\GINT\1789300 BORING LOGS.GPJ



HARTCROWSER

BORING B-1

PAGE 1 OF 1

CLIENT HDJ Design Group

PROJECT NAME VA Primary Care Facility

PROJECT NUMBER 15949-01

PROJECT LOCATION Vancouver, WA

DATE STARTED 3/18/2013 COMPLETED 3/18/2013

GROUND ELEVATION _____ HOLE SIZE 8.25 inch

DRILLING CONTRACTOR Western States Soil Conservation

GROUND WATER LEVELS:

DRILLING METHOD Hollow Stem Auger

AT TIME OF DRILLING ---

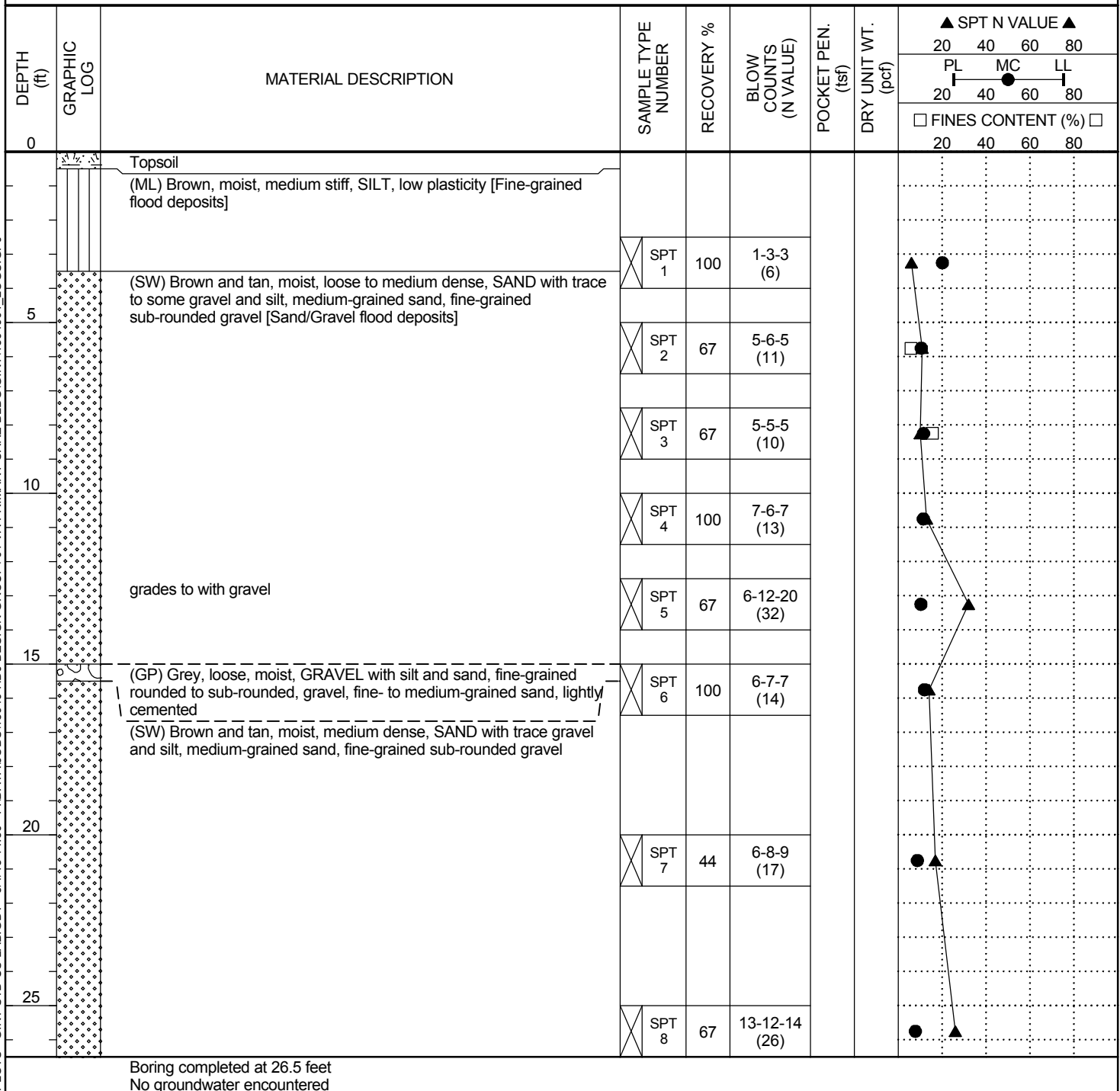
LOGGED BY M. Gummer CHECKED BY D. Trisler

AT END OF DRILLING ---

NOTE: _____

AFTER DRILLING ---

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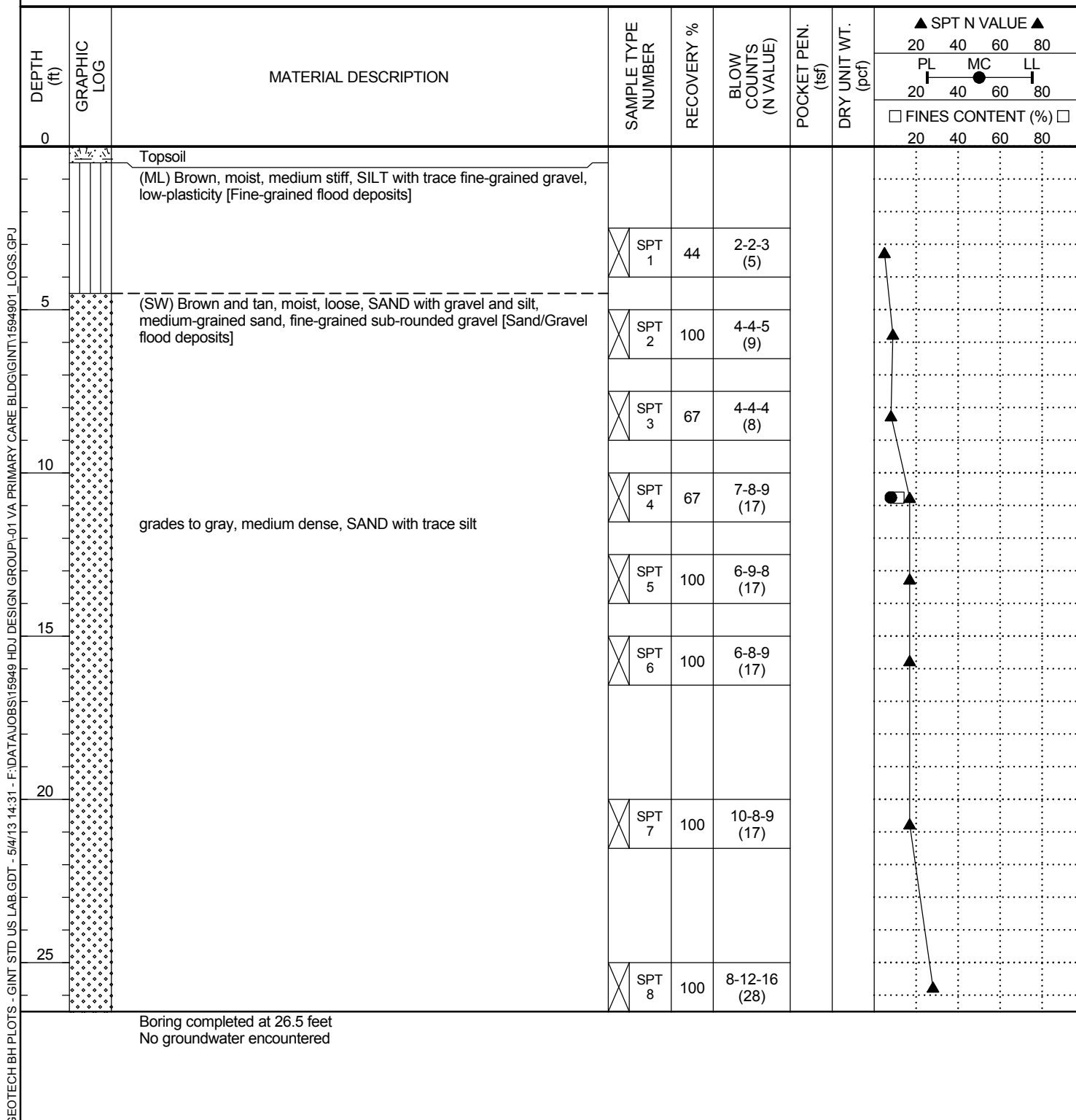


HARTCROWSER

BORING B-2

PAGE 1 OF 1

CLIENT <u>HDJ Design Group</u>	PROJECT NAME <u>VA Primary Care Facility</u>
PROJECT NUMBER <u>15949-01</u>	PROJECT LOCATION <u>Vancouver, WA</u>
DATE STARTED <u>3/18/2013</u> COMPLETED <u>3/18/2013</u>	GROUND ELEVATION _____ HOLE SIZE <u>8.25 inch</u>
DRILLING CONTRACTOR <u>Western States Soil Conservation</u>	GROUND WATER LEVELS:
DRILLING METHOD <u>Hollow Stem Auger</u>	AT TIME OF DRILLING <u>---</u>
LOGGED BY <u>M. Gummer</u> CHECKED BY <u>D. Trisler</u>	AT END OF DRILLING <u>---</u>
NOTE: _____	AFTER DRILLING <u>---</u>



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HARTCROWSER

BORING B-3 / IT-1

PAGE 1 OF 1

CLIENT	HDJ Design Group	PROJECT NAME	VA Primary Care Facility
PROJECT NUMBER	15949-01	PROJECT LOCATION	Vancouver, WA
DATE STARTED	3/18/2013	COMPLETED	3/18/2013
DRILLING CONTRACTOR	Western States Soil Conservation	GROUND ELEVATION	
DRILLING METHOD	Hollow Stem Auger	HOLE SIZE	8.25 inch
LOGGED BY	M. Gummer	CHECKED BY	D. Trisler
NOTE:			
GROUND WATER LEVELS:		AT TIME OF DRILLING ---	
		AT END OF DRILLING ---	
		AFTER DRILLING ---	

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲			
								20 40 60 80			
								PL MC LL			
								20 40 60 80			
								□ FINES CONTENT (%) □			
								20 40 60 80			
0		Topsoil									
		(ML) Brown, moist, medium stiff, SILT with some sand and trace gravel, low-plasticity, fine-grained sand, fine-grained sub-rounded gravel [Fine-grained flood deposits]	 SPT 1	100	2-2-3 (5)						
5		(SW) Brown and tan, dry to moist, loose, SAND with some gravel and silt, medium-grained sand, fine-grained sub-rounded gravel [Sand/Gravel flood deposits]	 SPT 2	100	2-4-4 (8)						
		grades to medium dense	 SPT 3	33	5-6-5 (11)						
10			 SPT 4	67	5-6-4 (10)						

Boring completed at 11.5 feet.
No groundwater encountered
Infiltration test IT-1 conducted at 6 feet.



HARTCROWSER

BORING B-4 / IT-2

PAGE 1 OF 1

CLIENT HDJ Design Group

PROJECT NAME VA Primary Care Facility

PROJECT NUMBER 15949-01

PROJECT LOCATION Vancouver, WA

DATE STARTED 3/18/2013 COMPLETED 3/18/2013

GROUND ELEVATION _____ HOLE SIZE 8.25 inch

DRILLING CONTRACTOR Western States Soil Conservation

GROUND WATER LEVELS:

DRILLING METHOD Hollow Stem Auger

AT TIME OF DRILLING ---

LOGGED BY M. Gummer CHECKED BY D. Trisler

AT END OF DRILLING ---

NOTE: _____

AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY %	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲			
								20	40	60	80
								PL	MC	LL	
0								20	40	60	80
								□ FINES CONTENT (%) □			
								20	40	60	80
		Topsoil									
		(ML) Grey to brown, moist, medium stiff to stiff, SILT with some gravel lenses, low plasticity, fine-grained sub-rounded gravel [Fine-grained flood deposits]	X SPT 1	100	5-4-4 (8)						
5			X SPT 2	22	6-7-8 (15)						
		grades to stiff to very stiff with fine-grained, sub-rounded gravel									
		(SW) Brown and tan, moist, loose to medium dense, SAND with trace to some silt and gravel, non-plastic, medium-grained sand, fine-grained sub-rounded to rounded gravel [Sand/Gravel flood deposits]	X SPT 3	100	3-4-3 (7)						
10			X SPT 4	33	6-6-5 (11)						
			X SPT 5	100	5-5-3 (8)						

Boring completed at 11.5 feet
No groundwater encountered
Infiltration test IT-2 conducted at 8 feet

APPENDIX B

LABORATORY TESTING

APPENDIX B LABORATORY TESTING

General

Soil samples obtained from the explorations were transported to our laboratory and evaluated to confirm or modify field classifications, as well as to evaluate engineering properties of the soils encountered. Representative samples were selected for laboratory testing. The tests were performed in general accordance with the test methods of the ASTM or other applicable procedures. A summary of the test results is included as Figure B-1.

Visual Classifications

Soil samples obtained from the explorations were visually classified in the field and in our geotechnical laboratory based on the USCS and ASTM classification methods. ASTM Test Method D 2488 was used to classify soils using visual and manual methods. ASTM Test Method D 2487 was used to classify soils based on laboratory test results.

Laboratory Test Results

Moisture Content

Moisture contents of samples were obtained in general accordance with ASTM Test Method D 2216. The results of the moisture content tests completed on samples from the explorations are presented on the exploration logs included in Appendix A and on Figure B-1 in this appendix.

Percent Fines

Fines content analyses were performed to determine the percentage of soils finer than the No. 200 sieve—the boundary between sand size particles and silt size particles. The tests were performed in general accordance with ASTM Test Method D 1140. The test results are indicated on the exploration logs included in Appendix A and on Figure B-1 in this appendix.

Sieve Analyses

Sieve analysis tests were performed to determine the quantitative distribution of particle sizes in the sample. The tests were performed in general accordance with ASTM Test Method D 6913 04 and D 1140. The “percent fines” portion of the test results are indicated on the appropriate exploration log included in Appendix A and on Figure B-1 in this appendix. The full test results are shown on Figure B-2 in this appendix.



HARTCROWSER

SUMMARY OF LABORATORY RESULTS

PAGE 1 OF 1

CLIENT HDJ Design Group

PROJECT NAME VA Primary Care Facility

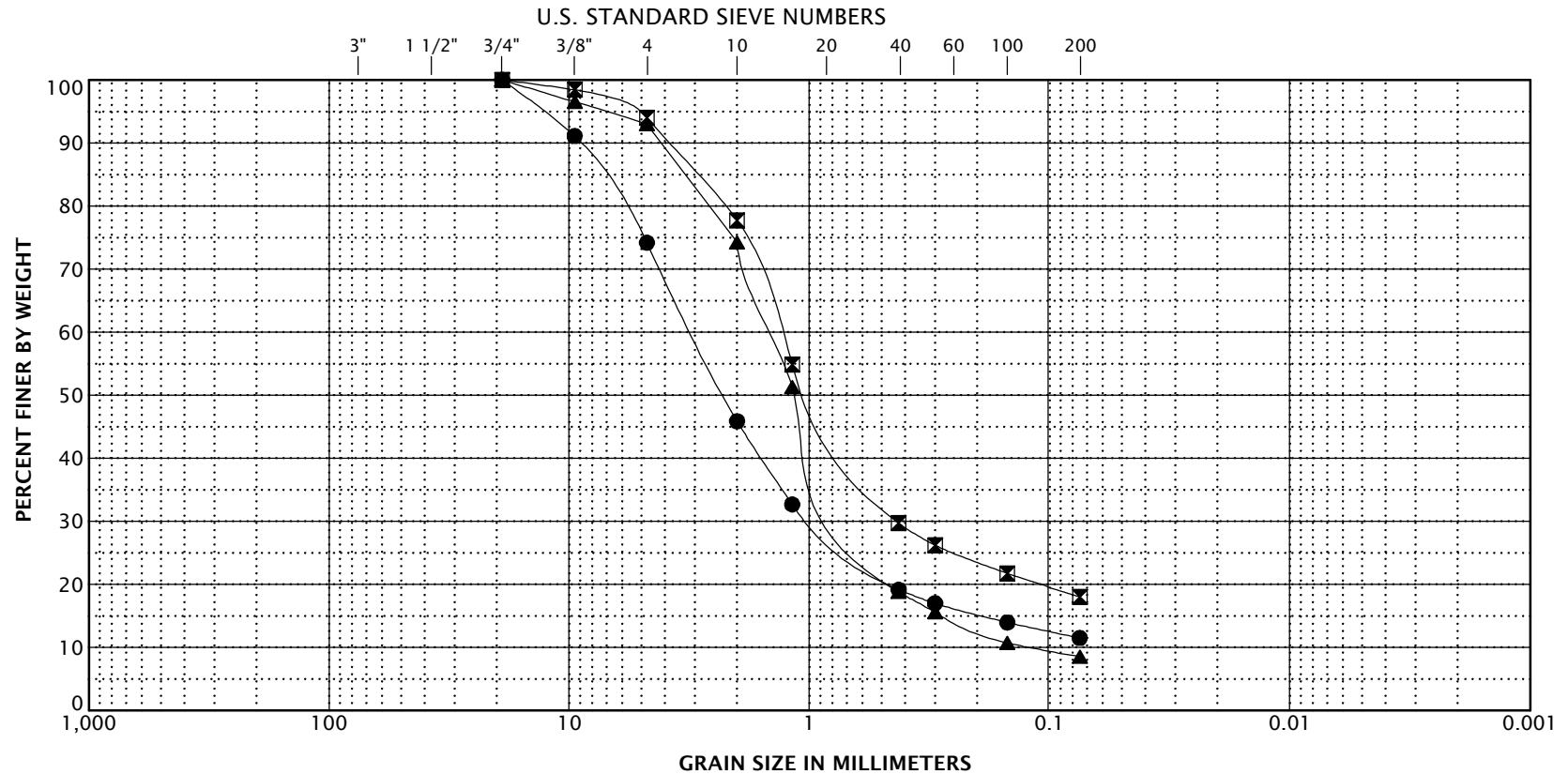
PROJECT NUMBER 15949-01

PROJECT LOCATION Vancouver, WA

Borehole	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Size (mm)	%<#200 Sieve	Classification	Water Content (%)	Dry Density (pcf)	
B-1	2.5							20.1		
B-1	5.0				0.075	6		10.5		
B-1	7.5				0.075	15		11.5		
B-1	10.0							11.4		
B-1	12.5							10.3		
B-1	15.0							11.9		
B-1	20.0							8.7		
B-1	25.0							7.8		
B-2	10.0				19	12		7.9		
B-3 / IT-1	2.5							21.2		
B-3 / IT-1	7.5				19	18		10.7		
B-4 / IT-2	7.5				19	9		11.2		
B-4 / IT-2	10.0							11.0		
B-4 / IT-2	12.5				0.075	13		9.4		

LAB SUMMARY - GINT STD US LAB.GDT - 25/3/13 13:08 - F:\DATA\JOBS\15949 HDJ DESIGN GROUP\01 VA PRIMARY CARE BLDG\GINT\1594901 LOGS.GPJ

Figure B-1



BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY

KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	D60	D50	D30	D10	D5	GRAVEL (PERCENT)	SAND (PERCENT)	SILT (PERCENT)	CLAY (PERCENT)
●	B-2	10.0	7.9	3.08	2.27	0.96			26	63	12	
▣	B-3 / IT-1	7.5	10.7	1.33	0.97	0.43			6	76	18	
▲	B-4 / IT-2	7.5	11.2	1.44	1.13	0.60	0.12		7	84	9	



15949-01

GRAIN-SIZE TEST RESULTS

VA PRIMARY CARE FACILITY
VANCOUVER, WA

FIGURE B-2