DRAFT

REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Fircrest Adult Training Program Renovation
1902 NE 150th Street
Shoreline, Washington

For
Trinity NAC
March 22, 2021

Project: NAC-1-02

March 22, 2021

Trinity NAC
2502 1st Avenue, Suite 200
Seattle, WA 98121-3131

Attention: Bill Rash, AIA, NCARB

Report of Geotechnical Engineering Services
Fircrest Adult Training Program Renovation
1902 NE 150th Street
Shoreline, Washington
Project: NAC-1-02

GeoDesign, Inc., DBA NV5 (GeoDesign) is pleased to submit our report of geotechnical engineering services for the planned improvements to the Fircrest Adult Training Program (ATP) renovation project. The project is located at the Washington State Department of Social and Health Services Fircrest facility in Shoreline, Washington. Our services for this project were conducted in accordance with our proposal dated December 9, 2020.

We appreciate the opportunity to be of continued service to you. Please call if you have questions regarding this report.

Sincerely,

GeoDesign, Inc., DBA NV5

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Kevin J. Lamb, P.E.
Principal Engineer

JTW:KJL:kt
Attachments
One copy submitted (via email only)
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ACRONYMS AND ABBREVIATIONS

AC	asphalt concrete
ASCE	American Society of Civil Engineers
ATP	Adult Training Program
ASTM	American Society for Testing and Materials
BGS	below ground surface
BMP	best management practice
CEC	cation exchange capacity
CSBC	crushed surfacing base course
g	gravitational acceleration (32.2 feet/second^2)
H:V	horizontal to vertical
IBC	International Building Code
LID	low-impact development
MCE	maximum considered earthquake
meq/100 g	milliequivalent per 100 grams
OSHA	Occupational Safety and Health Administration
PCC	portland cement concrete
pcf	pounds per cubic foot
pci	pounds per cubic inch
PIT	pilot infiltration test
psf	pounds per square foot
SFZ	Seattle fault zone
SPT	standard penetration test
WSS	Washington Standard Specifications for Road, Bridge, and Municipal Construction (2020)

1.0 INTRODUCTION

This report presents the results of GeoDesign's geotechnical investigation to support the planned improvements to the Fircrest ATP renovations project. The project is located at the Washington State Department of Social and Health Services Fircrest facility in Shoreline, Washington. The project includes interior improvements to the existing ATP building and construction of a new parking area.

The site location relative to surrounding physical features is shown on Figure 1. The proposed parking area and locations of our explorations are shown on Figure 2. The existing ATP building and locations of subsurface borings completed by others is shown on Figure 3. The logs of our explorations at the site are presented in Appendix A. The analytical laboratory report of the CEC and organic content test results is presented in Appendix B. An as-built plan for the existing ATP building, which includes logs of geotechnical borings drilled at the northeast and southeast corners of the building (borings B-3 and B-4), is presented in Appendix C.

Acronyms and abbreviations used herein are defined above, immediately following the Table of Contents.

2.0 PROJECT UNDERSTANDING

We understand the proposed improvements are generally limited to interior renovations to the existing ATP building and construction of a new parking area east of the building and on the east side of Circle Drive at the former laundry building location(Figure 2, and Appendix C). The ATP building is located west of 20th Avenue NE and the first floor of the building has been benched into the toe of an east-facing slope. The west wall of the first floor is a retaining wall. Portions of the slope meet the definitions of a geologic hazard area as defined in the City of Shoreline Municipal Code (SMC) 20.80 Critical Areas. Structural improvements and a new elevator are planned for the interior of the building and updated retaining wall parameters and seismic design parameters have been requested.

We reviewed as-built plans for the existing ATP building, which also includes logs of geotechnical borings drilled at the northeast and southeast corners of the building (presented in Appendix C). The general notes on the structural sheets indicate the foundation design is based on an allowable bearing pressure of 4,000 psf and lateral earth pressures are based on an equivalent fluid density of 30 pcf. Subsurface conditions encountered in the borings generally consist of loose to medium dense, silty sand with gravel extending to depths of 5 to 6 feet BGS underlain by glacially consolidated deposits of coarse sand and gravel to silty sand with gravel (glacial till). Groundwater, likely perched above the glacial till layer, was encountered in previous boring B-3 drilled at the northeast corner of the existing ATP building.

3.0 PURPOSE AND SCOPE

The purpose of our geotechnical engineering services was to provide geotechnical information and recommendations to support design and construction of the interior and frontage improvements and parking area as well as support evaluation of the existing retaining wall and the capacity for infiltration of stormwater below the proposed parking area. The specific scope of our services is summarized as follows:

- Reviewed existing information, including plans for the improvements and as-built plans for the existing building that include four existing borings at the existing ATP location.
- Coordinated and managed the field explorations, including public and private utility locates and scheduling of contractors and GeoDesign staff.
- Drilled one boring to a depth of 31 feet BGS and installed a monitoring well in the boring.
- Excavated three test pits to depths between 14 and 14.5 feet BGS. Completed small-scale PITs in each of the test pits at depths requested by the design team.
- Completed laboratory analysis to assist in characterization of physical parameters and water quality treatment characteristics of the soil.
- Performed engineering analysis and evaluated data derived from the subsurface investigation.
- Provided this geotechnical report that summarizes our findings and provides recommendations to support the proposed improvements.

4.0 SITE CONDITIONS

4.1 GENERAL

The site is located at the Washington State Department of Social and Health Services Fircrest facility in Shoreline, Washington. The ATP building is located west of 20th Avenue NE. The proposed parking area is located at the former location of the laundry building on the west side of 20th Avenue NE (Figure 2). Surficial conditions were determined during several visits to the site. Subsurface conditions were evaluated by reviewing existing boring logs, drilling one boring, and excavating three test pits. The soil boring and test pit explorations completed for this study were completed in the proposed parking area (Figure 2).

4.2 SURFACE CONDITIONS

The existing ATP building is located west of 20th Avenue NE and the first floor of the building has been benched into the toe of an east-facing slope. The surrounding area on the northwest and south sides of the building is relatively level. The area north of the building is s landscaped lawn area with a hardscape-surfaced area and planters and is where the geothermal well field is proposed. East the ATP building are residential units with landscaped lawn areas between them. South of the ATP building is a small AC-paved parking area. The west wall of the first floor is a retaining wall and the engineered steep slope above the retaining wall is covered with landscaping and mature evergreen trees. Concrete walkways traverse the slope northwest of the ATP building.

The proposed new parking area is located east of the ATP building on the east side of Circle Drive at the former laundry building location. The laundry building burned down in 2018 and all that remains is the concrete floor slab. Around the perimeter of the slab are AC- and gravel-covered parking areas.

4.3 SLOPES

West of the ATP building the ground surface slopes up to the upland area on the western half of the Fircrest campus. The slope varies from 20 percent to approximately 50 percent, with the steeper slope areas west of the central and southern portions of the ATP building. The vertical elevation change from the ATP building up to the top of the slope is approximately 30 feet.

The slope is well vegetated with brush and trees. Surficial indications of erosion were not observed. The slope appears stable and surficial indicators of deep or shallow slope instability were not observed.

The slope meets the City of Shoreline SMC 20.80.220 classification for Moderate to High Risk geologic hazard areas. The proposed ATP building improvements are not expected to extend into the geologic hazard area and no impacts are anticipated or will require mitigation.

4.4 SUBSURFACE CONDITIONS

4.4.1 General

Our subsurface exploration program consisted of drilling one boring (B-1) to a depth of 31 feet BGS and excavating three test pits (TP-1 through TP-3) to depths between 14 and 14.5 feet BGS. The approximate locations of our explorations are shown on Figure 2. A description of the field exploration program and the exploration logs are presented in Appendix A. We also reviewed as-built plans for the existing ATP building, which include logs of geotechnical borings drilled at the northeast and southeast corners of the building. The as-built plans with the logs are presented in Appendix C.

The test pits were completed around the perimeter of the former laundry building and the boring was completed near the center of the former building area. We encountered approximately 7 inches of concrete in the boring (existing slab) and approximately 6 inches of aggregate base in the test pits.

Fill consisting of medium dense, silty sand with gravel was encountered to depths between approximately 1 foot and 2 feet BGS.

Glacial till consisting of dense to very dense, silty sand with gravel and variable amounts of cobbles was encountered below the fill to depths between 8 and 10 feet BGS. The upper 2 to 3.5 feet of the glacial till in the test pits has been weathered and is distinguished on the logs as "weathered glacial till." It is similar in character to the underlying glacial till but is less dense due to weathering and disturbance. Based on SPT blow counts and excavation difficulty, the glacial till is generally dense to very dense and increases in density with depth.

Advance outwash, generally consisting of dense to very dense, silty sand with some gravel, is present below the glacial till at our exploration locations to the maximum depth in the test pits of 14.5 feet BGS and to 25 feet BGS in boring B-1.

For the existing ATP building, the existing borings generally indicate the subsurface conditions are loose to medium dense, silty sand with gravel extending to depths between 5 and 6 feet BGS underlying glacially consolidated deposits consisting of coarse sand and gravel to silty sand with gravel (glacial till) (Appendix C).

Environmental screening for the presence of volatile organic compounds was completed during excavation of the test pits. Odors or sheens were not noted or observed at the exploration locations.

4.4.2 Groundwater

Groundwater, likely perched above the glacial till layer, was encountered in the existing boring B-3 completed near the northeast corner of the ATP building at depths between 8 and 10 feet BGS.

In our explorations in the proposed parking area, groundwater seepage was not observed in the test pit explorations to the maximum depth explored of 14.5 feet BGS. Groundwater was encountered in boring B-1 at a depth of approximately 20 feet BGS during drilling. A 2-inch-diameter standpipe piezometer was installed in boring B-1 to monitor groundwater levels.

A data logger was installed in the well at a depth of approximately 29.5 feet BGS to record regular groundwater measurements. Depth to groundwater varied from approximately 19 to 20 feet BGS during the monitoring period that extended from February 3, 2021 through March 10, 2021. Groundwater measurements obtained from the well for the monitoring period are shown on Figure 4.

5.0 INFILTRATION TESTING

Small-scale PITs were performed in the three test pits in general accordance with the 2016 City of Shoreline Engineering Development Manual (City of Shoreline, 2016). The test pits were excavated using a mini excavator. The size of test pits was generally rectangular and approximately 2.5 feet wide by 6 feet long. The PITs were performed near the anticipated bottom of the infiltration/detention system at a depth of 8 feet BGS. Soil conditions encountered at the base of the infiltration tests consist generally of dense, silty sand with gravel glacial till (TP-1) or advance outwash (TP-2 and TP-3) material.

An electronic pressure transducer and data logger were placed in the test pits to measure groundwater levels at regular short-term intervals throughout the saturation period and during the test. The test was repeated as time and the infiltration rate permitted. Up to approximately 12 to 18 inches of water was established in the test pit during the test. The infiltration rate measured near the end of the test, which allows for the longest saturation period, is used to calculate the short-term infiltration rate, as summarized in Table 1.

Table 1. Soil Infiltration Rate Analysis

Infiltration Location	Soil Type	Test Depth (feet BGS)	Averaged Measured Short-Term Infiltration Rate (inches per hour)
TP-1	Dense, silty SAND with gravel	8	1.3
TP-2	Dense, silty SAND, trace gravel	8	2.2
TP-3	Dense, silty SAND, minor gravel	8	0.7

6.0 DESIGN RECOMMENDATIONS AND CONCLUSIONS

6.1 GENERAL

Based on our review of the proposed preliminary development plans and the results of our exploration and analyses, it is our opinion that the proposed development is geotechnically feasible. Our recommendations are provided in the following sections.

6.2 SEISMIC DESIGN CRITERIA

6.2.1 Seismicity

Washington State is situated at a convergent continental margin and is susceptible to subduction zone, intraplate, and shallow crustal source earthquakes. We reviewed published geologic maps for the site vicinity to evaluate seismic hazards. The site is approximately 10 miles north of the SFZ.

The SFZ represents a 2- to 4-mile-wide zone, extending from the Kitsap Peninsula near Bremerton to the Sammamish Plateau. Within the SFZ are several east to west-trending fault splays of the Seattle fault (Johnson et al., 1999). The Seattle fault is thought to be a reverse fault, with the south side "shoved up." The SFZ is considered an active major fault and can produce earthquakes of Magnitude ~7 with associated surface rupture and ground motions, posing a significant hazard to the Puget Sound Region (Sherrod et al., 2008). Geologic evidence indicates at least three episodes of movement on the fault within the last 10,000 years, with the most recent earthquake with surface rupture approximately 1,100 years ago (Nelson et al., 2000).

6.2.2 IBC Parameters

Boring B-1 encountered very dense, glacially consolidated soil within 2 feet of the ground surface with SPT blow counts exceeding 50 blows per foot. Similar conditions were encountered in the previous borings drilled for the ATP building and similar conditions are expected to extend to over 100 feet BGS, as confirmed in the geothermal test boring. We believe these conditions support classification of the site as Site Class C. Based on our explorations and analysis, the following design parameters can be applied if the building is designed using the applicable provisions of ASCE 7-16. The parameters in Table 2 should be used to compute seismic base shear forces (ASCE 7-16).

Table 2. ASCE 7-16 Seismic Design Parameters

Parameter	Short Period	1 Second
MCE Spectral Acceleration	S_s = 1.268 g	S_1 = 0.442 g
Site Class	C	
Site Coefficient	F_a = 1.2	F_v = 1.858
Adjusted Spectral Acceleration	S_{MS} = 1.521 g	S_{M1} = 0.664 g
Design Spectral Response Acceleration Parameters	S_{DS} = 1.014 g	S_{D1} = 0.442 g

6.2.3 Landslide Hazards

The site is relatively flat and underlain by dense/hard glacial till deposits. Landslide hazard risk for the site is very low.

6.2.4 Liquefaction

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. The excessive buildup of pore water pressure results in the sudden loss of shear strength in a soil. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate.

Based on the results of our explorations, the site in underlain by dense to very dense/hard glacial till consisting of silty sand and sandy silt. We anticipate the potential for liquefaction is very low for this site.

6.2.5 Lateral Spreading

Lateral spreading is a liquefaction-related seismic hazard and occurs on gently sloping or flat sites underlain by liquefiable sediment adjacent to an open face (such as riverbanks). Liquefied soil adjacent to an open face will tend to flow, resulting in surface cracking and lateral displacement towards the open face. The magnitude of lateral spreading decreases with distance from the open face. Based on the soil encountered at the site and distance from an open face, lateral spreading is not considered a hazard at this site.

6.2.6 Surficial Rupture

The site is approximately 10 miles north of the SFZ. The risk of surficial rupture for the site is low.

6.3 SHALLOW FOUNDATIONS

6.3.1 General

The existing ATP building foundations were design using an allowable bearing pressure of 4,000 psf based on the as-built plans. The site is underlain by dense glacial till. New foundations for upgrades within the ATP building, such as the elevator pit, and elsewhere, supported on undisturbed glacial till or outwash soil may be designed using an allowable bearing pressure of 4,000 psf. Where new foundations are located adjacent to an existing foundation, they should bear at similar bottom of foundation elevations as the existing foundations.

6.3.2 Bearing Capacity

Foundations bearing on the dense glacial till or compacted stabilization material placed over it may be sized based on an allowable bearing pressure of 4,000 psf. This is a net bearing pressure; the weight of the footing 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 those resulting from wind or seismic forces. Continuous wall and spread footings should be at least 18 inches wide. The bottom of exterior footings should be at least 18 inches below the lowest adjacent final grade. The bottom of interior footings should be placed at least 12 inches below the base of the floor slab.

6.3.3 Resistance to Sliding

Lateral loads on footings can be resisted by passive earth pressure on the sides of the foundation and by friction on the base of the footings. Passive earth pressure may be estimated using an equivalent fluid density of 350 pcf. Adjacent floor slabs, pavement, or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance. A coefficient of friction equal to 0.35 may be used when calculating resistance to sliding for footings in direct contact with the glacial till or structural fill. A safety factor of 1.5 has been applied to the recommended sliding friction and passive pressure.

6.3.4 Settlement

For foundations designed in accordance with the recommendations provided above, total post-construction settlement should be less than ½ inch and differential settlement less than ¼ inch.

6.4 FLOOR SLABS

Satisfactory subgrade support for building floor slabs supporting up to 350 psf areal loading can be obtained on subgrade that is scarified and compacted to 95 percent of the maximum dry density, as determined by ASTM D1557.

A minimum 6-inch-thick layer of crushed surfacing base course, WSS 9-03.9(3) – Crushed Surfacing, should be placed and compacted over the prepared subgrade to provide uniform support beneath the slab.

A subgrade modulus of 200 pci may be used to design the floor slab.

The near-surface soil typically has a fines content in excess of 15 percent. In areas where moisture-sensitive floor slab and flooring will be installed, the installation of a vapor barrier is warranted to reduce the potential for moisture transmission through and efflorescence growth on the slab and flooring.

6.5 RETAINING STRUCTURES

6.5.1 Conventional Below-Grade or Retaining Structures

We understand additional analysis is required to evaluate the existing retaining wall. We reviewed as-built plans for the existing ATP building. The general notes on the structural sheets indicate that lateral earth pressures for retaining wall design are based on an equivalent fluid density of 30 pcf. This value is suitable for the dense glacial till soil encountered in the boring and for walls that are free to rotate about their base. Braced walls should be designed for at-rest conditions. Additional recommendations for below-grade walls are provided below.

6.5.1.2 Wall Design Parameters

For unrestrained retaining walls, an equivalent fluid density of 30 pcf is appropriate for design assuming drained conditions and that active earth pressure conditions develop behind the wall as a result of wall deflection. Where retaining walls are restrained from rotation prior to being backfilled, an equivalent fluid density of 45 pcf should be used for design for the at-rest condition.

A superimposed seismic lateral force should be calculated based on a dynamic force of $6.5H^2$ pounds per lineal foot of wall (where H is the height of the wall in feet) and applied a distance of 0.6H above the base of the wall.

If surcharges (e.g., building foundations, vehicles, etc.) are located within a horizontal distance from the back of a wall equal to twice the height of the wall, additional pressures will need to be accounted for in the wall design. Our office should be contacted for appropriate wall surcharges based on the actual magnitude and configuration of the applied loads.

The base of the wall footing excavations should extend a minimum of 12 inches below the lowest adjacent grade and be designed in accordance with the recommendations provided in the "Shallow Foundations" section.

6.5.1.3 Wall Backfill

Backfill material placed behind retaining walls and extending a horizontal distance of ½H (where H is the height of the retaining wall) should consist of select granular material that meets the specifications provided in WSS 9-03.12(2) – Gravel Backfill for Walls. We recommend the select granular wall backfill be separated from general fill, native soil, and/or topsoil using a geotextile fabric that meets the specifications provided in WSS 9-33.2 – Geosynthetic Properties for drainage geotextiles.

Backfill should be placed and compacted as recommended for structural fill, except for backfill placed immediately adjacent to walls. Backfill adjacent to walls should be compacted to a lesser standard to reduce the potential for generation of excessive pressure on the walls. Backfill located within a horizontal distance of 3 feet from the retaining walls should be compacted to approximately 90 percent of the maximum dry density, as determined by ASTM D1557. Backfill placed within 3 feet of the wall should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (such as a jumping jack or vibratory plate compactor). If flatwork (slabs, sidewalk, or pavement) will be placed adjacent to the wall, we recommend the upper 2 feet of fill be compacted to 95 percent of the maximum dry density, as determined by ASTM D1557.

6.5.1.4 Wall Drainage

The above design parameters have been provided assuming back-of-wall drains will be installed to prevent buildup of hydrostatic pressures behind all walls. If a drainage system is not installed, our office should be contacted for revised design forces.

Positive drainage should be provided behind below-grade walls and retaining walls by placing a minimum 1-foot-wide zone of free-draining backfill directly behind the wall. The free-draining backfill should meet the criteria for WSS 9-03.12(4) – Gravel Backfill for Drains. The free-draining backfill zone should extend from the base of the wall to within 2 feet of the finished ground surface. The top 2 feet of fill should consist of relatively impermeable or native soil to prevent infiltration of surface water into the wall drainage zone.

Perforated collector pipes should be placed at the base of the walls. The pipe should be embedded in a minimum 2-foot-wide zone of drain rock. The drain rock should meet specifications provided in the "Materials" section. The drain rock should be wrapped in a geotextile fabric that meets the specifications for drainage geotextiles as described in the "Materials" section. The collector pipes should discharge at an appropriate location away from the base of the wall. Unless measures are taken to prevent backflow into the drainage system of the wall, the discharge pipe should not be tied directly into stormwater drain systems.

6.6 INFILTRATION

6.6.1 Design Infiltration Rate

As discussed in the "Subsurface Conditions" section, the soil encountered near the base of the anticipated stormwater management systems consists of dense, glacially consolidated material generally composed of silty sand with varying gravel content.

The infiltration rate determined using the PIT procedure is a short-term infiltration rate. A correction factor is necessary to account for the small scale of the test. Additional correction factors are necessary to account for testing uncertainties, site variability, and long-term reduction in permeability due to biological activity and accumulation of fines. The recommended correction factors to be applied to the "short-term" rate measured in the tests are summarized as follows:

- Correction factor $F_{testing}$ accounts for uncertainties in testing methods. A correction factor of 0.5 is typically applied to rates from small-scale PITs.
- Correction factor $F_{variability}$ accounts for site subsurface variability and the number of locations tested. We recommend a correction factor $F_{variability}$ of 0.45.
- Correction factor F_m accounts for reduction in infiltration rates over the long term due to siltation and bio-buildup. We recommend a correction factor of 0.9.

The total correction factor to be applied is obtained by multiplying the individual correction factors. A cumulative correction factor of 0.20 should be applied to the measured infiltration rate. Table 3 summarizes the infiltration test results along with the correction factor.

Table 3. Soil Infiltration Rate Analysis[1]

Infiltration Location	Soil Type	Averaged Short-Term Infiltration Rate (inches per hour)	Recommended Long-Term Design Infiltration Rate[1] (inches per hour)
TP-1	Dense, silty SAND with gravel	1.3	0.26
TP-2	Dense, silty SAND, trace gravel	2.2	0.44
TP-3	Dense, silty SAND, minor gravel	0.7	0.14

1. Based on the recommended combined correction factor of 0.20.

We recommend the facility in the proposed parking area be designed using an average long-term infiltration rate of 0.25 inch per hour.

6.6.2 Soil Suitability for Treatment

CEC and organic content testing were also completed on samples collected at the base of the test pits to evaluate soil capacity for water quality treatment. Our subcontracted laboratory, AMTest Laboratories, performed the testing. The test results are presented in Appendix B and the results are summarized in Table 4.

Table 4. CEC and Organic Content Analytical Results Summary[1]

Exploration	Sample Depth (feet BGS)	Soil Type	CEC (meq/100 g)	Organic Content (percent)
TP-1	8	Dense, silty SAND with gravel	1.8	1.2
TP-2	1	Dense, silty SAND, trace gravel	1.0	0.7
TP-3	1	Dense, silty SAND, minor gravel	1.5	0.8

1. Suitability for Water Quality Treatment: CEC greater than 5 meq/100 g and organic content a minimum of 1.0 percent

The results of the tests indicate that the CEC for the soil at a depth of 8 feet BGS is typically less than 2 meq/100 g, which is less than the required 5 meq/100 g. The organic content of the soil ranges between 0.7 and 1.2 percent, with an average value of 0.9 percent, which is less than the 1 percent required for water quality treatment.

Based on the available test results, soil amendment will be necessary to address water quality treatment.

6.6.3 Groundwater Separation

We anticipate the depth of LID infiltration elements will be approximately 8 feet BGS. Stormwater Standards require a minimum of 5 feet of separation between the bottom of infiltration facilities or areas and groundwater. Groundwater measurements in the monitoring well on site indicate that 10-feet of separation exists.

6.7 GEOLOGIC HAZARD AREAS

As discussed in the “Slopes” section, portions of the slope west and south of the ATP building meet the City of Shoreline SMC 20.80.220 classification for Moderate to High Risk geologic hazard areas. Indications of instability were not observed in the areas and the proposed work is expected to be outside of the geologic critical area. The building is located along the toe of the slope and the proposed work will not impact existing slope stability nor impact adjacent properties.

Soil in the area generally meets the classification of “severe” erosion hazard, particularly on slopes that exceed 15 percent. The temporary increase in erosion hazard during construction, due to activities that disturb the ground surface, can be mitigated through appropriate BMPs such as stabilized construction entrances and haul roads, silt fencing, and straw wattles and by placing sediment socks in catch basins. The appropriate BMPs should be maintained after the site is restored while the permanent landscaping or surface finishes become established.

7.0 CONSTRUCTION

The proposed parking area was previously developed and what remains is a concrete floor slab surrounded with gravel or AC pavement hardscape areas. Earthwork site preparation activities will include removing the existing PCC floor slab and surrounding AC. It should include removal of previously installed utilities or foundation elements to avoid variations in subgrade consistency.

The soil to be exposed during grading operations has a high fines content, is moisture sensitive, and will deteriorate rapidly in wet weather where left exposed. If earthwork construction is expected to extend into the wet season, we recommend stabilizing exposed areas with a 12-inch-thick layer of CSBC material.

During excavation of the test pits, spoils were monitored for volatile organic compounds. Although no odors or sheens, indicating contamination, were detected, the previous development history and use as a laundry facility should be considered and impacted soil may be encountered.

7.1 SUBGRADE VERIFICATION

Exposed subgrades should be evaluated by a representative from GeoDesign to verify conditions are as anticipated and will provide the required support. Subgrade evaluation should be performed by probing with a foundation probe beneath foundations. If soft or loose zones are identified, these areas should be excavated to the extent indicated by the engineer or technician and replaced with structural fill or stabilization material.

7.2 EXCAVATION

7.2.1 General

The soil at the site can be excavated with conventional earthwork equipment. Excavations should stand vertical to a depth of approximately 4 feet, provided groundwater seepage is not observed in the trench walls.

Open excavation techniques may be used to excavate utility trenches with depths greater than 4 feet, provided the walls of the excavation are cut at appropriate cut slopes determined by the contractor. Approved temporary shoring is recommended where sloping is not possible. If a conventional shield is used, the contractor should limit the length of open trench. If shoring is used, we recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the overall plan of operation and the subsurface conditions. All excavations should be made in accordance with applicable OSHA, local, and state regulations.

7.2.2 Temporary Slopes

Based on soil conditions encountered during our explorations, temporary slopes for excavations of 1.25H:1V may be used to vertical depths of 15 feet or less, provided groundwater seepage is not significant, groundwater remains below the base of the excavation, surcharge loads are not present within 10 feet of the top of the slope, and the slopes are observed by the geotechnical engineer on a regular basis during construction. At this inclination, the slopes may ravel and require some on-going repair.

If seepage is encountered, it may be necessary to flatten the slopes to protect the surface from raveling or provide dewatering. All cut slopes should be protected from erosion by covering them with plastic sheeting or other stabilizing cover during the rainy season. If sloughing or instability is observed, the slope may need to be flattened or the cut supported by shoring.

Excavations should not undermine adjacent utilities, foundations, walkways, streets, or other hardscapes unless special shoring or underpinned support is provided. Unsupported excavations should not be conducted within a downward and outward 1H:1V projection starting at least 10 feet outside the edge of an adjacent structural feature.

7.2.3 Dewatering

Shallow excavations (less than 5 feet) may encounter limited seepage from perched water. In our opinion, significant dewatering operations will not likely be necessary. Dewatering systems are best designed by the contractor; however, it is our opinion that it should be possible to remove groundwater encountered by pumping from a sump. More intense use of pumps may be required at certain times of the year and where more intense seepage occurs. Removed water should be routed to a suitable discharge point.

If significant groundwater is present at the base of utility excavations, we recommend placing up to 6 inches of stabilization material at the base of the excavation.

7.3 MATERIALS

Fill material will be required for site grading, backfilling over-excavations, pavement support, installation of utilities, and drainage. Recommended fill materials are discussed below.

7.3.1 General

All material used as structural fill should be free of organic material or other unsuitable materials and (except where modified below) have a maximum particle size of 3 inches. A brief characterization of some of the acceptable material and our recommendations for their use as structural fill are provided below.

7.3.2 On-Site Soil

The on-site material encountered in our explorations has a high fines content, is sensitive to changes in moisture content, and will deteriorate under construction traffic and/or when exposed to wet weather. Although the on-site material does not meet the gradation requirements for imported structural fill, as defined below, we anticipate that some of the on-site material identified as silty sand with gravel can be used for fill but will be limited to use during the dry season and it will require moisture conditioning prior to use.

Deleterious material (such as wood, organics, and man-made material) should be removed from native soil prior to use as fill. The use of on-site soil as fill should be subject to review and approval by GeoDesign. It will be prudent to provide a 12-inch-thick cap of imported structural fill over areas where on-site soil is exposed or used as fill.

When used as structural fill, the on-site soil should be placed in lifts with a maximum uncompacted thickness of 10 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

7.3.3 Imported Granular Material

Structural fill placed for general site grading in improved areas should consist of clean, free-draining granular soil (sand and gravel) that is free from organic material or other deleterious and man-made materials, with a maximum particle size of 3 inches and a maximum fines content of 5 percent by dry weight passing the U.S. Standard No. 200 sieve. The use of granular, free-draining material will increase the workability of the material during the wet season and the likelihood that the material can be placed and adequately compacted.

Imported granular material used for structural fill should be naturally occurring pit- or quarry-run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in WSS 9-03.14(1) – Gravel Borrow, with the exception that the percentage passing the U.S. Standard No. 200 sieve does not exceed 5 percent by dry weight. Structural fill should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

7.3.4 Stabilization Material

Stabilization material used to backfill over-excavations below structures should consist of imported shot rock, quarry spalls, or crushed ballast. The material should have a maximum particle size of 6 inches, should have less than 5 percent by dry weight passing the U.S. Standard

No. 4 sieve, and should have at least two mechanically fractured faces. The material should be free of organic material and other deleterious materials. Materials that meet the specifications provided in WSS 9-13.7(2) – Backfill for Rock Wall, WSS 9-13.1(5) – Quarry Spalls, or WSS 9-27.3(6) – Stone are generally acceptable for use. Stabilization material should be placed in lifts between 12 and 18 inches thick and compacted to a firm condition with the bucket of an excavator.

7.3.5 Drain Rock

Drain rock used in subsurface drains or against retaining walls should consist of granular material with a maximum particle size of 1 inch and should meet the specifications provided in WSS 9-03.12(4) – Gravel Backfill for Drains. The material should be free of roots, organic material, and other unsuitable materials; should have less than 2 percent by dry weight passing the U.S. Standard No. 200 sieve (washed analysis); and should have at least two mechanically fractured faces.

7.3.6 Floor Slab and Pavement Base Rock

Imported granular material used as aggregate base for floor slabs, pavement, and beneath hardscape areas should consist of 1½-inch-minus material meeting the specifications provided in WSS 9-03.9(3) – Crushed Surfacing, Base Course, with the exception that the aggregate should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve and at least two mechanically fractured faces. It should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

7.3.7 Retaining Wall Select Backfill

Retaining wall select backfill should consist of well-graded sand or gravel with not more than 5 percent by dry weight passing the U.S. Standard No. 200 sieve and meeting WSS 9-03.12(2) – Gravel Backfill for Walls. Retaining wall backfill should be compacted in accordance with recommendations provided in the "Wall Backfill" section.

7.3.8 Geotextiles

7.3.8.1 Separation and Drainage Geotextile

We recommend using a non-woven geotextile drainage material around subsurface drains to separate drain rock from adjacent materials. The geotextile should conform to the specifications for non-woven separation material provided in WSS 9-33.2(1) – Geotextile Properties, Table 3 Geotextile for Separation or Soil Stabilization. A suitable non-woven material meeting these recommendations is Tencate Mirafi 160N.

8.0 OBSERVATION OF CONSTRUCTION

Recommendations provided in this report assume that GeoDesign will be retained to provide geotechnical consultation and observation services during construction. Satisfactory earthwork and foundation performance depends to a large degree on the quality of construction. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions requires experience with the site conditions and an understanding of the geotechnical recommendations; therefore, GeoDesign personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated and to verify that the work is completed in accordance with the construction drawings and specifications.

Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings, project specifications, and our recommendations.

We recommend that GeoDesign be retained to observe all earthwork activities, including the following:

- Excavation activities
- Subgrade preparation prior to fill placement or foundation construction
- Placement and compaction of fill, including fill placed in utility trenches, around buried structures, and around the stormwater management system
- Laboratory compaction and field moisture-density tests

9.0 LIMITATIONS

We have prepared this report for use by Trinity NAC and the design and construction team for the proposed development. The data and report can be used for bidding or estimating purposes, but our report, conclusions, and interpretations should not be construed as warranty of the subsurface conditions and are not applicable to other sites.

Exploration observations indicate soil conditions only at specific locations and only to the depths penetrated. They do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during excavation and construction, re-evaluation will be necessary.

The site development plans and design details were preliminary at the time this report was prepared. When the design has been finalized and if there are changes in the site grades or location, configuration, design loads, or type of construction, the conclusions and recommendations presented may not be applicable. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in this report for consideration in design.

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

♦ ♦ ♦

We appreciate the opportunity to be of continued service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

GeoDesign, Inc., DBA NV5

DRAFT

Kevin J. Lamb, P.E.
Principal Engineer

REFERENCES

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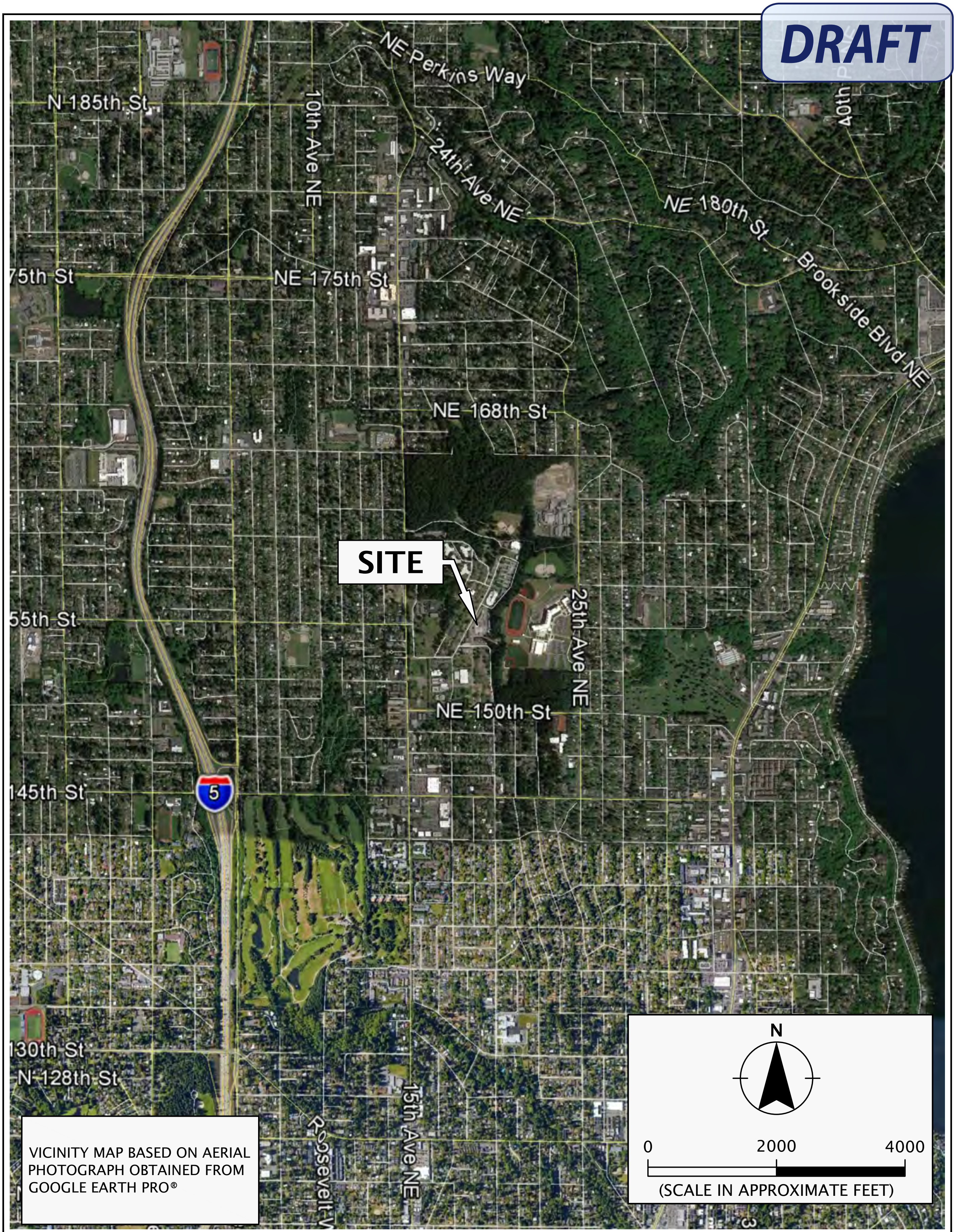
FIGURES



VICINITY MAP BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®

GEODESIGN INC AN NV5 COMPANY	NAC-1-02	VICINITY MAP	
	MARCH 2021	FIRCREST ATP RENOVATION SHORELINE, WA	FIGURE 1

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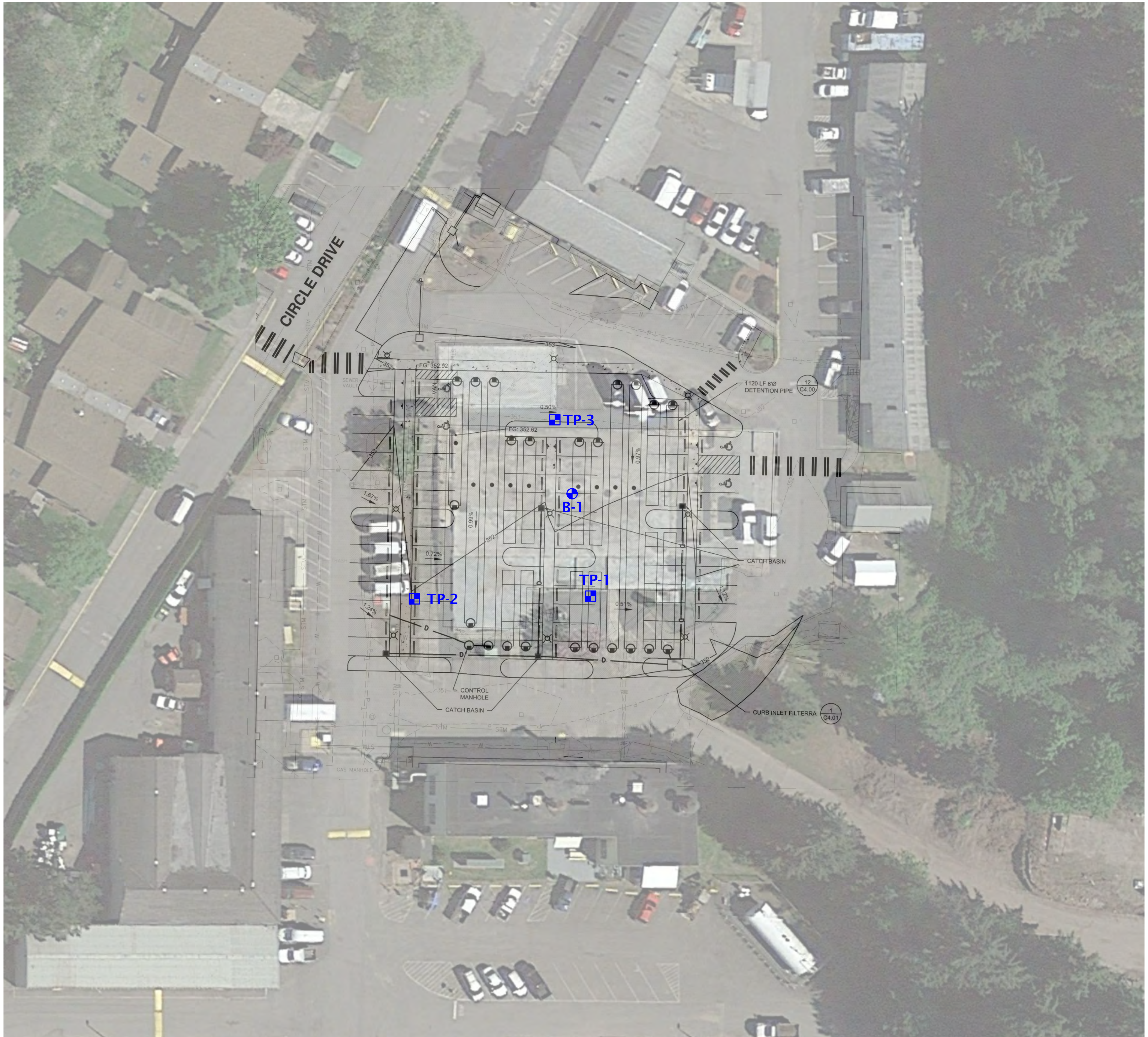
File Name: J:\M-R\NAC\NAC-1\NAC-1-02\Figures\CAD\NAC-1-02-VM01.dwg | Layout: FIGURE 1

DRAFT



LEGEND:

— — — —	SITE BOUNDARY
B-1	BORING
TP-1	TEST PIT

N

0 50 100

(SCALE IN FEET)

NOTES:

1. SITE PLAN BASED ON IMAGE OF SHEET C1.01 GRADING AND DRAINAGE PLAN EAST DATED NOVEMBER 11, 2020 PREPARED BY NAC ARCHITECTURE.
2. AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO FEBRUARY 2, 2021.

FIGURE 2

SITE PLAN
PROPOSED PARKING AREA

FIRCREST ATP RENOVATION
SHORELINE, WA

NAC-1-02

MARCH 2021

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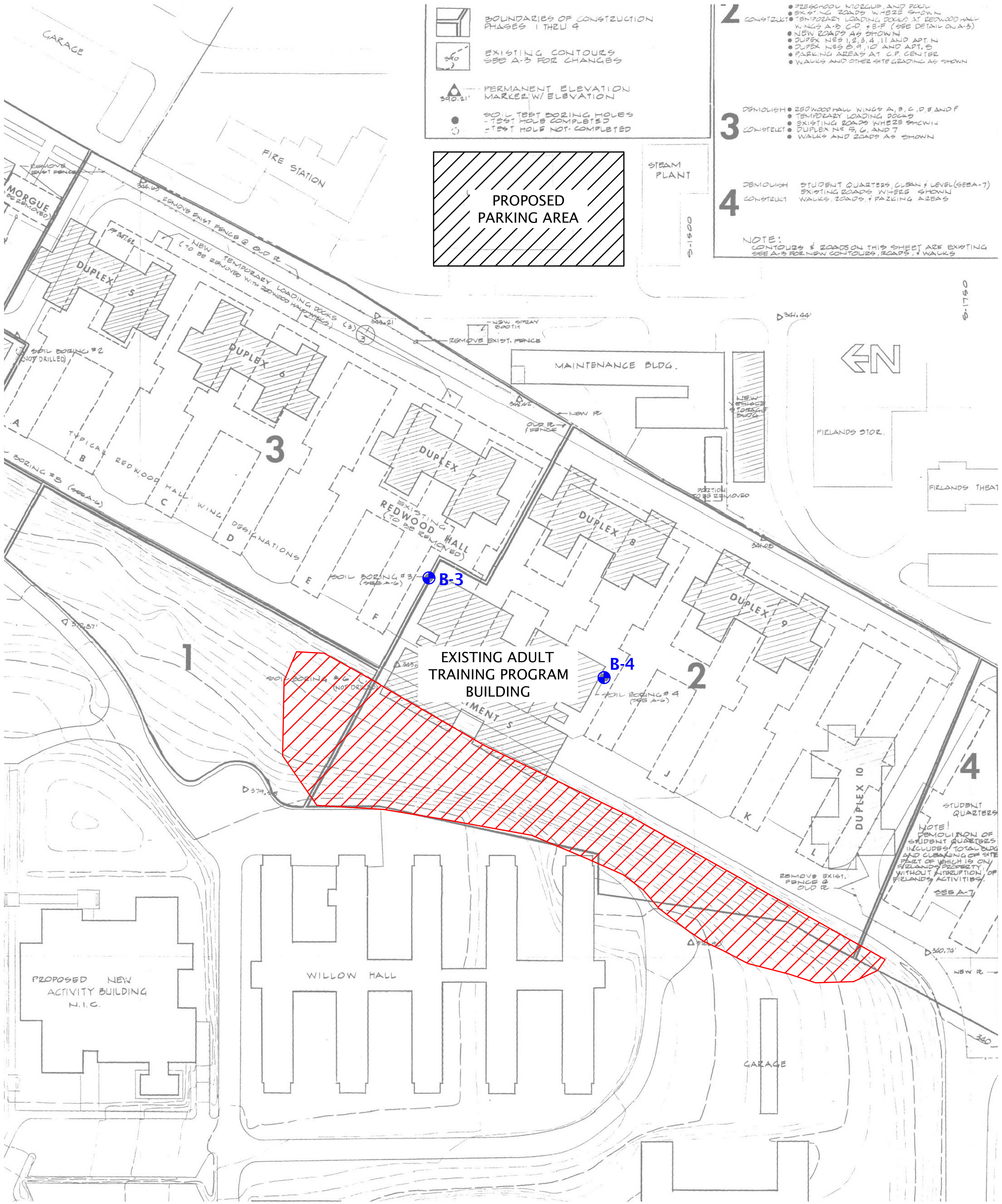

LEGEND:

MODERATE TO HIGH LANDSLIDE HAZARD AREA

B-3 EXISTING BORING DRILLED BY OTHERS

(NOT TO SCALE)

NOTE:

1. SITE PLAN BASED ON IMAGE OF SHEET A2 DATED JULY 1, 1970 PREPARED BY SHAVEY & SCHMIDT A.I.A., ARCHITECTS.

FIGURE 3

SITE PLAN
EXISTING ADULT TRAINING PROGRAM BUILDING

FIRCREST ATP RENOVATION
SHORELINE, WA

NAC-1-02

MARCH 2021

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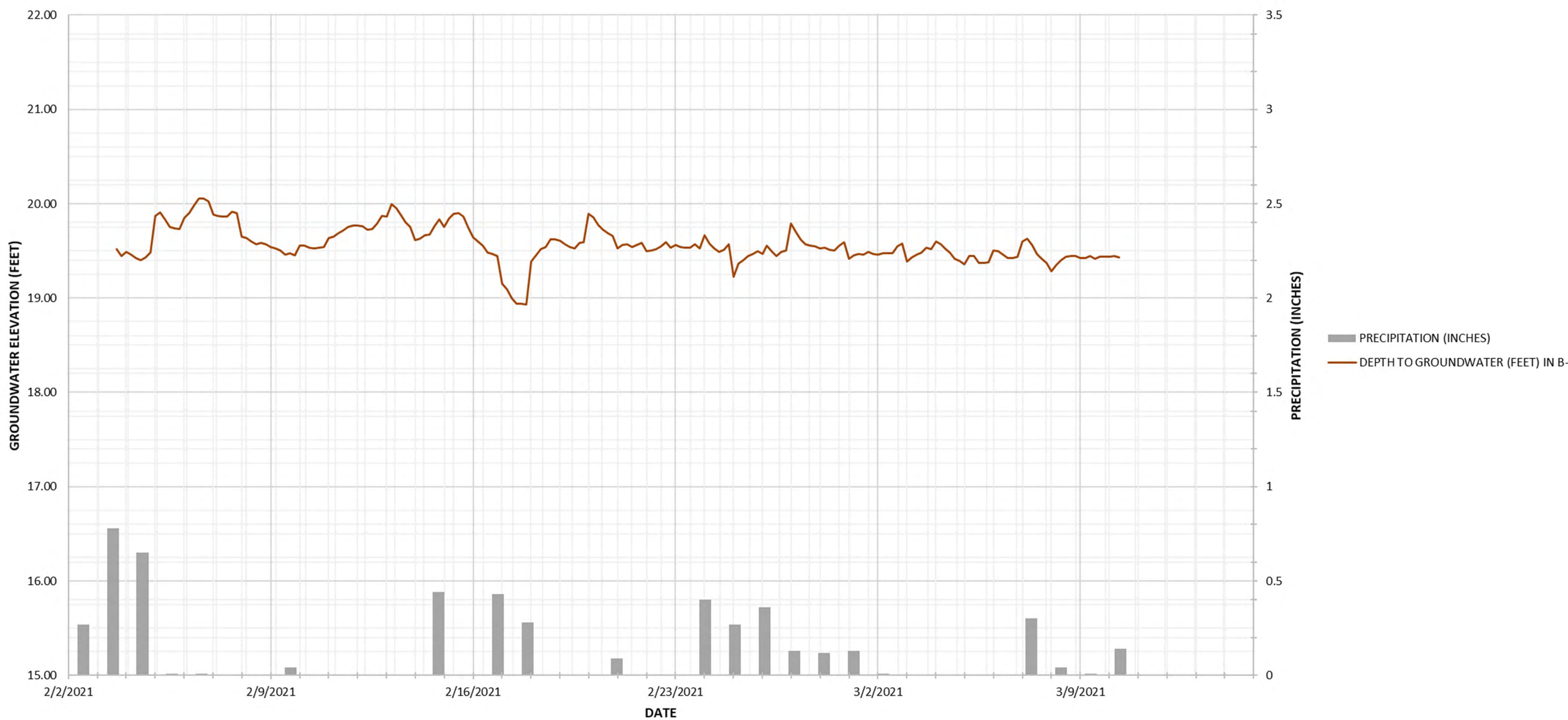

FIGURE 4

GROUNDWATER MEASUREMENTS

FIRCREST ATP RENOVATION
SHORELINE, WA

NAC-1-02

MARCH 2021

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APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

Subsurface conditions at the site were explored by drilling one boring (B-1) to a depth of 31 feet BGS on January 25, 2021 and excavating three test pits (TP-1 through TP-3) to depths of up to 14.5 feet BGS on January 19, 2021. The boring was drilled by Boretec1 using hollow-stem auger drilling methods. The test pits were completed by Continental Dirt Contractors using a Komatsu PC88 rubber-tracked excavator. The exploration logs are presented in this appendix.

The approximate locations of our explorations are shown on Figure 2. The exploration locations were selected based on our project understanding communicated by the client and adjusted based on accessibility and avoidance of existing underground utilities. This information should be considered accurate only to the degree implied by the methods used.

SOIL SAMPLING

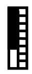
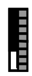
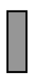
A member of our geotechnical staff observed the explorations. We collected disturbed and relatively undisturbed soil samples from the explorations for geotechnical laboratory testing.

We collected samples from the borings using 1½-inch-inside diameter, split-spoon sampler in general accordance with ASTM D1586. We used a 140-pound hammer free-falling 30 inches to drive the split-spoon samplers into the soil a total distance of 18 inches. We recorded on the exploration logs the number of blows required to drive the sampler the final 12 inches, unless otherwise noted. Representative grab samples of the soils observed in the test pit explorations were collected from the walls and/or base of the test pits using the excavator bucket. Sampling methods and intervals are shown on the exploration logs.

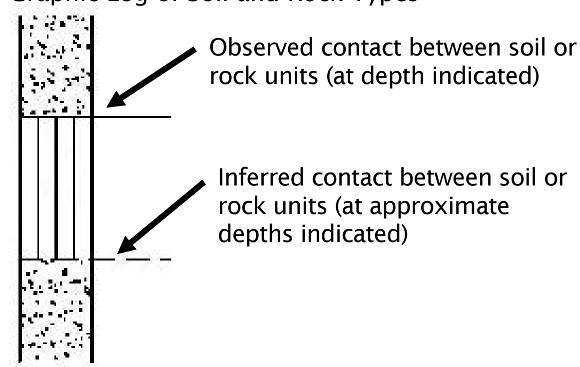
The average efficiency of the automatic SPT hammer used by Boretec1 was 91.9 percent. The calibration testing results are presented at the end of this appendix.

SOIL CLASSIFICATION

The soil samples were classified in accordance with the "Exploration Key" (Table A-1) and "Soil Classification System" (Table A-2), which are presented in this appendix. The exploration logs indicate the depths at which the soils or their characteristics change, although the change could be gradual. A horizontal line between soil types indicates an observed (visual or excavation resistance) change. If the change occurred between sample locations and was not observed or obvious, the depth was interpreted, and the change is indicated using a dashed line. Classifications are shown on the exploration logs.

SYMBOL	SAMPLING DESCRIPTION
	Location of sample collected in general accordance with ASTM D1586 using Standard Penetration Test with recovery
	Location of sample collected using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D1587 with recovery
	Location of sample collected using Dames & Moore sampler and 300-pound hammer or pushed with recovery
	Location of sample collected using Dames & Moore sampler and 140-pound hammer or pushed with recovery
	Location of sample collected using 3-inch-O.D. California split-spoon sampler and 140-pound hammer with recovery
	Location of grab sample
	Rock coring interval
	Water level during drilling
	Water level taken on date shown

Graphic Log of Soil and Rock Types



GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits	P	Pushed Sample
CBR	California Bearing Ratio	PP	Pocket Penetrometer
CON	Consolidation	P200	Percent Passing U.S. Standard No. 200 Sieve
DD	Dry Density		
DS	Direct Shear	RES	Resilient Modulus
HYD	Hydrometer Gradation	SIEV	Sieve Gradation
MC	Moisture Content	TOR	Torvane
MD	Moisture-Density Relationship	UC	Unconfined Compressive Strength
NP	Non-Plastic	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
P	Pushed Sample	NS	No Visible Sheen
PID	Photoionization Detector Headspace Analysis	SS	Slight Sheen
		MS	Moderate Sheen
ppm	Parts per Million	HS	Heavy Sheen

RELATIVE DENSITY - COARSE-GRAINED SOIL

Relative Density	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)
Very Loose	0 – 4	0 – 11	0 – 4
Loose	4 – 10	11 – 26	4 – 10
Medium Dense	10 – 30	26 – 74	10 – 30
Dense	30 – 50	74 – 120	30 – 47
Very Dense	More than 50	More than 120	More than 47

CONSISTENCY - FINE-GRAINED SOIL

Consistency	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)	Unconfined Compressive Strength (tsf)
Very Soft	Less than 2	Less than 3	Less than 2	Less than 0.25
Soft	2 – 4	3 – 6	2 – 5	0.25 – 0.50
Medium Stiff	4 – 8	6 – 12	5 – 9	0.50 – 1.0
Stiff	8 – 15	12 – 25	9 – 19	1.0 – 2.0
Very Stiff	15 – 30	25 – 65	19 – 31	2.0 – 4.0
Hard	More than 30	More than 65	More than 31	More than 4.0

PRIMARY SOIL DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOIL (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (< 5% fines)	GW or GP	GRAVEL
		GRAVEL WITH FINES (≥ 5% and ≤ 12% fines)	GW-GM or GP-GM	GRAVEL with silt
			GW-GC or GP-GC	GRAVEL with clay
		GRAVEL WITH FINES (> 12% fines)	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND (50% or more of coarse fraction passing No. 4 sieve)	CLEAN SAND (<5% fines)	SW or SP	SAND
		SAND WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SAND WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOIL (50% or more passing No. 200 sieve)	SILT AND CLAY	Liquid limit less than 50	ML	SILT
			CL	CLAY
			CL-ML	silty CLAY
			OL	ORGANIC SILT or ORGANIC CLAY
		Liquid limit 50 or greater	MH	SILT
			CH	CLAY
			OH	ORGANIC SILT or ORGANIC CLAY
HIGHLY ORGANIC SOIL			PT	PEAT

MOISTURE CLASSIFICATION

Term	Field Test
dry	very low moisture, dry to touch
moist	damp, without visible moisture
wet	visible free water, usually saturated

ADDITIONAL CONSTITUENTS

Secondary granular components or other materials such as organics, man-made debris, etc.

Percent	Silt and Clay In: Fine-Grained Soil	Silt and Clay In: Coarse-Grained Soil	Percent	Sand and Gravel In: Fine-Grained Soil	Sand and Gravel In: Coarse-Grained Soil
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	Indicate %

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DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	CONCRETE (7.0 inches)			Flush-mount monument with 1.5 feet of concrete backfill
	Medium dense, gray-brown, silty SAND with gravel and cobbles (SM); moist - **FILL**.	0.6		
	Very dense, gray-brown, silty SAND with gravel (SM); moist - **GLACIAL TILL**.	2.0		Bentonite chips
2.5			50-50/5"	
5.0			27-50/6"	2-inch, Schedule 40 PVC well casing
7.5			81	
10.0	Dense, gray-brown, silty SAND with gravel (SM); moist - **ADVANCE OUTWASH**.	10.0	46	
12.5	very dense, with cobbles at 12.5 feet		50/6"	
				12/20 filter pack sand
15.0	dense, without cobbles at 15.0 feet		32	
				2-inch, Schedule 40 PVC screen, 0.010-inch slot width
17.5				
20.0				◁ 20.0 feet, during drilling

GRAPHIC LOG | TESTING | SAMPLE

▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC%

0 50 100

DRILLED BY: Boretec1 LOGGED BY: R. Hilal COMPLETED: 01/25/21

BORING METHOD: hollow-stem auger (see document text) BORING BIT DIAMETER: 8 inches

GEODESIGN INC AN NV5 COMPANY

NAC-1-02

MARCH 2021

BORING B-1

FIRCREST ATP RENOVATION
SHORELINE, WA

FIGURE A-1

BORING LOG - GDI-NV5 - 1 PER PAGE NAC-1-02-B1-TP1_3.GPJ GDI_NV5.GDT PRINT DATE: 3/22/21:MGL:KT

DRAFT

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100)	INSTALLATION AND COMMENTS
20.0		very dense; moist to wet at 20.0 feet				50	
22.5							
25.0		Very dense, gray-brown, silty SAND with gravel (SM); moist to wet, layers of interbedded sandy SILT - **GLACIAL TILL**.	25.0			73	
27.5							
30.0						29-50/6"	6-inch, threaded cap DOE I.D Well #BJI196
		Exploration completed at a depth of 31.0 feet. Hammer efficiency factor is 91.9 percent.	31.0				Surface elevation was not measured at the time of exploration.
32.5							
35.0							
37.5							
40.0							

DRILLED BY: Boretec1 LOGGED BY: R. Hilal COMPLETED: 01/25/21

BORING METHOD: hollow-stem auger (see document text) BORING BIT DIAMETER: 8 inches

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MARCH 2021

BORING B-1
(continued)

FIRCREST ATP RENOVATION
SHORELINE, WA

FIGURE A-1

DRAFT

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		AGGREGATE BASE (6.0 inches).					
		Medium dense, brown, silty SAND with gravel (SM); moist - **FILL**.	0.5		☒		Minor caving observed from 1.0 to 3.0 feet.
2.5		Medium dense, light brown SAND with silt and gravel (SP-SM); moist - **WEATHERED GLACIAL TILL**.	1.5		☒		
5.0		Dense, light gray, silty SAND with gravel (SM); moist - **GLACIAL TILL**.	5.0		☒		Infiltration test at 8.0 feet.
7.5							
10.0		Dense, gray-brown, silty SAND with gravel (SM); moist to wet - **ADVANCE OUTWASH**.	10.0		☒		
12.5							
15.0		Exploration completed at a depth of 14.5 feet.	14.5				No groundwater seepage observed to the depth explored. Surface elevation was not measured at the time of exploration.
17.5							
20.0						0 50 100	

EXCAVATED BY: Continental Dirt Contractors — LOGGED BY: R. Hilal — COMPLETED: 01/19/21

EXCAVATION METHOD: excavator (see document text)

GEODESIGN INC — AN NV5 COMPANY

NAC-1-02 — MARCH 2021

TEST PIT TP-1

FIRCREST ATP RENOVATION
SHORELINE, WA

FIGURE A-2

DRAFT

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		AGGREGATE BASE (6.0 inches)					
		Dense, brown, silty SAND with gravel (SM); moist - **FILL**.	0.5		☒		
2.5		Dense, light brown SAND with silt and gravel (SP-SM); moist - **WEATHERED GLACIAL TILL**.	1.5		☒		Minor caving observed from 2.0 to 4.0 feet.
		medium dense at 4.0 feet					
5.0		Dense, light gray, silty SAND with gravel (SM); moist - **GLACIAL TILL**.	5.0				
7.5		Dense, gray-brown, silty SAND (SM), trace gravel; moist to wet - **ADVANCE OUTWASH**.	8.0		☒		Infiltration test at 8.0 feet.
10.0							
12.5					☒		
15.0		Exploration completed at a depth of 14.5 feet.	14.5				No groundwater seepage observed to the depth explored. Surface elevation was not measured at the time of exploration.
17.5							
20.0							

0 50 100

EXCAVATED BY: Continental Dirt Contractors LOGGED BY: R. Hilal COMPLETED: 01/19/21

EXCAVATION METHOD: excavator (see document text)

GEODESIGN INC AN NV5 COMPANY	NAC-1-02	**TEST PIT TP-2**	
	MARCH 2021	FIRCREST ATP RENOVATION SHORELINE, WA	**FIGURE A-3**

DRAFT

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		AGGREGATE BASE (6.0 inches).					
		Medium dense, brown, silty SAND with gravel (SM), trace organics; moist - **FILL**.	0.5		☒		
		Medium dense, light brown, silty SAND with gravel (SM); moist - **WEATHERED GLACIAL TILL**.	1.0		☒		
2.5							
		Dense, gray, silty SAND with gravel and cobbles (SM); moist - **GLACIAL TILL**.	3.0				
5.0							
7.5							
		Dense, gray-brown, silty SAND (SM), minor gravel; moist to wet - **ADVANCE OUTWASH**.	8.0		☒		Infiltration test at 8.0 feet.
10.0							
12.5					☒		
		Exploration completed at a depth of 14.0 feet.	14.0				No groundwater seepage observed to the depth explored. No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
15.0							
17.5							
20.0						0 50 100	

EXCAVATED BY: Continental Dirt Contractors | LOGGED BY: R. Hilal | COMPLETED: 01/19/21

EXCAVATION METHOD: excavator (see document text)

GEODESIGN INC AN NV5 COMPANY	NAC-1-02	**TEST PIT TP-3**
	MARCH 2021	FIRCREST ATP RENOVATION SHORELINE, WA — **FIGURE A-4**

Pile Dynamics, Inc. Page 1
Case Method & iCAP® Results PDIPLOT2 2014.2.48.0 - Printed 09-October-2015

BORETEC-1-01 - B-1 20FT TRACK RIG EC-95
OP: WMN Date: 05-October-2015

AR: 1.41 in^2 SP: 0.492 k/ft^3
LE: 24.10 ft EM: 30,000 ksi
WS: 16,807.9 f/s JC: 0.00 []

ETR: Energy Transfer Ratio
EMX: Max Transferred Energy
FVP: Force/Velocity proportionality
CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
CSB: Compression Stress at Bottom
BPM: Blows per Minute
VMX: Maximum Velocity
FMX: Maximum Force

BL#	depth	BLC	ETR	EMX	FVP	CSX	CSI	CSB	BPM	VMX	FMX
	ft	bl/ft	(%)	k-ft	[]	ksi	ksi	ksi	bpm	f/s	kips
8	20.00	10	93.9	0.3	0.6	25.5	25.9	0.00	56	16.575	35.9
9	20.10	10	88.4	0.3	0.6	25.7	25.9	0.00	57	16.655	36.2
10	20.20	10	90.3	0.3	0.5	25.1	25.4	0.00	54	16.925	35.5
11	20.31	10	93.3	0.3	0.6	25.8	26.2	0.00	56	16.634	36.4
12	20.41	10	86.7	0.3	0.7	26.2	26.7	0.00	56	15.753	36.9
13	20.51	10	82.5	0.3	0.7	26.6	26.6	0.00	55	16.138	37.5
14	20.61	10	89.3	0.3	0.6	23.7	24.7	0.00	55	17.030	33.4
15	20.71	10	84.5	0.3	0.6	23.5	24.9	0.00	56	16.260	33.1
16	20.82	10	86.3	0.3	0.6	23.6	24.5	0.00	56	17.248	33.3
17	20.92	10	88.3	0.3	0.5	26.7	27.1	0.00	55	17.003	37.7
18	21.02	10	90.6	0.3	0.6	26.2	26.2	0.00	56	17.131	36.9
19	21.12	10	90.4	0.3	0.6	25.3	25.6	0.00	56	17.131	35.6
20	21.22	10	91.4	0.3	0.5	24.9	25.6	0.00	57	17.302	35.2
21	21.33	10	92.2	0.3	0.6	25.3	25.7	0.00	55	17.584	35.7
22	21.43	10	86.0	0.3	0.6	24.4	24.5	0.00	56	16.086	34.4
23	21.53	10	89.3	0.3	0.7	25.5	25.7	0.00	55	16.791	35.9
24	21.63	10	90.2	0.3	0.6	24.1	24.5	0.00	55	16.422	34.0
25	21.73	10	82.8	0.3	0.6	24.5	25.8	0.00	56	16.371	34.5
26	21.84	10	84.9	0.3	0.5	24.6	25.6	0.00	56	16.824	34.7
27	21.94	10	91.7	0.3	0.6	25.9	26.9	0.00	55	17.706	36.5
28	22.04	10	89.4	0.3	0.6	25.6	26.9	0.00	56	17.223	36.1
29	22.14	10	84.9	0.3	0.6	25.6	26.7	0.00	56	17.130	36.1
30	22.24	10	88.0	0.3	0.5	25.4	25.9	0.00	56	17.228	35.8
31	22.35	10	89.6	0.3	0.5	26.6	27.3	0.00	55	16.948	37.5
32	22.45	10	89.9	0.3	0.6	25.8	26.5	0.00	56	17.692	36.3
33	22.55	10	92.1	0.3	0.6	25.5	26.0	0.00	55	17.539	36.0
34	22.65	10	91.0	0.3	0.6	24.7	25.2	0.00	55	16.685	34.8
35	22.76	10	90.5	0.3	0.6	25.3	25.7	0.00	57	17.032	35.7
36	22.86	10	91.5	0.3	0.6	25.2	26.7	0.00	56	17.461	35.5
37	22.96	10	87.3	0.3	0.6	25.3	26.8	0.00	56	17.394	35.6
38	23.06	10	89.6	0.3	0.6	24.3	24.9	0.00	56	16.758	34.2
39	23.16	10	90.6	0.3	0.5	24.6	25.9	0.00	56	17.063	34.7
40	23.27	10	86.8	0.3	0.7	25.8	27.4	0.00	56	16.481	36.3
41	23.37	10	88.6	0.3	0.6	25.3	26.9	0.00	55	17.395	35.6
42	23.47	10	88.6	0.3	0.6	25.9	27.2	0.00	57	17.135	36.5
43	23.57	10	91.0	0.3	0.5	26.9	27.7	0.00	56	17.975	37.9
44	23.67	10	89.3	0.3	0.6	25.2	27.0	0.00	56	17.433	35.6
45	23.78	10	84.0	0.3	0.6	26.0	28.4	0.00	55	17.365	36.7
46	23.88	10	87.6	0.3	0.5	25.8	27.7	0.00	57	17.391	36.4
47	23.98	10	88.2	0.3	0.6	26.3	28.6	0.00	56	17.397	37.1
48	24.08	10	84.0	0.3	0.6	26.5	28.4	0.00	55	17.536	37.3
57	25.00	10	103.1	0.4	0.5	27.4	31.1	0.00	55	19.420	38.6
58	25.28	4	87.0	0.3	0.5	27.0	30.5	0.00	56	19.110	38.1
59	25.56	4	87.8	0.3	0.7	26.0	29.5	0.00	55	17.983	36.7
60	25.83	4	85.1	0.3	0.5	26.6	30.9	0.00	57	18.220	37.5

BORETEC-1-01 - B-1 20FT
OP: WMN

TRACK RIG EC-95
Date: 05-October-2015

BL#	depth	BLC	ETR	EMX	FVP	CSX	CSI	CSB	BPM	VMX	FMX
	ft	bl/ft	(%)	k-ft	[]	ksi	ksi	ksi	bpm	f/s	kips
61	26.11	4	86.9	0.3	0.5	26.8	30.5	0.00	56	19.337	37.7
62	26.39	4	89.7	0.3	0.5	27.4	30.3	0.00	55	19.459	38.6
63	26.67	4	88.1	0.3	0.5	27.3	30.9	0.00	56	19.451	38.4
64	26.94	4	88.6	0.3	0.5	26.0	29.3	0.00	56	18.572	36.7
65	27.22	4	95.3	0.3	0.4	26.8	30.2	0.00	56	19.280	37.7
66	27.50	4	92.5	0.3	0.5	26.8	30.7	0.00	56	19.630	37.8
67	27.78	4	87.7	0.3	0.5	27.1	30.7	0.00	56	19.739	38.3
68	28.06	4	88.9	0.3	0.4	27.2	30.3	0.00	55	19.655	38.3
75	30.00	4	89.0	0.3	0.6	30.6	33.9	0.00	56	17.359	43.1
76	30.15	7	97.9	0.3	0.6	30.3	32.7	0.00	55	17.416	42.8
77	30.29	7	92.1	0.3	0.6	31.6	36.1	0.00	56	17.529	44.6
78	30.44	7	93.9	0.3	0.6	31.8	36.1	0.00	55	17.712	44.9
79	30.59	7	98.1	0.3	0.6	30.2	32.9	0.00	56	17.551	42.5
80	30.74	7	94.5	0.3	0.6	31.7	36.3	0.00	55	17.653	44.7
81	30.88	7	97.0	0.3	0.6	31.2	35.2	0.00	56	17.942	43.9
82	31.03	7	97.9	0.3	0.6	31.4	36.1	0.00	55	18.048	44.2
83	31.18	7	92.5	0.3	0.6	31.4	35.6	0.00	57	17.902	44.2
84	31.32	7	95.6	0.3	0.8	28.9	31.3	0.00	55	16.198	40.7
85	31.47	7	95.9	0.3	0.5	31.0	35.3	0.00	55	17.957	43.7
86	31.62	7	100.9	0.4	0.6	31.7	36.0	0.00	55	17.571	44.7
87	31.76	7	95.5	0.3	0.6	31.4	35.6	0.00	57	17.729	44.3
88	31.91	7	101.4	0.4	0.7	30.5	34.1	0.00	55	17.089	43.0
89	32.06	7	95.1	0.3	0.6	32.1	36.4	0.00	56	17.729	45.2
90	32.21	7	93.3	0.3	0.6	31.0	35.1	0.00	55	17.368	43.7
91	32.35	7	94.1	0.3	0.6	31.4	35.1	0.00	56	17.399	44.3
92	32.50	7	93.8	0.3	0.5	30.8	34.8	0.00	56	17.457	43.4
93	32.65	7	96.8	0.3	0.5	31.0	35.6	0.00	55	17.609	43.7
94	32.79	7	96.3	0.3	0.6	31.2	34.7	0.00	56	17.409	43.9
95	32.94	7	96.4	0.3	0.6	31.6	35.8	0.00	55	17.378	44.6
96	33.09	7	96.6	0.3	0.6	31.1	34.4	0.00	55	17.449	43.8
97	33.24	7	93.4	0.3	0.6	32.0	35.7	0.00	55	17.550	45.2
98	33.38	7	94.6	0.3	0.6	30.9	34.1	0.00	56	16.940	43.6
99	33.53	7	93.2	0.3	0.6	32.0	35.8	0.00	56	17.159	45.1
109	35.00	7	86.5	0.3	0.7	31.0	32.9	0.00	56	16.563	43.7
110	35.15	7	99.0	0.3	0.6	30.5	31.1	0.00	55	17.381	42.9
111	35.29	7	95.2	0.3	0.7	30.6	31.8	0.00	58	17.089	43.2
112	35.44	7	89.1	0.3	0.7	30.8	32.1	0.00	56	16.936	43.5
113	35.59	7	94.9	0.3	0.7	31.7	33.0	0.00	56	17.320	44.8
114	35.74	7	87.5	0.3	0.7	31.2	33.3	0.00	57	17.189	44.0
115	35.88	7	101.6	0.4	0.7	30.9	32.3	0.00	55	17.022	43.6
116	36.03	7	92.3	0.3	0.6	32.1	34.8	0.00	56	17.318	45.2
117	36.18	7	108.0	0.4	0.5	31.1	32.8	0.00	55	17.741	43.8
118	36.32	7	98.2	0.3	0.7	29.5	32.4	0.00	56	16.443	41.6
119	36.47	7	97.7	0.3	0.6	30.4	33.2	0.00	56	17.030	42.9
120	36.62	7	102.1	0.4	0.7	28.8	30.1	0.00	56	17.030	40.7
121	36.76	7	99.7	0.3	0.6	29.3	32.6	0.00	56	16.624	41.3
122	36.91	7	95.3	0.3	0.6	29.1	32.2	0.00	56	16.424	41.1
123	37.06	7	87.6	0.3	0.7	28.1	29.9	0.00	56	16.646	39.6
124	37.21	7	96.3	0.3	0.8	26.9	27.6	0.00	55	15.949	38.0
125	37.35	7	96.5	0.3	0.6	29.1	30.5	0.00	55	17.299	41.1
126	37.50	7	93.3	0.3	0.6	29.5	30.9	0.00	56	17.425	41.6
		Average	91.9	0.3	0.6	27.9	30.1	0.00	56	17.398	39.4
		Std. Dev.	4.9	0.0	0.1	2.7	3.8	0.00	1	0.833	3.8

BORETEC-1-01 - B-1 20FT
OP: WMN

TRACK RIG EC-95
Date: 05-October-2015

BL#	depth	BLC	ETR	EMX	FVP	CSX	CSI	CSB	BPM	VMX	FMX
	ft	bl/ft	(%)	k-ft	[]	ksi	ksi	ksi	bpm	f/s	kips

Total number of blows analyzed: 96

BL#	Sensors
8-48	F3: [SPT B2] 218.9 (1.00); F4: [SPT B1] 217.8 (1.00); A3: [K0035] 295.0 (1.00); A4: [K5175] 354.0 (1.00)
57-126	F3: [SPT B1] 217.8 (1.00); F4: [SPT B2] 218.9 (1.00); A3: [K5175] 354.0 (1.00); A4: [K0035] 295.0 (1.00)

BL#	Comments
48	N: 6, 11, 28
57	LE = 29.20 ft; WC = 16,778.2 f/s
68	N: 7, 7, 6
75	LE = 34.30 ft; WC = 16,781.6 f/s
99	N: 5, 11, 14
109	LE = 39.30 ft; WC = 16,731.7 f/s
126	N: 8, 8, 10

Time Summary

Drive	43 seconds	2:08 PM - 2:09 PM (10/5/2015) BN 8 - 48
Stop	14 minutes 9 seconds	2:09 PM - 2:23 PM
Drive	11 seconds	2:23 PM - 2:23 PM BN 57 - 68
Stop	12 minutes 12 seconds	2:23 PM - 2:36 PM
Drive	25 seconds	2:36 PM - 2:36 PM BN 75 - 99
Stop	11 minutes 49 seconds	2:36 PM - 2:48 PM
Drive	18 seconds	2:48 PM - 2:48 PM BN 109 - 126

Total time [00:39:50] = (Driving [00:01:38] + Stop [00:38:11])

APPENDIX B

APPENDIX B

LABORATORY TESTING

CEC

CEC tests were completed by AMTest Laboratories in Kirkland, Washington, to help assess the suitability of on-site soil for water quality treatment.

ORGANIC CONTENT

Organic content tests were completed by AMTest Laboratories in Kirkland, Washington, to help assess the suitability of on-site soil for water quality treatment.

Am Test Inc.
13600 NE 126TH PL
Suite C
Kirkland, WA 98034
(425) 885-1664
www.amtestlab.com

Professional Analytical Services

ANALYSIS REPORT

GeoDesign, Inc.
19201 120TH AVE NE
BOTHELL, WA 98011
Attention: ROBBIE HILAL
Project Name: FIRCREST ATP RENNOVATION
Project #: NAC_1_02
PO Number: NAC_1_02
All results reported on an as received basis.

Date Received: 01/22/21
Date Reported: 2/10/21

AMTEST Identification Number **21-A000954**
Client Identification **TP-1 S-3 W8'**
Sampling Date **01/19/21**

Conventionals

PARAMETER	RESULT	UNITS	Q	D.L.	METHOD	ANALYST	DATE
Cation Exchange Capacity	1.8	meq/100g		0.5	SW-846 9081	JDR	02/01/21

Miscellaneous

PARAMETER	RESULT	UNITS	Q	D.L.	METHOD	ANLST	DATE
Organic Matter	1.2	%			SM 2540G	DM	01/25/21

AMTEST Identification Number **21-A000955**
Client Identification **TP-2 S-3 W8'**
Sampling Date **01/19/21**

Conventionals

PARAMETER	RESULT	UNITS	Q	D.L.	METHOD	ANALYST	DATE
Cation Exchange Capacity	1.0	meq/100g		0.5	SW-846 9081	JDR	02/01/21

Miscellaneous

PARAMETER	RESULT	UNITS	Q	D.L.	METHOD	ANLST	DATE
Organic Matter	0.7	%			SM 2540G	DM	01/25/21

AMTEST Identification Number **21-A000956**
Client Identification **TP-3 S-3 W8'**
Sampling Date **01/19/21**

Conventionals

PARAMETER	RESULT	UNITS	Q	D.L.	METHOD	ANALYST	DATE
Cation Exchange Capacity	1.5	meq/100g		0.5	SW-846 9081	JDR	02/01/21

Miscellaneous

PARAMETER	RESULT	UNITS	Q	D.L.	METHOD	ANLST	DATE
Organic Matter	0.8	%			SM 2540G	DM	01/25/21

Kathy Fugiel
President

AmTest Chain of Custody Record

AMTEST LABORATORIES

13600 NE 126th PL, Suite C, Kirkland, WA 98034
Ph (425) 885-1664 Fx (425) 820-0245
www.amtestlab.com

Chain of Custody No.

Client Name & Address: GeoDesign Inc. (NV5 company) 19201 120th Ave NE #201 Bothell, WA 98011	Invoice To: SAME
Contact Person: Robbie Hild	Invoice Contact: Robbie Hild
Phone No: (206)459-4792	PO Number: Job NAC-1-02
Fax No:	Invoice Ph/Fax: (206)459.479
E-mail: robbie.hild@NV5.com	Invoice E-mail: robbie.hild@NV5.com
Report Delivery: (Choose all that apply) Mail / Fax / Email / Posted Online	Data posted to online account: YES / NO Web Login ID:

Special Instructions:

Requested TAT: **(Rush must be pre-approved by lab)** Standard RUSH (5 Day / 3 Day / 48 HR / 24 HR) — Temperature upon Receipt: 11.2

Project Name: Fircrest ATP Remodel

Project Number: NAC-1-02

AmTest ID	Client ID (35 characters max)	Date Sampled	Time Sampled	Matrix	No. of containers	VEC	organic matter							QA/QC
954	TP-1 S-3 @ 8'	1/19/21	AM											
55	TP-2 S-3 @ 8'	1/19/21	AM											
56	TP-3 S-3 @ 8'	1/19/21	AM											

Collected/Relinquished By:	Date	Time	Received By:	Date	Time
Robbie Hild	1/22/21	12:30	[signature]	1/22/21	1230
Relinquished By: Robbie Hild	Date	Time	Received By:	Date	Time
Relinquished By:	Date	Time	Received By:	Date	Time

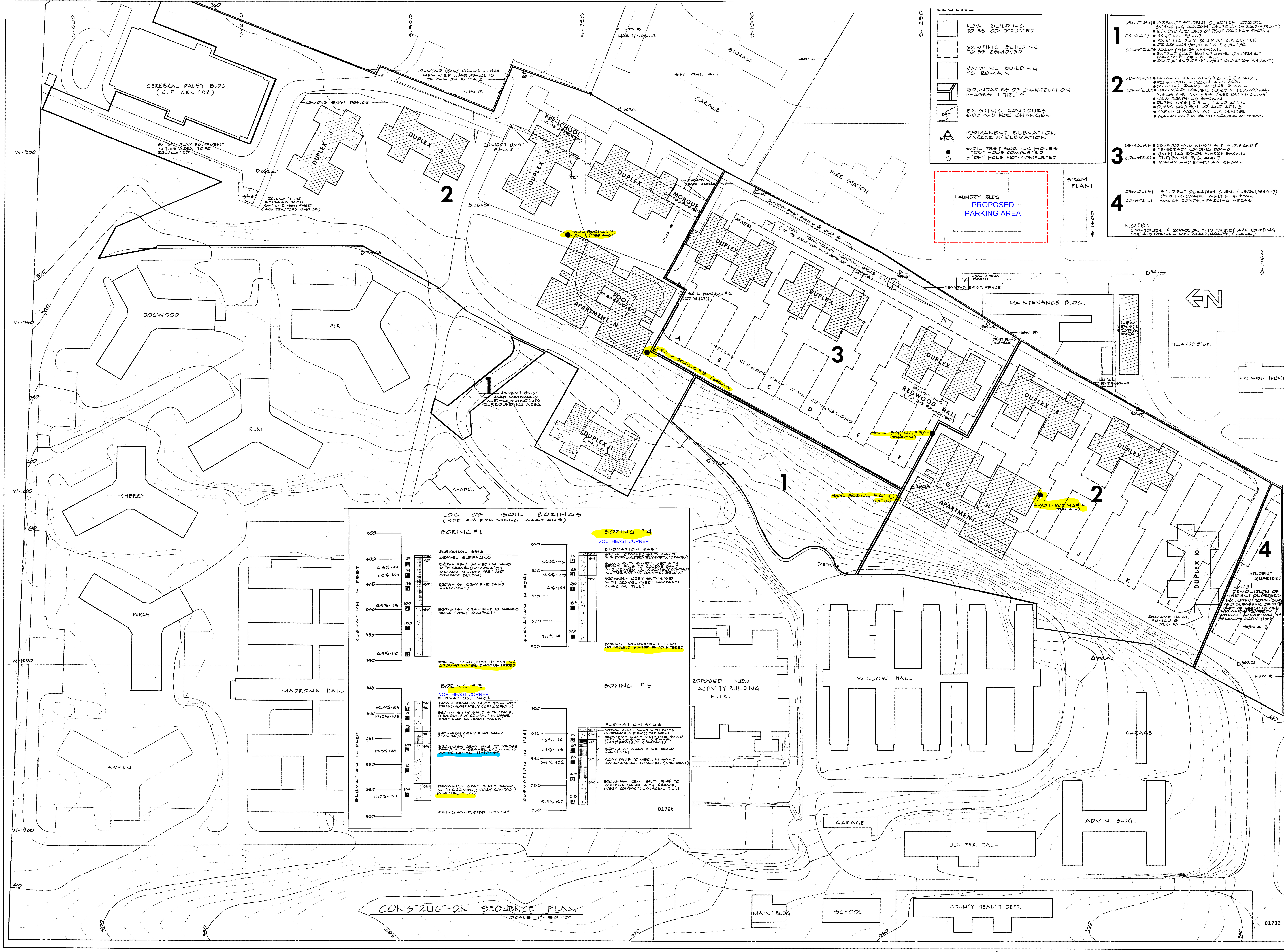
COMMENTS:

client

APPENDIX C

APPENDIX C

ATP BUILDING EXISTING BORING LOGS



Revisions:
5-1-72 DUPLEX 11 - DELETED PER ALTERNATE
12-27-72 "AS-BUILT" CONDITION

Sheet Contents: CONSTRUCTION SEQUENCE PLAN

Drawn: ATWOOD
Traced:
Checked: DeCHASSE
Date: 7-1-70

Approvals:
Agency:
Job Capt.:
Section Head:
Agency Director:
Supv. of Eng. & Arch.:

REGISTERED ARCHITECT
STATE OF WASHINGTON

Prepared by:
SHAVEY & SCHMIDT A.I.A., ARCHITECTS
710 CHERRY STREET SEATTLE, WASHINGTON 98104

Project Title:
REPLACE REDWOOD HALL
FIRCREST SCHOOL
SEATTLE, WASHINGTON

State Job No.: 67-366
Sheet No.: A2

STATE OF WASHINGTON - DEPARTMENT OF GENERAL ADMINISTRATION - DIV. OF ENGINEERING & ARCHITECTURE

67-366

1702