GeoDesign Inc

PRELIMINARY REPORT OF DUE DILIGENCE GEOTECHNICAL ENGINEERING SERVICES

Proposed School Campus
Warrenton, Oregon

For
Warrenton-Hammond School District
July 25, 2018

GeoDesign Project: WarrHammSD-1-01

GEODESIGN INC

July 25, 2018

Warrenton-Hammond School District
820 SW Cedar Avenue
Warrenton, OR 97146

Attention: Mark Jeffery

Preliminary Report of Due Diligence
Geotechnical Engineering Services
Proposed School Campus
Warrenton, Oregon
GeoDesign Project: WarrHammSD-1-01

GeoDesign, Inc. is pleased to submit this due diligence report of geotechnical engineering services for the planned Warrenton-Hammond School District (WHSD) Campus Tax Lots 100, 103, and 2301 located east of Dolphin Road in Warrenton, Oregon. We understand that WHSD is interested in building a new K-12 school campus that is outside of the tsunami hazard zone in three phases. Our services for this project were conducted in accordance with our scope and updated fee dated May 31, 2018.

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

Sincerely,

GeoDesign, Inc.

Shawn M. Dimke, P.E., G.E.
Principal Engineer

CMC:SMD:kt
Attachments
One copy submitted (via email only)
Document ID: WarrHammSD-1-01-072518-geor.docx

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ACRONYMS AND ABBREVIATIONS

ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BGS	below ground surface
CPT	cone penetrometer test
CSZ	Cascadia Subduction Zone
DOGAMI	Oregon Department of Geology and Mineral Industries
FEMA	Federal Emergency Management Agency
g	gravitational acceleration (32.2 feet/second^2)
GPS	global positioning system
H:V	horizontal to vertical
IBC	International Building Code
LiDAR	light detection and ranging
OSHA	Occupational Safety and Health Administration
OSSC	Oregon Standard Specifications for Construction (2018)
pcf	pounds per cubic foot
psf	pounds per square foot
psi	pounds per square inch
SOSSC	State of Oregon Structural Specialty Code
WHSD	Warrenton-Hammond School District

1.0 INTRODUCTION

GeoDesign, Inc. is pleased to submit this due diligence report of geotechnical engineering services for the planned Warrenton-Hammond School District (WHSD) Campus Tax Lots 100, 103, and 2301 located east of Dolphin Road in Warrenton, Oregon. We understand that WHSD is interested in building a new K-12 school campus that is outside of the tsunami hazard zone. The new campus would be constructed in three phases, with the first phase consisting of a new building for Grades 6 through 8. The area of interest primarily includes Tax Lots 103, 2301, and the west one-third of 100. Figure 1 shows the site location relative to existing topographic and physical features. Preliminary development plans were not available for our review at the time of this report. Our services constitute a "due diligence" preliminary evaluation of the proposed site, with the primary purpose of our services to identify "fatal flaws" associated with development of the site for WHSD.

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

2.0 SCOPE OF SERVICES

The purpose of our geotechnical engineering services was to characterize site subsurface conditions and provide a due diligence preliminary geotechnical engineering evaluation of the site for use in a feasibility study for the proposed campus. Our scope of services included the following:

- Coordinated and managed the field evaluation, including utility checks, site access, and scheduling subcontractors and GeoDesign field staff.
- Reviewed prior geotechnical and geological reports and information available for the site.
- Conducted a field reconnaissance of the site.
- Completed the following subsurface explorations at the site:
  - Excavated 10 test pits to depths between 11.0 and 11.5 feet BGS.
  - Completed three CPT probes to depths between 41.0 and 61.4 feet BGS (practical refusal).
- Collected disturbed and undisturbed soil samples for laboratory testing at select depths from the test pit explorations.
- Classified the materials encountered in and maintained a detailed log of the test pit explorations.
- Completed the following laboratory tests on select soil samples collected from the test pit explorations:
  - Seventeen moisture content determinations in general accordance with ASTM D2216
  - One Atterberg limits test in general accordance with ASTM D4318
- Provided preliminary recommendations for site preparation, grading, and earthwork.
- Provided preliminary recommendations for the preferred foundation type. We anticipate that structures can be supported on shallow spread footings and foundation recommendations will include allowable capacity, settlement estimates, and lateral resistance parameters.
- Provided preliminary recommendations for preparation of floor slab subgrades.

- Provided preliminary recommendations for the management of identified groundwater conditions that may affect the performance of structures.
- Evaluated seismic hazards, including liquefaction, lateral spreading, tsunami inundation, and ground rupture.
- Provided preliminary recommendations for 2015 IBC seismic coefficients.
- Provided this preliminary report summarizing our explorations, laboratory testing, and preliminary recommendations.

3.0 SITE CONDITIONS

3.1 GEOLOGIC CONDITIONS

The site is located in the Clatsop Plains, which resides on the western flank of the Coast Range physiographic province. The Clatsop Plains is composed of a series of alluvial and marine terraces flanked by ocean beaches to the west, Youngs Bay and the Columbia River to the north, and the Coast Range uplands to the east. The terraces represent river and ocean wave-cut platforms formed on marine bedrock during past high sea level stands. The wave-cut platforms are covered by terrace sediments consisting of beach, fluvial, and estuary deposits and are generally mantled by old sand dune deposits. The marine terraces have been tectonically uplifted and faulted to their present position and deeply weathered and incised by coastal streams.

The near-surface geologic unit is mapped as Quaternary Age (up to 2 million years before present) terrace deposits. The deposits consist of unconsolidated clayey silt, clay, sand, and gravel (Niem and Niem, 1985; and Schlicker et al., 1972). The terrace deposits are locally overlain by eolian dune sand. Based on CPT logs completed for the project, it appears the unconsolidated sediments extend approximately 20 to 40 feet BGS at the site.

The terrace deposits overlie a lower member of the Smuggler Cove Formation, which is described as a late Eocene Age (approximately 37 million to 38 million years before present) silty claystone (Niem and Niem, 1985). The Smuggler Cove Formation is considered to extend approximately several hundred feet BGS in the site vicinity.

3.2 GEOLOGIC HAZARDS

We completed a review of available geologic publications, hazard mapping, and LiDAR topography along with a site reconnaissance to identify hazards relevant to the site. In our opinion, the major geologic and coastal hazards include earthquakes and tsunami inundation. A discussion of geologic and coastal hazards are summarized in the following sections.

3.2.1 Coastal Flooding

Geologic hazard mapping by Schlicker et al. (1972) indicates that the lower elevations on the site, primarily in the drainages, are flood hazard areas. We reviewed available flood hazard maps (Flood Insurance Rate Maps) for the site vicinity (FEMA, 2018). The proposed site is located in Zone X, which is described as an area of minimal flood hazard.

3.2.2 Earthquakes and Tsunami Inundation

Earthquake sources in the site vicinity include CSZ plate interface earthquakes, CSZ intraplate earthquakes (also referred to as Benioff Zone or intraslab earthquakes), and local crustal earthquakes. In our opinion, CSZ plate interface earthquakes are the most significant contributor to the earthquake hazard at the site. Geologic evidence indicates that CSZ plate interface earthquakes occur approximately every 200 to 800 years with average recurrence intervals of approximately 600 years. The most recent of these plate interface earthquakes, having a magnitude of approximately 9, occurred approximately 300 years ago.

Tsunamis in the Pacific Ocean can be generated from near-source locations (such as fault rupture on the CSZ) or from far-source locations (such as large subduction zone earthquakes in the Pacific Ocean [Alaska or Japan]). The proposed building site is elevated, and based on tsunami mapping (DOGAMI, 2013), tsunami events are not expected to inundate the building site. Lower non-building portions of the site as shown on Figure 2 may be inundated from a CSZ-generated tsunami.

3.2.3 Fault Rupture

Faults are not mapped beneath the site, and the nearest mapped fault is approximately 7 miles away. Consequently, it is our opinion that the probability of surface fault rupture beneath the site is low.

3.2.4 Liquefaction and Lateral Spreading

Liquefaction is caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. In general, loose, saturated sand soil with low silt and clay content is the most susceptible to liquefaction. The soil underlying the site is primarily silt and clay, so liquefaction and lateral spreading associated with liquefaction are not expected to be a design consideration for the proposed project.

3.3 SURFACE CONDITIONS

The site is located on relatively flat-lying ground between elevations of 40 and 60 feet that contains several incised drainage ravines that trend to the south. Figure 2 shows the existing site features and topography along with locations of our subsurface explorations. A low ridge extends up to an elevation of approximately 90 feet east of the site that has gentle to moderate slopes. A majority of the site is covered by agricultural grass. The drainages located in the south portion of the site contain dense vegetation and tree cover. The northwest corner of the site contains a gravel access road from Dolphin Road that leads to a variety of imported stockpiled materials. The stockpiled materials generally consisted of silt; silty gravel; coarse, angular, crushed aggregate; and boulders. The ground surrounding the stockpiles appeared to have been graded in the past and contained evidence of imported fill material. Based on a review of historical aerial photographs, we noted a disturbance area, indicating active fill placement in the northwest corner of the site. The disturbance appears to have started between 2006 and 2009. The area of disturbance is shown as the approximate undocumented fill area on Figure 2.

3.4 SUBSURFACE CONDITIONS

3.4.1 General

We explored subsurface conditions by excavating 10 test pits (TP-1 through TP-10) to depths between 11.0 and 11.5 feet BGS and completing three CPT probes (CPT-1 through CPT-3) to refusal depths between 41.0 and 61.4 feet BGS. The approximate exploration locations are shown on Figure 2. The test pit logs and results of the laboratory testing completed at the site by GeoDesign are presented in Appendix A. The CPT logs are presented in Appendix B.

In general, subsurface conditions consist of silt and clay soil with thin silty sand interbeds overlying sedimentary bedrock, which presumably resulted in refusal of the CPT probes. Localized areas of fill were also encountered in the west portion of the site. We encountered topsoil and tilled soil zones ranging from 6 to 18 inches thick and surficial root zones ranging from 2 to 3 inches thick. The following sections provide a detailed description of the subsurface conditions encountered.

3.4.2 Undocumented Fill

Undocumented fill presumably from nearby commercial development was encountered from the ground surface to 3.0 to 6.0 feet BGS in test pits TP-7, TP-8, and TP-10. The fill consists of stiff to very stiff silt with varying amount of sand, gravel, cobbles, and wood. The fill overlies a 1.0- to 2.0-foot-thick layer of medium stiff to stiff, dark brown silt that was interpreted to represent a buried topsoil layer. The area of undocumented fill is shown on Figure 2.

3.4.3 Native Soil

The topsoil and fill are underlain by native soil consisting of layered silt and clay with thin layers of silty sand that extend to the bottom of our test pit explorations. The silt and clay soil encountered in the test pits is generally medium stiff to stiff with trace organics. Medium dense, fine, silty sand layers ranging in thickness from 2 to 18 inches were encountered in several of the test pits. Very stiff to hard silt and clay was generally encountered at depths of approximately 20.2, 14.0, and 32.5 feet BGS in CPT-1, CPT-2, and CPT-3, respectively. Soft, fine-grained soil was also observed from approximately 8.9 to 10.2 feet BGS in CPT-1.

The tested moisture content of the silt and clay ranged from 33 to 64 percent at the time of our explorations. The tested moisture content of the silty sand ranged from 24 to 37 percent at the time of our explorations. One Atterberg limits test indicated the clay soil exhibits high plasticity with a plasticity index of 43.

3.4.4 Groundwater

Slow groundwater seepage was observed in test pits TP-5, TP-6, and TP-10 at depths ranging between 8.0 and 9.0 feet BGS. Pore pressure data from the CPT probes indicate groundwater at depths of approximately 10.5 feet BGS. Based on our observations and research, we anticipate water can be encountered at shallow depths below the ground surface, particularly near lower drainage areas and during the wet season. The depth to groundwater may fluctuate in response to seasonal changes, prolonged rainfall, changes in surface topography, and other factors not observed in this study.

4.0 CONCLUSIONS AND PRELIMINARY RECOMMENDATIONS

Based on our explorations, laboratory testing, and a review of information for the site, it is our opinion that the site is acceptable for development and the anticipated foundation loads can be supported on shallow spread footings bearing on minimum 6-inch-thick granular pads. The following are expected to be the primary geotechnical considerations impacting the proposed development of the site.

- The lower elevations of the site are in the mapped tsunami inundation area as shown on Figure 2. Although not shown on Figure 2, much of the tsunami inundation area is also mapped as wetlands. We understand the proposed development would need to be located above the mapped tsunami inundation area.
- Undocumented fill and a buried topsoil zone was encountered in the west portion of the site in the area indicated on Figure 2. Foundations should not be supported on undocumented fill or buried topsoil. The undocumented fill and buried topsoil zone should be removed in structural areas and replaced with structural fill if it is not removed for required site grading. Alternately, it may be possible to leave some of the undocumented fill in place in pavement areas with limited risk by only improving the surface of the exposed subgrade.
- Clay was encountered at shallow depths at the site and one Atterberg limits test indicates the clay exhibits high plasticity. We recommend evaluating the shrink-swell potential of the on-site soil with further laboratory testing. Depending on the tested expansion potential, design measures such as deeper granular pads for footings and foundation drains extending to the base of the granular pads could be needed to limit the shrink-swell potential for the proposed buildings.
- The majority of the site has a tilled zone from prior agricultural use. The tilled zone is soft when wet and has poor engineering properties from repeated disturbance. In areas where proposed site cuts will not remove the tilled zone, we recommend the tilled zone be removed and replaced with structural fill, scarified and re-compacted as structural fill, or stabilized using cement and/or lime amendment within all building and pavement areas.
- The on-site soil can be used for structural fill. Given the fine-grained nature of the soil at the site, the use of the on-site soil for structural fill can be sensitive to small changes in moisture content and difficult, if not impossible, to adequately compact during wet weather or when the moisture content of the soil is more than a couple of percent above the optimum required for compaction. We anticipate that the moisture content of the soil currently will be above optimum and drying will be required for use as structural fill. Drying the soil will require an extended period of dry weather, typically experienced from early July to mid-October. Alternately, on-site soil can be amended for placement as structural fill without drying to the optimum moisture content for compaction.
- SOSSC requires a seismic hazard evaluation for special occupancy structures. Special occupancy structures include "buildings for every public, private or parochial school through secondary level or day care centers with a capacity greater than 250 individuals." Accordingly, a seismic hazard report will be required as part of the final geotechnical report for the site.

Our preliminary design and construction recommendations for the project are provided in the following sections.

5.0 PRELIMINARY DESIGN RECOMMENDATIONS

5.1 PERMANENT SLOPES

Permanent cut slopes in the lower half of the site should not exceed a gradient of 3H:1V, and cut and fill slopes for the remainder of the site should not exceed a gradient of 2H:1V unless specifically evaluated for stability. Slopes that will be maintained by mowing should not be constructed steeper than 3H:1V. Slopes should be planted with appropriate vegetation to provide protection against erosion as soon as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

5.2 DRAINAGE

5.2.1 Surface Drainage

We recommend that all roof drains be connected to a tightline leading to storm drain facilities. Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. We also recommend that ground surfaces adjacent to buildings be sloped away from the buildings to facilitate drainage away from the buildings.

5.2.2 Foundation Drains

We recommend installing a perimeter foundation drain around new buildings. The foundation drains should be constructed at a minimum slope of approximately ½ percent and pumped or drained by gravity to a suitable discharge. The perforated drainpipe should not be tied to a stormwater drainage system without backflow provisions. The foundation drains should consist of 4-inch-diameter, perforated drainpipe embedded in a minimum 2-foot-wide zone of crushed drain rock that extends up to 6 inches BGS and is wrapped in a drainage geotextile. The invert elevation of the drainpipe should be installed at least 18 inches below the finish floor elevation. The drain rock and drainage geotextile should meet the requirements specified in the "Materials" section.

5.3 SEISMIC DESIGN CRITERIA

5.3.1 Seismic Design Parameters

Preliminary seismic code-based seismic design parameters for the 2015 IBC are provided below. However, for the proposed special occupancy structures at the site, a site-specific seismic hazard evaluation in accordance with the 2014 SOSSC and ASCE 7-10 will be required for final development.

Table 1. Seismic Design Parameters

Parameter	Short Period (T_s = 0.2 second)	1 Second Period (T_1 = 1.0 second)
Spectral Acceleration, S	S_s = 1.331 g	S_1 = 0.682 g
Site Class	D	
Site Coefficient, F	F_a = 1.000	F_v = 1.500
Spectral Acceleration Parameters, S_M	S_{MS} = 1.331 g	S_{M1} = 1.023 g
Design Spectral Acceleration Parameters, S_D	S_{DS} = 0.887 g	S_{D1} = 0.682 g

5.3.2 Liquefaction and Lateral Spreading

Liquefaction is caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. In general, loose, saturated sand soil with low silt and clay content is the most susceptible to liquefaction. Saturated silty soil with low plasticity is moderately susceptible to liquefaction or cyclic failure under relatively higher levels of ground shaking. We did not encounter any significant amount of soil considered to be susceptible to liquefaction or cyclic failure at the site. Since the site is not near an open face with saturated conditions and has low susceptibility to liquefaction, lateral spreading is expected to be negligible at this site.

5.4 PRELIMINARY SHALLOW FOUNDATION RECOMMENDATIONS

5.4.1 General

Based on the results of our explorations, the proposed buildings can likely be supported on shallow spread footings bearing on minimum 6-inch-thick granular pads over firm, undisturbed native soil. Deeper granular pads may be necessary if soft, loose, or deleterious material is required or further testing indicates a shrink-swell potential for the on-site soil.

5.4.2 Bearing Capacity and Dimensions

Shallow footings can be proportioned using a preliminary allowable bearing pressure of 2,500 psf. The value above 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 can be doubled for short-term loads resulting from wind or seismic forces.

Isolated column and continuous wall footings should have minimum widths of 24 and 18 inches, respectively. The base of exterior footings should be founded at least 18 inches below the lowest adjacent finished grade. Interior footings can be founded 12 inches below the bottom of the floor slab.

Total consolidation-induced settlement should be less than 1 inch, with differential settlement of up to ½ inch between lightly loaded and heavily loaded footings.

5.4.3 Resistance to Sliding

Lateral loads on footings can be resisted by passive earth pressure on the sides of the structures and by friction on the base of the footings. Our analysis indicates that the available passive earth pressure for footings confined by on-site soil and structural fill is 350 pcf, modeled as an equivalent fluid pressure. Typically, the movement required to develop the available passive resistance may be relatively large; therefore, we recommend using a reduced passive pressure of 250 pcf equivalent fluid pressure. Adjacent floor slabs, pavements, or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance. For footings in contact with imported granular material, a coefficient of friction equal to 0.40 may be used when calculating resistance to sliding.

5.4.4 Subgrade Evaluations

All footing subgrades should be evaluated by a member of our geotechnical staff. Observations should also evaluate whether all loose or soft material, organics, unsuitable fill, prior topsoil zones, or softened subgrades (if present) have been removed and native soil subgrades have not dried excessively. Localized deepening of footing excavations may be required to penetrate debris, fill, softened, dried, or deleterious material, if encountered.

6.0 CONSTRUCTION

6.1 SITE PREPARATION

6.1.1 Demolition

Demolition includes removing existing buildings, pavements, concrete curbs, abandoned utilities, and any subsurface elements. Demolished material should be transported off site for disposal. Excavations remaining from removing basements (if present), foundations, utilities, and other subsurface elements should be backfilled with structural fill where these are below planned site grades. The base of the excavations should be excavated to expose firm subgrade before filling. The sides of the excavations should be cut into firm material and sloped a minimum of 1½H:1V. Utility lines abandoned under new structural components should be completely removed and backfilled with structural fill. Soft or disturbed soil encountered during demolition should be removed and replaced with structural fill.

6.1.2 Stripping

The existing topsoil zone should be stripped and removed from all fill areas. Based on our explorations, the average depth of stripping will be approximately 2 to 3 inches, although greater stripping depths may be required to remove localized zones of loose or organic soil. Greater stripping depths should be anticipated in areas with thicker vegetation and along the base of draws. 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 used in landscaped areas.

6.1.3 Subgrade Evaluation

Upon completion of stripping and subgrade stabilization and prior to the placement of fill or pavement improvements, the exposed subgrade should be evaluated by proof rolling. The subgrade should be proof rolled with a fully loaded dump truck or similar heavy, rubber tire construction equipment to identify soft, loose, or unsuitable areas. A member of our geotechnical staff should observe proof rolling to evaluate yielding of the ground surface. During wet weather, subgrade evaluation should be performed by probing with a foundation probe rather than proof rolling. Subgrades should be covered to avoid excessive drying. Areas that appear soft or loose or subgrades that have dried excessively should be improved in accordance with subsequent sections of this report.

6.1.4 Test Pit Locations

The test pit excavations were backfilled using relatively minimal compactive effort; therefore, soft areas can be expected at these locations. We recommend that this relatively uncompacted soil be removed from the test pits located within proposed foundation and paved areas to a depth of 3 feet BGS. The resulting excavation should be brought back to grade with structural fill. Deeper removal depth will be required where foundations are located over test pit locations.

6.2 SUBGRADE PROTECTION

The fine-grained soil present on this site is easily disturbed. If not carefully executed, site preparation, utility trench work, and roadway excavation can create extensive soft areas and significant repair costs can result. Earthwork planning, regardless of the time of year, should include considerations for minimizing subgrade disturbance.

If construction occurs during or extends into the wet season, or if the moisture content of the surficial soil is more than a couple percentage points above optimum, site stripping and cutting may need to be accomplished using track-mounted equipment. Likewise, the use of granular haul roads and staging areas will be necessary for support of construction traffic during the rainy season or when the moisture content of the surficial soil is more than a few percentage points above optimum. The amount of staging and haul road areas, as well as the required thickness of granular material, will vary with the contractor's sequencing of a project and type/frequency of construction equipment. Based on our experience, between 12 and 18 inches of imported granular material is generally required in staging areas and between 18 and 24 inches in haul roads areas. Stabilization material may be used as a substitute, provided the top 4 inches of material consists of imported granular material. The actual thickness will depend on the contractor's means and methods and, accordingly, should be the contractor's responsibility. In addition, 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, stabilization material, and geotextile fabric should meet the specifications in the "Materials" section.

As an alternative to thickened crushed rock sections, haul roads and utility work zones may be constructed using amended subgrades overlain by a crushed rock wearing surface. If the subgrade is amended, the thickness of granular material in staging areas and along haul roads can typically be reduced to between 6 and 9 inches. This recommendation is based on an assumed minimum unconfined compressive strength of 100 psi for subgrade amended to a depth of 12 to 16 inches. The actual thickness of the amended material and imported granular material will depend on the contractor's means and methods and, accordingly, should be the contractor's responsibility. Amendment is discussed in the "Materials" section.

6.3 *EXCAVATION*

6.3.1 Excavation and Shoring

Temporary excavation sidewalls should stand vertical to a depth of approximately 4 feet, provided groundwater seepage is not observed in the sidewalls. Open excavation techniques may be used to excavate trenches with depths between 4 and 8 feet, provided the walls of the excavation are cut at a slope of 1H:1V and groundwater seepage is not present. At this inclination, the slopes may slough and require some ongoing repair. Excavations should be flattened to 1½H:1V or 2H:1V if excessive sloughing or raveling occurs. In lieu of large and open cuts, approved temporary shoring may be used for excavation support. A wide variety of shoring and dewatering systems are available. Consequently, we recommend that the contractor be responsible for selecting the appropriate shoring and dewatering systems. If box shoring is used, it should be understood 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, caving of the sidewalls may occur. The presence of caved material will limit the ability to properly backfill and compact the trenches. The contractor should be prepared to fill voids between the box shoring and the sidewalls of the trenches with sand or gravel before caving occurs.

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. All excavations should be made in accordance with applicable OSHA and state regulations.

6.3.2 Trench Dewatering

Dewatering will be required if groundwater is encountered. A sump located within the trench excavation will likely be sufficient to remove the accumulated water, depending on the amount and persistence of water seepage and the length of time the trench is left open. Flow rates for dewatering are likely to vary depending on location, soil type, and the season during which the excavation occurs. The dewatering systems should be capable of adapting to variable flows.

If groundwater is present at the base of utility excavations, we recommend placing at least 12 inches of stabilization material at the base of the excavations. Trench stabilization material should meet the requirements provided in the "Materials" section.

We note that these recommendations are for guidance only. The dewatering of excavations is the sole responsibility of the contractor, as the contractor is in the best position to select these systems based on their means and methods.

6.3.3 Safety

All excavations should be made in accordance with applicable OSHA requirements and regulations of the state, county, and local jurisdiction. While this report describes certain approaches to excavation and dewatering, the contract documents should specify that the contractor is responsible for selecting excavation and dewatering methods, monitoring the excavations for safety, and providing shoring (as required) to protect personnel and adjacent structural elements.

6.4 MATERIALS

6.4.1 Structural Fill

6.4.1.1 General

Fill should be placed on subgrade that has been prepared in conformance with the "Site Preparation" section. 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 material and should meet the specifications provided in OSSC 00330 (Earthwork), OSSC 00400 (Drainage and Sewers), and OSSC 02600 (Aggregates), depending on the application. A brief characterization of some of the acceptable materials and our recommendations for their use as structural fill is provided in this section.

In locations where fill is to be placed on slopes steeper than 5H:1V, level benches should be cut into the existing sloping surfaces to remove the surface loose material and should extend into the structural fill of the existing embankment. The benches should be a minimum of 10 feet wide or 1½ times the width of the compaction equipment, whichever is wider.

6.4.1.2 On-Site Soil

The material at the site should be suitable for use as general structural fill in some areas, provided it is properly moisture conditioned; free of debris, organic material, and particles over 6 inches in diameter; and meets the specifications provided in OSSC 00330.12 (Borrow Material).

Based on laboratory test results, the moisture content of the on-site soil at the time of our explorations was above the optimum for compaction. Moisture conditioning (drying) will be required to use on-site soil for structural fill. Accordingly, extended dry weather (typically experienced between early July and mid-October) will be required to adequately condition and place the soil as structural fill. It will be difficult, if not impossible, to adequately compact on-site soil during the rainy season or during prolonged periods of rainfall.

When used as structural fill, native soil should be placed in lifts with a maximum uncompacted thickness of 6 to 8 inches and compacted to not less than 92 percent of the maximum dry density for fine-grained soil and 95 percent of the maximum dry density for granular soil, as determined by ASTM D1557.

6.5.1.3 Imported Granular Material

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 OSSC 00330.14 (Selected Granular Backfill) or OSSC 00330.15 (Selected Stone Backfill). The imported granular material should also 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 fractured faces. Material with a higher fines content of up to 12 percent is permissible provided compaction can be achieved.

Imported granular material 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. During the wet season or when wet subgrade conditions exists, the initial lift should be approximately 18 inches in uncompacted thickness and should be compacted by rolling with a smooth-drum roller without using vibratory action.

6.5.1.4 Stabilization Material

Stabilization material used in staging or haul road areas, or as trench stabilization material, should consist of 4- or 6-inch-minus pit- or quarry-run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in OSSC 00330.15 (Selected Stone Backfill). The material should have a maximum particle size of 6 inches, less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve, and at least two mechanically fractured faces. The material should be free of organic matter and other deleterious material. Stabilization material should be placed in lifts between 12 and 24 inches thick and compacted to a firm condition.

6.5.1.5 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 1½ inches and less than 10 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in OSSC 00405.13 (Pipe Zone Material). The pipe zone backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

Within roadway alignments, the remainder of the trench backfill up to the subgrade elevation should consist of well-graded granular material with a maximum particle size of 2½ inches and less than 10 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in OSSC 00405.14 (Trench Backfill; Class B, C, or D). This material should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department. The upper 3 feet of the trench backfill should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D1557.

Outside of structural improvement areas (e.g., roadway alignments or building pads) trench backfill placed above the pipe zone may consist of general fill material that is free of organics and material over 6 inches in diameter and meets the specifications provided in OSSC 00405.14 (Trench Backfill; Class A, B, C, or D). This general trench backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

6.5.1.6 Drain Rock

Drain rock should consist of angular, granular material with a maximum particle size of 2 inches and should meet the specifications provided in OSSC 00430.11 (Granular Drain Backfill Material). The material should be free of roots, organic matter, and other unsuitable material; have less than 2 percent by dry weight passing the U.S. Standard No. 200 sieve (washed analysis); and have at least two mechanically fractured faces. Drain rock should be compacted to a well-keyed, firm condition.

6.5.1.7 Aggregate Base Rock

Imported granular material used as base rock for building floor slabs and pavements should consist of ¾- or 1½-inch-minus material (depending on the application) and meet the requirements in OSSC 00641 (Aggregate Subbase, Base, and Shoulders). In addition, the aggregate should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve. The aggregate base should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

6.5.2 Geotextile Fabric

6.5.2.1 Subgrade Geotextile

Subgrade geotextile should conform to OSSC Table 02320-1 and OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles.

6.5.2.2 Drainage Geotextile

Drainage geotextile should conform to Type 2 material of OSSC Table 02320-1 and OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles.

6.5.3 Soil Amendment

6.5.3.1 General

In conjunction with an experienced contractor, the on-site soil can be amended to obtain suitable support properties without shrink-swell potential. Successful use of soil amendment depends on the use of correct mixing techniques, soil moisture content, and amendment quantities. Soil amending should be conducted in accordance with the specifications provided in OSSC 00344 (Treated Subgrade). The amount of lime or cement used during treatment should be based on an assumed soil dry unit weight of 100 pcf.

6.5.3.2 Subgrade Stabilization

Specific recommendations based on exposed site conditions for soil amending can be provided if necessary. However, for preliminary design purposes, we recommend a target strength for amended soils of 100 psi. The amount of lime and/or cement necessary will vary with moisture content, soil type, and desired strength. It is difficult to predict field performance of soil to lime and cement amendment due to variability in soil response, and we recommend laboratory testing to confirm expectations. Typically, 3 to 6 percent dried quicklime by weight or 4 to 8 percent cement by weight is required to stabilize soil. For preliminary design purposes, we recommend assuming 7 percent cement by weight will be necessary to amend the on-site soil for placement as structural fill at the current moisture contents. The amount of lime and cement added to the soil may need to be adjusted based on field observations and performance. Moreover, depending on the time of year and moisture content levels during amendment, water may need to be applied during tilling to appropriately condition the soil moisture content.

A minimum curing of four days is required between treatment and construction traffic access. Construction traffic should not be allowed on unprotected amended subgrade. To protect the treated surfaces from abrasion or damage, the finished surface should be covered with 4 to 6 inches of imported granular material.

The crushed rock placed over treated subgrades typically becomes contaminated with soil during construction. Contaminated base rock should be removed and replaced with clean rock in pavement areas. The actual thickness of the amended material and imported granular material for haul roads and staging areas will depend on the anticipated traffic, as well as the contractor's means and methods and, accordingly, should be the contractor's responsibility.

6.5.3.3 Other Considerations
On-site soil that because of elevated moisture contents would not otherwise be suitable for structural fill may be amended and placed as fill over a subgrade prepared in conformance with the "Site Preparation" section. Typically, a minimum curing of four days is required between treatment and construction traffic access. Consecutive lifts of fill may be treated immediately after the previous lift has been amended and compacted (e.g., the four-day wait period does not apply). However, where the final lift of fill is a building or roadway subgrade, the four-day wait period is in effect.

Portland cement- and lime-amended soil is hard and has low permeability. This soil does not drain well and it is not suitable for planting. Future planted areas should not be amended, if practical, or accommodations should be made for drainage and planting. Moreover, amending soil within building areas must be done carefully to avoid trapping water under floor slabs. We should be contacted if this approach is considered. Amendment should not be used if runoff during construction cannot be directed away from adjacent wetlands.

7.0 OBSERVATION OF CONSTRUCTION

Satisfactory foundation and earthwork performance depends to a large degree on quality of construction. Sufficient observation 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 the subsurface exploration. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect if subsurface conditions change significantly from those anticipated.

We recommend that GeoDesign be retained to observe earthwork activities, including stripping, proof rolling of the subgrade and repair of soft areas, footing subgrade preparation, performing laboratory compaction and field moisture-density tests, observing final proof rolling of the pavement subgrade and base rock, and asphalt concrete placement and compaction.

8.0 LIMITATIONS

We have prepared this preliminary report for use by Warrenton-Hammond School District and members of the design and construction team for the proposed project. 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 nearby building 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 the course of 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 for the buildings, 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 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 our 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 the report was prepared. No warranty, express or implied, should be understood.

♦ ♦ ♦

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

Sincerely,

GeoDesign, Inc.

Charles M. Clough, C.E.G.
Project Engineering Geologist

Shawn M. Dimke, P.E., G.E.
Principal Engineer

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63114PE
OREGON
March 14, 2006
SHAWN DIMKE
EXPIRES: 12/31/19

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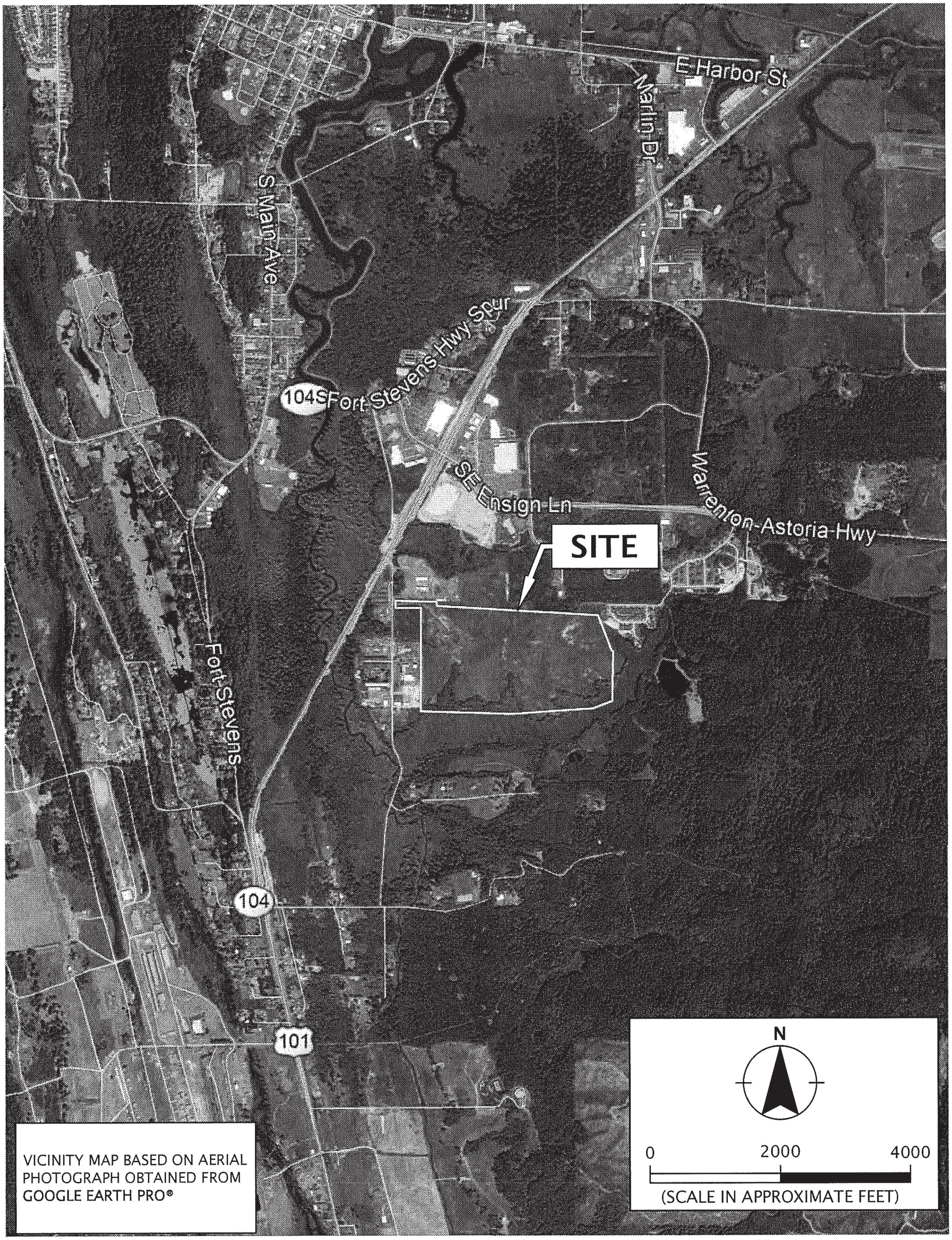
FIGURES



GEODESIGN INC 9450 SW Commerce Circle - Suite 300 Wilsonville OR 97070 503.968.8787 www.geodesigninc.com	WARRHAMDSD-1-01	VICINITY MAP	
	JULY 2018	PROPOSED SCHOOL CAMPUS WARRENTON, OR	FIGURE 1

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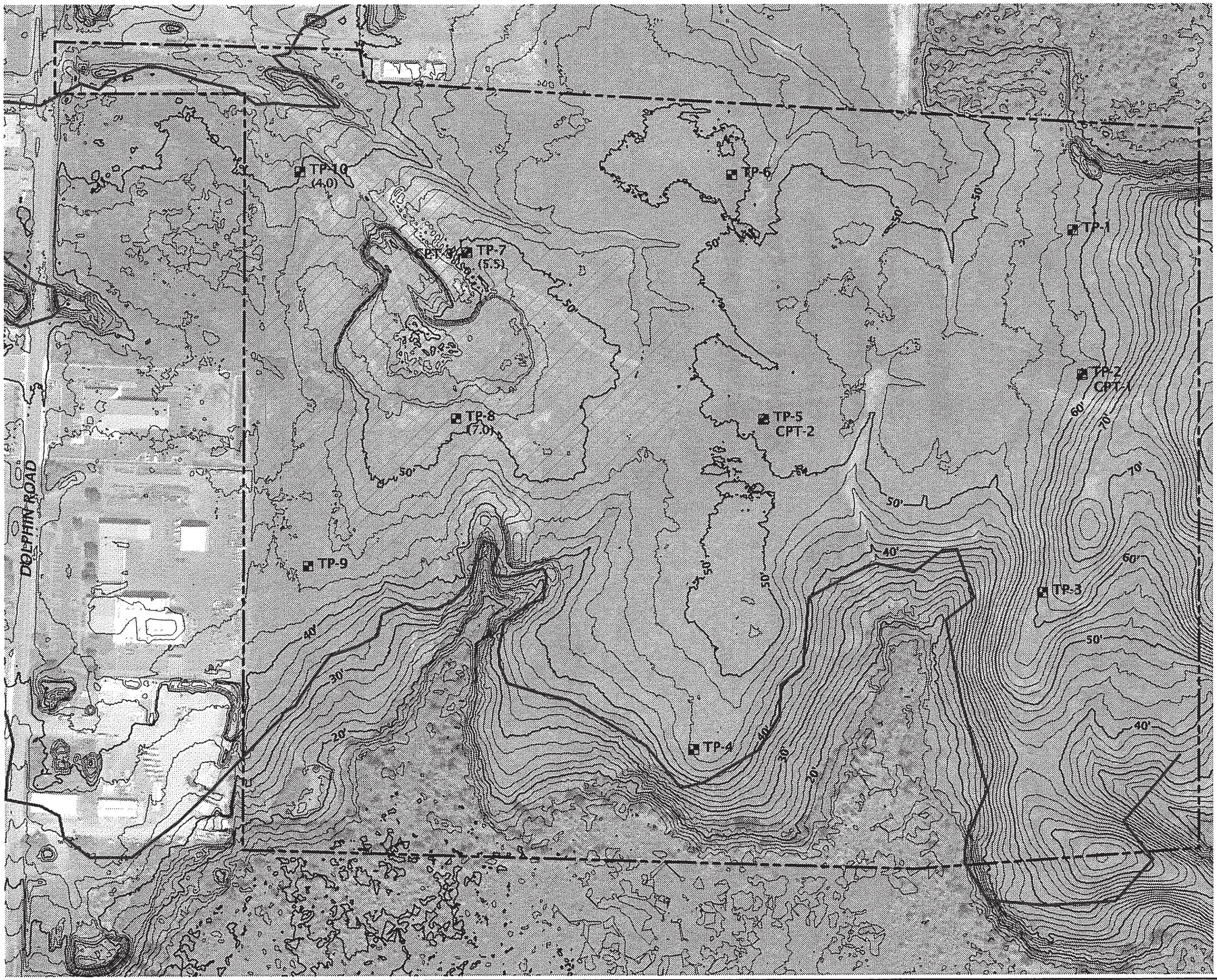
File Name: J:\S-Z\WarrHammSD\WarrHammSD-1\WarrHammSD-1-01\Figures\CAD\WarrHammSD-1-01-VM01.dwg | Layout: FIGURE 1



LEGEND:

TP-1	TEST PIT
(5.5)	DEPTH IN FEET OF UNDOCUMENTED FILL AND BURIED TOPSOIL WHERE ENCOUNTERED
CPT-1	CONE PENETROMETER
	SITE BOUNDARY
	TSUNAMI INUNDATION LIMIT
	APPROXIMATE UNDOCUMENTED FILL AREA

N

0 200 400

(SCALE IN FEET)

SITE PLAN BASED ON AERIAL PHOTOGRAPH FROM GOOGLE EARTH PRO JULY 6, 2018

FIGURE 2

SITE PLAN

PROPOSED SCHOOL CAMPUS
WARRENTON, OR

WARRHAMDSD-1-01

JULY 2018

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File Name: J:\S-Z\WarrHammSD\WarrHammSD-1-01\Figures\CAD\WarrHammSD-1-01-SP01.dwg | Layout: FIGURE 2