Exhibit 11 CUP24-1001

Geotechnical Site Investigation

Camas High School Field House

Camas, Washington

December 20, 2019

Geotechnical ■ Environmental ■ Special Inspections

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GEOTECHNICAL SITE INVESTIGATION
CAMAS HIGH SCHOOL FIELD HOUSE
CAMAS, WASHINGTON

Prepared For:	**Mr. Chris Robertson** **Robertson Engineering, PC** **1101 Broadway Street #201** **Vancouver, WA 98660**
Site Location:	**26600 SE 15th Street** **Parcel No. 178111000** **Camas, Washington**
Prepared By:	**Columbia West Engineering, Inc.** **11917 NE 95th Street** **Vancouver, Washington 98682** **Phone: 360-823-2900** **Fax: 360-823-2901**
Date Prepared:	**December 20, 2019**

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1.0 INTRODUCTION

Columbia West Engineering, Inc. (Columbia West) was retained by Robertson Engineering, PC to conduct a geotechnical site investigation for the proposed Camas High School Field House project located in Camas, Washington. The purpose of the investigation was to observe and assess subsurface soil conditions at specific locations and provide geotechnical engineering analyses, planning, and design recommendations for proposed development. The specific scope of services was outlined in a proposal contract dated August 23, 2019. This report summarizes the investigation and provides field assessment documentation and laboratory analytical test reports. This report is subject to the limitations expressed in Section 6.0, *Conclusion and Limitations*, and Appendix E.

1.1 General Site Information

As indicated on Figures 1 and 2, the subject site is located at 26600 SE 15th Street in Camas, Washington. The proposed development area is comprised of a portion of tax parcel 178111000 totaling approximately 1.15 acres. The regulatory jurisdictional agency is the City of Camas, Washington. The approximate latitude and longitude are N 45° 36' 51" and W 122° 23' 58", and the legal description is a portion of the SE ¼ of Section 35, T2N, R3E Willamette Meridian.

1.2 Proposed Development

Correspondence with the design team indicates that proposed development will consist of an athletic field house structure and associated underground utilities, stormwater management facilities, and asphalt concrete access drives and walkways. Columbia West has not reviewed preliminary grading plans but understands that minor cut and fill will likely be proposed at the property. This report is based upon proposed development as described above and may not be applicable if modified.

2.0 REGIONAL GEOLOGY AND SOIL CONDITIONS

The subject site lies within the Willamette Valley/Puget Sound Lowland, a wide physiographic depression flanked by the mountainous Coast Range on the west and the Cascade Range on the east. Inclined or uplifted structural zones within the Willamette Valley/Puget Sound Lowland constitute highland areas and depressed structural zones form sediment-filled basins. The site is located in the eastern portion of the Portland/Vancouver Basin, an open, somewhat elliptical, northwest-trending syncline approximately 60 miles wide.

According to the *Geologic Map of the Camas Quadrangle, Clark County, Washington, and Multnomah County, Oregon* (USGS Geological Survey, Scientific Investigations Map 3017, 2008), site soils are mapped as Pleistocene- and Pliocene-aged, unconsolidated to cemented, thick bedded, pebble to boulder sedimentary conglomerate (Qtc).

The *Web Soil Survey* (United States Department of Agriculture, Natural Resource Conservation Service [USDA NRCS], 2019 Website) identifies surface soils as Hesson clay loam. Hesson series soils are generally fine-textured sands, silts, and clays with low permeability, moderate to high water capacity, and low shear strength. Hesson soils are generally moisture sensitive, somewhat compressible, and described as having low to moderate shrink-swell potential. The erosion hazard of these soils is slight primarily based primarily upon slope grade.

3.0 REGIONAL SEISMOLOGY

Recent research and subsurface mapping investigations within the Pacific Northwest appear to suggest the historic potential risk for a large earthquake event with strong localized ground movement may be underestimated. Past earthquakes in the Pacific Northwest appear to have caused landslides and ground subsidence, in addition to severe flooding near coastal areas. Earthquakes may also induce soil liquefaction, which occurs when elevated horizontal ground acceleration and velocity cause soil particles to interact as a fluid as opposed to a solid. Liquefaction of soil can result in lateral spreading and temporary loss of bearing capacity and shear strength.

There are at least four major known fault zones in the vicinity of the site that may be capable of generating potentially destructive horizontal accelerations. These fault zones are described briefly in the following text.

Portland Hills Fault Zone

The Portland Hills Fault Zone consists of several northwest-trending faults located along the northeastern margin of the Tualatin Mountains, also known as the Portland Hills, and the southwest margin of the Portland Basin. The fault zone is approximately 25 to 30 miles in length and is located approximately 15 miles west-southwest of the site. According to *Seismic Design Mapping, State of Oregon* (Geomatrix Consultants, 1995), there is no definitive consensus among geologists as to the zone fault type. Several alternate interpretations have been suggested.

According to the *USGS Earthquake Hazards Program*, the fault was originally mapped as a down-to-the-northeast normal fault, but has also been mapped as part of a regional-scale zone of right-lateral, oblique slip faults, and as a steep escarpment caused by asymmetrical folding above a south-west dipping, blind thrust fault. The Portland Hills fault offsets Miocene-aged Columbia River Basalts, and Miocene- to Pliocene-aged sedimentary rocks of the Troutdale Formation. No fault scarps on surficial Quaternary-aged deposits have been described along the fault trace, and the fault is mapped as buried by the Pleistocene-aged Missoula flood deposits.

However, evidence suggests that fault movement has impacted shallow Holocene-aged deposits and deeper Pleistocene-aged sediments. Seismologists recorded a magnitude (M) 3.2 earthquake in November 2012, and a M3.9 earthquake in April 2003 thought to be associated with the fault zone near Kelly Point Park. A M3.5 earthquake also possibly associated with the Portland Hills Fault Zone occurred approximately 1.3 miles east of the fault in 1991. Therefore, the Portland Hills Fault Zone is generally thought to be potentially active and capable of producing potentially damaging earthquakes.

Gales Creek-Newberg-Mt. Angel Fault Zone

Located approximately 36 miles west-southwest of the site, the northwest-striking, approximately 50-mile long Gales Creek-Newberg-Mt. Angel Structural Zone forms the northwestern boundary between the Oregon Coast Range and the Willamette Valley, and consists of a series of discontinuous northwest-trending faults. The southern end of the fault zone forms the southwest margin of the Tualatin basin. Possible late-Quaternary-aged geomorphic surface deformation may exist along the structural zone (Geomatrix Consultants, 1995).

According to the *USGS Earthquake Hazards Program*, the Mount Angel fault is mapped as a high-angle, reverse-oblique fault, which offsets Miocene-aged rocks of the Columbia River Basalts, and Miocene and Pliocene-aged sedimentary rocks. The fault appears to have controlled emplacement of the Frenchman Spring Member of the Wanapum Basalts, and thus must have a history that predates the Miocene age of these rocks. No unequivocal evidence of deformation of Quaternary-aged deposits has been described, but a thick sequence of sediments deposited by the Missoula floods covers much of the southern part of the fault trace.

Although no definitive evidence of impacts to Holocene-aged sediments have clearly been identified, the Mount Angel fault appears to have been the location of minor earthquake swarms in 1990 near Woodburn, Oregon, and a M5.6 earthquake in March 1993 near Scotts Mills, approximately four miles south of the mapped extent of the Mt. Angel fault. It is unclear if the earthquake occurred along the fault zone or a parallel structure. Therefore, the Gales Creek-Newberg-Mt. Angel Structural Zone is considered potentially active.

Lacamas Lake-Sandy River Fault Zone

The northwest-trending Lacamas Lake Fault and northeast-trending Sandy River Fault intersect north of Camas, Washington approximately 0.8 miles south-southwest of the site, and form part of the northeastern margin of the Portland basin. According to *Geology and Groundwater Conditions of Clark County Washington* (USGS Water Supply Paper 1600, Mundorff, 1964) and the *Geologic Map of the Lake Oswego Quadrangle* (Oregon DOGAMI Series GMS-59, 1989), the Lacamas Lake fault zone consists of shear contact between the Troutdale Formation and underlying Oligocene-aged andesite-basalt bedrock. Secondary shear contact associated with the fault zone may have produced a series of prominent northwest-southeast geomorphic lineaments in proximity to the site.

According to the *USGS Earthquake Hazards Program* the fault has been mapped as a normal fault with down-to-the-southwest displacement and has also been described as a steeply northeast or southwest-dipping, oblique, right-lateral, slip-fault. The trace of the Lacamas Lake fault is marked by the very linear lower reach of Lacamas Creek. No fault scarps on Quaternary-aged surficial deposits have been described. The Lacamas Lake fault offsets Pliocene-aged sedimentary conglomerates generally identified as the Troutdale formation, and Pliocene- to Pleistocene-aged basalts generally identified as the Boring Lava formation.

Recent seismic reflection data across the probable trace of the fault under the Columbia River yielded no unequivocal evidence of displacement underlying the Missoula flood deposits, however, recorded mild seismic activity during the recent past indicates this area may be potentially seismogenic.

Cascadia Subduction Zone

The Cascadia Subduction Zone has recently been recognized as a potential source of strong earthquake activity in the Portland/Vancouver Basin. This phenomenon is the result of the earth's large tectonic plate movement. Geologic evidence indicates that volcanic ocean floor activity along the Juan de Fuca ridge in the Pacific Ocean causes the Juan de Fuca Plate to perpetually move east and subduct under the North American Continental Plate. The subduction zone results in historic volcanic and potential earthquake activity in proximity to the plate interface, believed to lie approximately 20 to 50 miles west of the general location of the Oregon and Washington coast (Geomatrix Consultants, 1995).

4.0 GEOTECHNICAL AND GEOLOGIC FIELD INVESTIGATION

A geotechnical field investigation consisting of visual reconnaissance, three test pits (TP-1 through TP-3), one infiltration test, and one soil boring (SB-1) was conducted at the site on November 5 and 11, 2019. Test pits were explored with a track-mounted excavator. Soil borings were explored with a track-mounted mud-rotary drill system. Subsurface soil profiles were logged in accordance with Unified Soil Classification System (USCS) specifications. Disturbed and relatively undisturbed soil samples were collected from relevant soil horizons and submitted for laboratory analysis. Analytical laboratory test results are presented in Appendix A. Exploration locations are indicated on Figure 2. Subsurface exploration logs are presented in Appendix B. Soil descriptions and classification information are provided in Appendix C. A photo log is presented in Appendix D.

4.1 Surface Investigation and Site Description

The approximate 1.15-acre subject site is located at 26600 SE 15th Street in Camas, Washington. The subject site is located on the Camas High School campus and is bounded by an access drive to the west, an access drive and parking lots to the south, tennis courts to the east, and undeveloped acreage to the north. No existing buildings were observed on the site. Observed utility infrastructure included an underground storm line extending southeast from the central portion of the site to the adjacent stormwater facility. The western and northern portions of the site consist of open, landscaped areas with several mature trees bordering the northern site boundary.

Field reconnaissance and topographic mapping published by *Clark County Maps Online* indicates relatively flat terrain with slope grades of 0 to 5 percent and site elevations ranging from 378 to 382 feet above mean sea level (amsl).

4.2 Subsurface Exploration and Investigation

Test pit explorations TP-1 through TP-3 were advanced at the site to a maximum depth of 14 feet below ground surface (bgs). Soil boring exploration (SB-1) was advanced to a maximum depth of 51 ½ feet bgs. Exploration locations were selected to observe subsurface soil characteristics in proximity to proposed development areas and are indicated on Figure 2. Detailed field logs of the encountered materials are presented in Appendix B, *Subsurface Exploration Logs.*

4.2.1 Soil Type Description

The field investigation indicated the presence of approximately 6 to 12 inches of sod and topsoil in the areas observed. Underlying the topsoil layer, undocumented fill and subsurface soils resembling native USDA Hesson soil series descriptions were encountered. Subsurface lithology may generally be described by soil types identified in the following text.

Soil Type 1 – Undocumented FILL

Soil Type 1 represents undocumented FILL and was observed to primarily consist of tan, mottled, moist, medium dense clayey sand with gravel. Soil Type 1 was observed at ground surface in explorations TP-1 and TP-2 and extended to an observed depth of approximately 24 inches. Soil Type 1 was underlain by Soil Type 2 in test pit TP-1 and Soil Type 3 in test pit TP-2. Additional recommendations regarding Soil Type 1 are provided in Section 5.1.1, *Undocumented Fill.*

Soil Type 2 – Sandy Lean CLAY with Gravel

Soil Type 2 was observed to primarily consist of brown, mottled, moist, medium stiff to stiff sandy lean CLAY with gravel. Soil Type 2 was observed below the topsoil layer in soil boring SB-1, below Soil Type 1 in test pit TP-1, and below Soil Type 3 in test pit TP-2. Soil Type 2 extended to observed depths ranging from approximately 3 to 5 feet bgs where it was underlain by Soil Type 4.

Soil Type 3 – Fat CLAY with Sand

Soil Type 3 was observed to primarily consist of gray to tan, mottled, moist, stiff fat CLAY with sand. Soil Type 3 was observed below the topsoil layer in test pit TP-3 and below Soil Type 1 in test pit TP-2. Soil Type 3 extended to an observed depth of approximately 2 ½ feet bgs, where it was underlain by Soil Type 2 in TP-2 and Soil Type 4 in TP-3.

Recommendations regarding the suitability of Soil Type 3 to be reused as structural fill or bear structural foundations are presented respectively in Section 5.2, *Engineered Structural Fill* and Section 5.4, *Foundations*.

Analytical laboratory testing conducted upon a representative soil sample obtained from test pit TP-2 indicated approximately 85 percent by weight passing the No. 200 sieve and an in situ moisture content of approximately 40 percent. Atterberg Limits analysis indicated a liquid limit of 76 percent and a plasticity index of 50 percent. The laboratory tested sample of Soil Type 3 is classified CH according to USCS specifications and A-7-6(47) according to AASHTO specifications.

Soil Type 4 – Sedimentary CONGLOMERATE

Soil Type 4 was observed to consist of tan to orange-brown, moderately- to severely-weathered, moist, loose to dense sedimentary CONGLOMERATE of poorly-graded gravel in a sand, silt, and clay matrix. Soil Type 4 was observed below Soil Type 2 in explorations TP-1, TP-2, and SB-1 and below Soil Type 3 in test pit TP-3. Soil Type 4 extended to the maximum depth of exploration in each of the observed locations. Soil Type 4 may represent unconsolidated to cemented, thick-bedded, pebble to boulder sedimentary conglomerate (QTc) of Evarts, 2008.

Analytical laboratory testing conducted upon representative soils samples obtained from explorations TP-2 and SB-1 indicated approximately 8 to 39 percent by weight passing the No. 200 sieve and in situ moisture contents ranging from approximately 19 to 56 percent. Atterberg Limits analysis indicated liquid limits ranging from 47 to 57 percent and plasticity index ranging from 18 to 24 percent. Laboratory tested samples of Soil Type 4 are classified GP-GM and SM according to USCS specifications and A-2-7(0) and A-7-5(5) according to AASHTO specifications.

4.2.2 Groundwater

Groundwater was not encountered in the test pit explorations to the maximum explored depth of 14 feet bgs. Due to the use of mud-rotary drilling techniques, depth to groundwater was not measured within soil boring SB-1. Review of nearby well logs obtained from the State of Washington Department of Ecology indicates that groundwater levels in the area are approximately 18 to 180 feet bgs. Variations in groundwater elevations likely reflect the screened interval depth of these wells, changes in ground surface elevation, and the presence of multiple aquifers and confining units.

Groundwater levels are often subject to seasonal variance and may rise during extended periods of increased precipitation. Perched groundwater may also be present in localized areas. Seeps and springs may become evident during site grading, primarily along slopes or in areas cut below existing grade. Structures, roads, and drainage design should be planned accordingly.

5.0 DESIGN RECOMMENDATIONS

The geotechnical site investigation suggests the proposed development is generally compatible with surface and subsurface soils, provided the recommendations presented in this report are utilized and incorporated into the design and construction processes. The primary geotechnical concerns associated with the site are undocumented fill and high-plasticity soils. Design recommendations are presented in the following text sections.

5.1 Site Preparation and Grading

Vegetation, organic material, unsuitable fill, and deleterious material that may be encountered should be cleared from areas identified for structures and site grading. Vegetation, other organic material, and debris should be removed from the site. Stripped topsoil should also be removed, or used only as landscape fill in nonstructural areas with slopes less than 25 percent. The stripping depth for sod and highly organic topsoil is anticipated to vary between approximately 6 and 12 inches. Stripping depths may increase in areas of heavy organics or disturbed soil. Actual stripping depths should be determined based upon visual observations made during construction when soil conditions are exposed. The post-construction maximum depth of landscape fill placed or spread at any location onsite should not exceed one foot.

Previously disturbed soil, debris, or unconsolidated fill encountered during grading or construction activities should be removed completely and thoroughly from structural areas. This includes old foundations, basement walls, utilities, associated soft soils, and debris. Excavation areas should be backfilled with engineered structural fill.

Test pits excavated during site exploration were backfilled loosely with onsite soils. These test pits should be located and properly backfilled with structural fill during site improvements construction. Trees, stumps, and associated roots should also be removed from structural areas, individually and carefully. Resulting cavities and excavation areas should be backfilled with engineered structural fill.

Site grading activities should be performed in accordance with requirements specified in the 2015 *International Building Code* (IBC), Chapter 18 and Appendix J, with exceptions noted in the text herein. Site preparation, soil stripping, and grading activities should be observed and documented by Columbia West.

5.1.1 Undocumented Fill

As previously described, undocumented fill was observed in areas proposed for development. Approximate locations where undocumented fill was observed are indicated on Figure 2. The undocumented fill was observed to primarily consist of tan, mottled, moist, medium dense clayey sand with gravel. Undocumented fill extended to an approximate depth of 24 inches in locations observed.

Undocumented fill and other previously disturbed soils or debris are not suitable for bearing structures in their current state and should be removed completely and thoroughly from proposed building envelopes. In some areas, undocumented fill may directly overlie vegetation and the original topsoil layer. This material should also be removed completely. Upon removal of undocumented fill, Columbia West should observe the exposed subgrade to verify adequate support conditions.

Based upon Columbia West’s investigation, most undocumented fill soils (clean clayey sand with gravel) appear to be acceptable for reuse as structural fill, provided materials are observed to exhibit index properties similar to those observed during this investigation and that construction adheres to the specifications presented in this report. Portions of undocumented fill found to contain highly organic soils, debris, or other deleterious material are not suitable for re-use and should be thoroughly removed. Recommendations regarding the suitability of reusing existing fill soils as structural fill material should be provided in the field by Columbia West during construction. It should be noted that the limited scope of exploration conducted for this investigation cannot wholly eliminate uncertainty regarding the presence of unsuitable soils in areas not explored.

5.2 Engineered Structural Fill

Areas proposed for fill placement should be appropriately prepared as described in the preceding text. Surface soils should then be scarified and compacted prior to additional fill placement. Engineered structural fill should be placed in loose lifts not exceeding 12 inches in depth and compacted using standard conventional compaction equipment. The soil moisture content should be within two percentage points of optimum conditions. A field density at least equal to 95 percent of the maximum dry density, obtained from the standard Proctor moisture-density relationship test (ASTM D698), is recommended for structural fill placement. Engineered structural fill placed on sloped grades should be benched to provide a horizontal surface for compaction.

Compaction of engineered structural fill should be verified by nuclear gauge field compaction testing performed in accordance with ASTM D6938. Field compaction testing should be performed for each vertical foot of engineered fill placed followed by subsequent proof-roll evaluation where feasible. Engineered fill placement should be observed by Columbia West.

Engineered structural fill placement activities should be performed during dry summer months if possible. Some clean native soils (Soil Type 2 and Soil Type 4) may be suitable for use as structural fill if adequately dried or moisture-conditioned to achieve recommended compaction specifications. Native soils with a plasticity index greater than 25 should be evaluated and approved by Columbia West prior to re-use as structural fill. Native fat CLAY soils (Soil Type 3) are not anticipated to be suitable for reuse as structural fill.

Fine-textured soils may require addition of moisture during late summer months or after extended periods of warm dry weather. Compacted fine-textured fill soils should be covered shortly after placement. If adequate compaction is not achievable with clean native soils, import structural fill consisting of granular fill meeting WSDOT specifications for *Gravel Borrow 9-03.14(1)* is recommended.

Representative samples of proposed engineered structural fill should be submitted for laboratory analysis and approval by Columbia West prior to placement. Laboratory analyses should include particle-size gradation and standard Proctor moisture-density analysis.

5.3 Cut and Fill Slopes

Fill placed on existing grades steeper than 5H:1V should be horizontally benched at least 10 feet into the slope. Fill slopes greater than six feet in height should be vertically keyed into existing subsurface soil. A typical fill slope cross-section is shown in Figure 3. Drainage implementations, including subdrains or perforated drain pipe trenches, may also be necessary in proximity to cut and fill slopes if seeps or springs are encountered. Drainage design may be performed on a case-by-case basis. Extent, depth, and location of drainage may be determined in the field by Columbia West during construction when soil conditions are exposed. Failure to provide adequate drainage may result in soil sloughing, settlement, or erosion.

Final cut or fill slopes at the site should not exceed 2H:1V or 20 feet in total height without individual slope stability analysis. The values above assume a minimum horizontal setback for loads of 10 feet from top of cut or fill slope face or overall slope height divided by three (H/3), whichever is greater. A minimum slope setback detail for structures is presented in Figure 4.

Concentrated drainage or water flow over the face of slopes should be prohibited, and adequate protection against erosion is required. Fill slopes should be constructed by placing fill material in maximum 12-inch level lifts, compacting as described in Section 5.2, *Engineered Structural Fill* and horizontally benching where appropriate. Fill slopes should be overbuilt, compacted, and trimmed at least two feet horizontally to provide adequate compaction of the outer slope face. Proper cut and fill slope construction is critical to overall project stability and should be observed and documented by Columbia West.

5.4 Foundations

Foundations for proposed structures are anticipated to consist of shallow continuous perimeter or column spread footings. Correspondence with the project structural engineer, Kramer Ghelen and Associates, Inc., indicates that foundation loads are not anticipated to exceed approximately 4 kips per foot for perimeter footings or 75 kips per column. If actual loading exceeds anticipated loading, additional analysis should be conducted for the specific load conditions and proposed footing dimensions. Footings should be designed by a licensed structural engineer and conform to the recommendations below.

The existing ground surface should be prepared as described in Section 5.1, *Site Preparation and Grading*, and Section 5.2, *Engineered Structural Fill*. Foundations should bear only upon firm, native soils (Soil Type 2 or Soil Type 4) or engineered structural fill.

To evaluate bearing capacity for proposed structures, serviceability and reliability of shear resistance for subsurface soils was considered. Allowable bearing capacity is typically a function of footing dimension and subsurface soil properties, including settlement and shear resistance. Based upon in situ field testing and laboratory analysis, an estimated allowable static bearing capacity of 3,000 psf may be achieved by adhering to the following design and construction recommendations. Footings should maintain a minimum embedment depth of 36 inches below the lowest adjacent grade and bear only upon Soil Type 2, Soil Type 4, or engineered structural fill. Soil Types 1 or 3, if encountered within proposed foundation alignments, should be over-excavated to expose Soil Type 2 or 4. Over-excavations which extend beyond the minimum embedment recommendation may be backfilled with 1 ¼”-0 crushed aggregate compacted to at least 95 percent of the modified Proctor maximum dry density (ASTM D1557).

Bearing capacity may be increased by one-third for transient lateral forces such as seismic or wind. The estimated coefficient of friction between in situ compacted native soil or engineered structural fill and in-place poured concrete is 0.40. Lateral forces may also be resisted by an assumed passive soil equivalent fluid pressure of 250 psf/f against embedded footings.

Footings should extend to a depth at least 36 inches below lowest adjacent grade to provide adequate bearing capacity and protection against frost heave. Foundations constructed during wet weather conditions will require over-excavation of saturated subgrade soils and granular structural backfill prior to concrete placement. Over-excavation recommendations should be provided by Columbia West during foundation excavation and construction. Excavations adjacent to foundations should not extend within a 2H:1V angle projected down from the outside bottom footing edge without additional geotechnical analysis.

Foundations should not be permitted to bear upon undocumented fill (Soil Type 1), disturbed soil, or Soil Type 3. Because soil is often heterogeneous and anisotropic, Columbia West should observe foundation excavations prior to placing forms or reinforcing bar to verify subgrade support conditions are as anticipated in this report.

5.4.1 Luminaire, Signal, and Sign Foundations

Foundations for luminaire, signal, and sign poles should be designed in accordance with the *International Building Code (IBC) Chapter 18* by a licensed structural engineer. Based upon review of *IBC* literature, and SPT blow count observations made during the field exploration, the allowable lateral bearing pressure for foundations installed in competent native Soil Type 2, Soil Type 4, or engineered structural fill is 150 psf/ft up to a maximum of 2,500 psf. Columbia West should be contacted to review foundation designs and evaluate compatibility with geotechnical design assumptions.

5.5 Slabs on Grade

The proposed structures may have slab-on-grade floors. Slabs should be supported on firm, competent, in situ native soil or engineered structural fill. Disturbed soils and unsuitable fills in proposed slab locations should be removed and replaced with structural fill.

Preparation and compaction beneath slabs should be performed in accordance with the recommendations presented in Section 5.1, *Site Preparation and Grading* and Section 5.2, *Engineered Structural Fill*. Slabs should be underlain by at least 6 inches of free-draining 1¼" - 0 crushed aggregate meeting WSDOT 9-03.9(3). Geotextile filter fabric conforming to *WSDOT 2010 Standard Specification M 41-10, 9-33.2(1), Geotextile Properties, Table 3: Geotextile for Separation or Soil Stabilization* may be used below the crushed aggregate to increase subgrade support. The modulus of subgrade reaction is estimated to be 100 psi/inch. If desired, a moisture barrier may be constructed beneath the slabs. Slabs should be appropriately waterproofed in accordance with the desired type of finished flooring. Slab thickness and reinforcement should be designed by an experienced structural engineer in accordance with anticipated loads.

5.6 Static Settlement

Total long-term static footing displacement for shallow foundations constructed as described in this report is not anticipated to exceed approximately 1 inch. Differential settlement between comparably loaded footing elements is not expected to exceed approximately ½ inch over a span of 50 feet. The resulting vertical displacement after loading may be due to elastic distortion, dissipation of excess pore pressure, or soil creep.

5.7 Excavation

Soils at the site were explored to a maximum depth of approximately 51 ½ feet using a track-mounted mud-rotary drill system. Blasting or specialized rock-excavation techniques are not anticipated.

Groundwater was not encountered within test pit explorations to the maximum excavated depth of 14 feet bgs. However, perched groundwater layers may exist at shallower depths depending on seasonal fluctuations of the water table.

Based upon laboratory analysis and field testing, near-surface soils may be Washington State Industrial Safety and Health Administration (WISHA) Type C. For temporary open-cut excavations deeper than four feet, but less than 20 feet in soils of these types, the maximum allowable slope is 1.5H:1V. WISHA soil type should be confirmed during field construction activities by the contractor. Soil is often anisotropic and heterogeneous, and it is possible that WISHA soil types determined in the field may differ from those described above.

Site-specific shoring design may be required if open-cut excavations are infeasible or if excavations are proposed adjacent to existing infrastructure. Typical methods for stabilizing excavations consist of soldier piles and timber lagging, sheet pile walls, tiebacks and shotcrete, or pre-fabricated hydraulic shoring. Because lateral earth pressure distributions acting on below-grade structures are dependent upon the type of shoring system used, Columbia West should be contacted to conduct additional analysis when shoring type, excavation depths, and locations are known.

The contractor should be held responsible for site safety, sloping, and shoring. Columbia West is not responsible for contractor activities and in no case should excavation be conducted in excess of all applicable local, state, and federal laws.

5.8 Lateral Earth Pressure

If retaining walls are proposed, lateral earth pressures should be carefully considered in the design process. Hydrostatic pressure and additional surcharge loading should also be considered. Retained material may include engineered structural backfill or undisturbed native soil. Structural wall backfill should consist of imported granular material meeting *Section 9-03.12(2)* of WSDOT Standard Specifications. Backfill should be prepared and compacted to at least 95 percent of maximum dry density as determined by the modified Proctor test (ASTM D1557). Recommended parameters for lateral earth pressures for retained soils and engineered structural backfill consisting of imported granular fill meeting WSDOT specifications for *Gravel Backfill for Walls 9-03.12(2)* are presented in Table 1.

The design parameters presented in Table 1 are valid for static loading cases only and are based upon in situ undistributed native soils or compacted granular fill. The recommended earth pressures do not include surcharge loads, dynamic loading, hydrostatic pressure, or seismic design.

If seismic design is required for unrestrained walls, seismic forces may be calculated by superimposing a uniform lateral force of $10H^2$ pounds per lineal foot of wall, where H is the total wall height in feet. The resultant force should be applied at 0.6H from the base of the wall. If sloped backfill conditions are proposed for the site, Columbia West should be contacted for additional analysis and associated recommendations.

Table 1. Lateral Earth Pressure Parameters for Level Backfill

Retained Soil	Equivalent Fluid Pressure for Level Backfill			Wet Density	Drained Internal Angle of Friction
	At-rest	Active	Passive		
Undisturbed native Sandy Lean CLAY with Gravel (Soil Type 2)	59 pcf	40 pcf	331 pcf	115 pcf	29°
Undisturbed native Fat CLAY with Sand (Soil Type 3)	69 pcf	50 pcf	242 pcf	110 pcf	22°
Undisturbed native Sedimentary CONGLOMERATE (Soil Type 4)	53 pcf	34 pcf	424 pcf	120 pcf	34°
Approved Structural Backfill Material WSDOT 9-03.12(2) compacted aggregate backfill	52 pcf	32 pcf	568 pcf	135 pcf	38°

** The upper 6 inches of soil should be neglected in passive pressure calculations. If exterior grade from top or toe of retaining wall is sloped, Columbia West should be contacted to provide location-specific lateral earth pressures.*

A continuous one-foot-thick zone of free-draining, washed, open-graded 1-inch by 2-inch drain rock and a 4-inch perforated gravity drain pipe is assumed behind retaining walls. Geotextile filter fabric should be placed between the drain rock and backfill soil. Specifications for drain pipe design are presented in Section 5.11, *Drainage*. If walls cannot be gravity drained, saturated base conditions and/or applicable hydrostatic pressures should be assumed.

Final retaining wall design should be reviewed and approved by Columbia West. Retaining wall subgrade and backfill activities should also be observed and tested for compliance with recommended specifications by Columbia West during construction.

5.9 Seismic Design Considerations

According to the *American Society of Civil Engineers* (ASCE) *ASCE 7 Hazard Tool,* the anticipated peak ground and maximum considered earthquake spectral response accelerations resulting from seismic activity for the subject site are summarized in Table 2.

Table 2. Approximate Probabilistic Ground Motion Values for 'firm rock' sites based on subject property longitude and latitude

	2% Probability of Exceedance in 50 yrs
Peak Ground Acceleration	0.367 g
0.2 sec Spectral Acceleration	0.864 g
1.0 sec Spectral Acceleration	0.369 g

The listed probabilistic ground motion values are based upon "firm rock" sites with an assumed shear wave velocity of 2,500 ft/s in the upper 100 feet of soil profile. These values should be adjusted for site class effects by applying site coefficients Fa, Fv, and

F_{PGA} as defined in *ASCE 7-10, Tables 11.4-1, 11.4-2, and 11.8-1*. The site coefficients are intended to more accurately characterize estimated peak ground and respective earthquake spectral response accelerations by considering site-specific soil characteristics and index properties.

The *Site Class Map of Clark County, Washington* (Washington State Department of Natural Resources, 2004) indicates that site soils may represented by Site Class B to C as defined by the *ASCE 7, Chapter 20 Table 20.3-1*. However, subsurface exploration, in situ soil testing, and review of geologic mapping indicates that site soils exhibit characteristics of Site Class D. This site class designation indicates that some amplification of seismic energy may occur during a seismic event because of subsurface conditions.

Localized peak ground accelerations exceeding the adjusted values may occur in some areas in direct proximity to an earthquake's origin. This may be a result of amplification of seismic energy due to depth to competent bedrock, compression and shear wave velocity of bedrock, presence and thickness of loose, unconsolidated alluvial deposits, soil plasticity, grain size, and other factors.

Identification of specific seismic response spectra is beyond the scope of this investigation. If site structures are designed in accordance with recommendations specified in the *2015 IBC*, the potential for peak ground accelerations in excess of the adjusted and amplified values should be understood.

5.10 Soil Liquefaction and Dynamic Settlement

According to the *Liquefaction Susceptibility Map of Clark County Washington* (Washington State Department of Natural Resources, 2004), the site is mapped as very low susceptibility for liquefaction.

Liquefaction, defined as the transformation of the behavior of a granular material from a solid to a liquid due to increased pore-water pressure and reduced effective stress, may occur when granular materials quickly compact under cyclic stresses caused by a seismic event. The effects of liquefaction may include immediate ground settlement and lateral spreading.

Soils most susceptible to liquefaction are generally saturated, cohesionless, loose to medium-dense sands within 50 feet of the ground surface. Recent research has also indicated that low plasticity silts and clays may also be subject to sand-like liquefaction behavior if the plasticity index determined by the Atterberg Limits analysis is less than 8. Potentially liquefiable soils located above the existing, historic, or expected ground water levels do not generally pose a liquefaction hazard. It is important to note that changes in perched ground water elevation may occur due to project development or other factors not observed at the time of investigation.

The above-mentioned criteria were not observed during the geotechnical site investigation. Therefore, the potential for liquefaction of site soils is considered to be very low.

5.11 Drainage

At a minimum, site drainage should include surface water collection and conveyance to properly designed stormwater management structures and facilities. Drainage design in general should conform to City of Camas regulations. Finished site grading should be conducted with positive drainage away from structures. Depressions or shallow areas that may retain ponding water should be avoided. Roof drains, low-point drains, and perimeter foundation drains are recommended for structures. Drains should consist of separate systems and gravity flow with a minimum two-percent slope away from foundations into the stormwater system or approved discharge location.

Perimeter foundation drains should consist of 3-inch perforated PVC pipe surrounded by a minimum of 1 ft^3 of clean, washed drain rock per linear foot of pipe and wrapped with geotextile filter fabric. Open-graded drain rock with a maximum particle size of 3 inches and less than 2 percent passing the No. 200 sieve is recommended. Geotextile filter fabric should consist of Mirafi 140N or approved equivalent, with an apparent opening size (AOS) between No. 70 and No. 100 sieve. The water permittivity should be greater than 1.5/sec. Figure 5 presents a typical perimeter footing drain. Perimeter drains may limit increased hydrostatic pressure beneath footings and assist in reducing potential perched moisture areas.

Subdrains should also be considered if portions of the site are cut below surrounding grades. Shallow groundwater, springs, or seeps should be conveyed via drainage channel or perforated pipe into the stormwater management system or an approved discharge. Recommendations for design and installation of perforated drainage pipe may be performed on a case-by-case basis by Columbia West during construction. Failure to provide adequate surface and sub-surface drainage may result in soil slumping or unanticipated settlement of structures exceeding tolerable limits. A typical perforated drain pipe trench detail is presented in Figure 6.

Foundation drains and subdrains should be closely monitored after construction to assess their effectiveness. If additional surface or shallow subsurface seeps become evident, the drainage provisions may require modification or additional drains. Columbia West should be consulted to provide appropriate recommendations.

5.12 Infiltration Testing Results

To investigate the feasibility of subsurface disposal of stormwater, Columbia West conducted in situ infiltration testing at one location within the project area on November 5, 2019. Results, location, and associated depth of in situ infiltration testing are presented in Table 3. The reported infiltration rate, as defined by the soil coefficient of permeability, reflects approximate raw observed data, without application of a factor of safety. Soils in the tested location were observed and sampled where appropriate to adequately characterize the subsurface profile. Tested native soils were visually classified as CL, sandy lean CLAY with gravel.

Single-ring, falling head infiltration testing was performed by inserting a three-inch diameter pipe into the soil at the noted depth. The test was conducted by filling the apparatus with water and measuring time relative to changes in hydraulic head at regular intervals. Using Darcy's Law for saturated flow in homogenous media, the coefficient of permeability (k) was then calculated.

Table 3. Infiltration Test Data

Test Number	Location (See Figure 2)	Approximate Test Depth (feet bgs)	Approximate Depth to Groundwater on 11-05-19 (feet bgs)	USCS Soil Type (*Indicates Visual Classification)	Passing No. 200 Sieve (%)	Infiltration Rate (Coefficient of Permeability, k) (inches/hour)
IT-1.1	TP-1	3.0	Not Encountered to 14 feet	CL, Sandy Lean CLAY with Gravel*	–	< 0.1

Due to the observed presence of fine-textured, low permeability soils, subsurface disposal of concentrated stormwater is likely infeasible and is not recommended without further study.

5.13 Bituminous Asphalt and Portland Cement Concrete

Correspondence with the design team indicates that proposed development includes private asphalt paved access drives and walkways. Columbia West recommends adherence to City of Camas paving guidelines for roadway improvements in the public right-of-way. General recommendations for private onsite flexible pavement sections are summarized in Table 4.

Table 4. Private Onsite Flexible Pavement Section Recommendations

Pavement Section Layer	Minimum Layer Thickness		Specifications
	Passenger Vehicle Parking and Access Drives	*Heavy Truck Access Drives	
Asphalt concrete surface HMA Class ½" PG 64-22	3 inches	4 inches	91 percent of maximum Rice density (ASTM D2041)
Base course (WSDOT 9-03.9(3) 1¼"-0 crushed aggregate	8 inches	12 inches	95 percent of maximum modified Proctor density (ASTM D1557)
Scarified and compacted existing subgrade material	12 inches	12 inches	Compacted to 95 percent of maximum modified Proctor density (ASTM D1557)

*General recommendation based upon maximum traffic loading of up to 15 heavy trucks per day. If actual truck traffic exceeds 15 trucks per day, reduced pavement serviceability and design life should be expected.

For dry weather construction, pavement surface sections should bear upon competent subgrade consisting of scarified and compacted native soil or engineered structural fill. Wet weather pavement construction is discussed in Section 5.14, *Wet Weather Construction Methods and Techniques*. Subgrade conditions should be evaluated and tested by Columbia West prior to placement of crushed aggregate base. Subgrade evaluation should include nuclear gauge density testing and wheel proof-roll observations conducted with a loaded 12-cubic yard, double-axle dump truck or equivalent. Nuclear gauge density testing should be conducted at 150-foot intervals or as determined by the onsite geotechnical engineer. Subgrade soil should be compacted to at least 95 percent of the modified Proctor dry density, as determined by ASTM D1557. Areas of observed deflection or rutting during proof-roll evaluation should be excavated to a firm surface and replaced with compacted crushed aggregate.

Crushed aggregate base should be compacted and tested in accordance with the specifications outlined above. Asphalt concrete pavement should be compacted to at least 91 percent of maximum Rice density. Nuclear gauge density testing should be conducted to verify adherence to recommended specifications. Testing frequency should be in accordance with Washington Department of Transportation and City of Camas specifications.

Portland cement concrete curbs and sidewalks should be installed in accordance with City of Camas specifications. Curb and sidewalk aggregate base should be observed and proof-rolled by Columbia West. Soft areas that deflect or rut should be stabilized prior to pouring concrete. Concrete should be tested during installation in accordance with ASTM C171, C138, C231, C143, C1064, and C31. This includes casting of cylinder specimens at a frequency of four cylinders per 100 cubic yards of poured concrete. Recommended field concrete testing includes slump, air entrainment, temperature, and unit weight.

5.14 Wet Weather Construction Methods and Techniques

Wet weather construction often results in significant shear strength reduction and soft areas that may rut or deflect. Installation of granular working layers may be necessary to provide a firm support base and sustain construction equipment. Granular layers should consist of all-weather gravel, two- to four-inch gabion, or other similar material (six-inch maximum size with less than five percent passing the No. 200 sieve).

Construction equipment traffic across exposed soil should be minimized. Equipment traffic induces dynamic loading, which may result in weak areas and significant reduction in shear strength for wet soils. Wet weather construction may also result in generation of significant excess quantities of soft wet soil. This material should be removed from the site or stockpiled in a designated area.

Construction during wet weather conditions may require increased base thickness. Over-excavation of subgrade soils or subgrade amendment with lime and/or cement may be necessary to provide a firm base upon which to place crushed aggregate. Geotextile filter fabric is also recommended. If soil amendment with lime or cement is considered, Columbia West should be contacted to provide appropriate recommendations based upon observed field conditions and desired performance criteria.

Crushed aggregate base should be installed in a single lift with trucks end-dumping from an advancing pad of granular fill. During extended wet periods, stripping activities may also need to be conducted from an advancing pad of granular fill. Once installed, the crushed aggregate base should be compacted with several passes from a static drum roller. A vibratory compactor is not recommended because it may further disturb the subgrade. Subdrains may also be necessary to provide subgrade drainage and maintain structural integrity.

Crushed aggregate base should be compacted to at least 95 percent of maximum dry density according to the modified Proctor density test (ASTM D1557). Compaction should be verified by nuclear gauge density testing. Observation of a proof-roll with a loaded dump truck is also recommended as an indication of the compacted aggregate's performance.

It should be understood that wet weather construction is risky and costly. Columbia West should observe and document wet weather construction activities. Proper construction methods and techniques are critical to overall project integrity.

5.15 Erosion Control Measures

Based upon field observations and laboratory testing, the erosion hazard for site soils in flat to shallow-gradient portions of the property is likely to be low. The potential for erosion generally increases in sloped areas. Therefore, disturbance to vegetation in sloped areas should be minimized during construction activities. Soil is also prone to erosion if unprotected and unvegetated during periods of increased precipitation. Erosion can be minimized by performing construction activities during dry summer months.

Site-specific erosion control measures should be implemented to address the maintenance of exposed areas. This may include silt fence, biofilter bags, straw wattles, or other suitable methods. During construction activities, exposed areas should be well-compacted and protected from erosion with visqueen, surface tackifier, or other means, as appropriate. Temporary slopes or exposed areas may be covered with straw, crushed aggregate, or riprap in localized areas to minimize erosion. Erosion and water runoff during wet weather conditions may be controlled by application of strategically placed channels and small detention depressions with overflow pipes.

After grading, exposed surfaces should be vegetated as soon as possible with erosion-resistant native vegetation. Jute mesh or straw may be applied to enhance vegetation. Once established, vegetation should be properly maintained. Disturbance to existing native vegetation and surrounding organic soil should also be minimized during construction activities.

5.16 Utility Installation

Utility installation may require subsurface excavation and trenching. Excavation, trenching and shoring should conform to federal (Occupational Safety and Health Administration) (OSHA) (29 CFR, Part 1926) and *WISHA* (WAC, Chapter 296-155) regulations. Site soils may slough when cut vertically and sudden precipitation events or perched groundwater may result in accumulation of water within excavation zones and trenches.

Utilities should be installed in general accordance with manufacturer's recommendations. Utility trench backfill should consist of *WSDOT 9-03.19 Bank Run Gravel for Trench Backfill* or *WSDOT 9-03.14(2) Select Borrow* with a maximum particle size of 2 ½-inches. Trench backfill material within 18 inches of the top of utility pipes should be hand compacted (i.e., no heavy compaction equipment). The remaining backfill should be compacted to at least 95 percent of maximum dry density as determined by the standard Proctor moisture-density test (ASTM D698). Clean, free-draining, fine bedding sand is recommended for use in the pipe zone. With exception of the pipe zone, backfill should be placed in loose lifts not exceeding 12 inches in thickness.

Compaction of utility trench backfill material should be verified by nuclear gauge field compaction testing performed in accordance with ASTM D6938. It is recommended that field compaction testing be performed at 200-foot intervals along the utility trench centerline at the surface and midpoint depth of the trench. Compaction frequency and specifications may be modified for non-structural areas in accordance with recommendations of the site geotechnical engineer.

6.0 CONCLUSION AND LIMITATIONS

This geotechnical site investigation report was prepared in accordance with accepted standard conventional principles and practices of geotechnical engineering. This investigation pertains only to material tested and observed as of the date of this report, and is based upon proposed site development as described in the text herein. This report is a professional opinion containing recommendations established by engineering interpretations of subsurface soils based upon conditions observed during site exploration. Soil conditions may differ between tested locations or over time. Slight variations may produce impacts to the performance of structural facilities if not adequately addressed. This underscores the importance of diligent QA/QC construction observation and testing to verify soil conditions are as anticipated in this report.

Therefore, this report contains several recommendations for field observation and testing by Columbia West personnel during construction activities. Columbia West cannot accept responsibility for deviations from recommendations described in this report. Future performance of structural facilities is often related to the degree of construction observation by qualified personnel. These services should be performed to the full extent recommended.

This report is not an environmental assessment and should not be construed as a representative warranty of site subsurface conditions. The discovery of adverse environmental conditions, or subsurface soils that deviate from those described in this report, should immediately prompt further investigation. The above statements are in lieu of all other statements expressed or implied.

This report was prepared solely for the client and is not to be reproduced without prior authorization from Columbia West. Final engineering plans and specifications for the project should be reviewed and approved by Columbia West as they relate to geotechnical and grading issues prior to final design approval. Columbia West is not responsible for independent conclusions or recommendations made by other parties based upon information presented in this report. Unless a particular service was expressly included in the scope, it was not performed and there should be no assumptions based upon services not provided. Additional report limitations and important information about this document are presented in Appendix E. This information should be carefully read and understood by the client and other parties reviewing this document.

Sincerely,

COLUMBIA WEST ENGINEERING, Inc.

LANCE V. LEHTO
STATE OF WASHINGTON
36301
REGISTERED
PROFESSIONAL ENGINEER

12-20-19

Lance V. Lehto, PE, GE
President

REFERENCES

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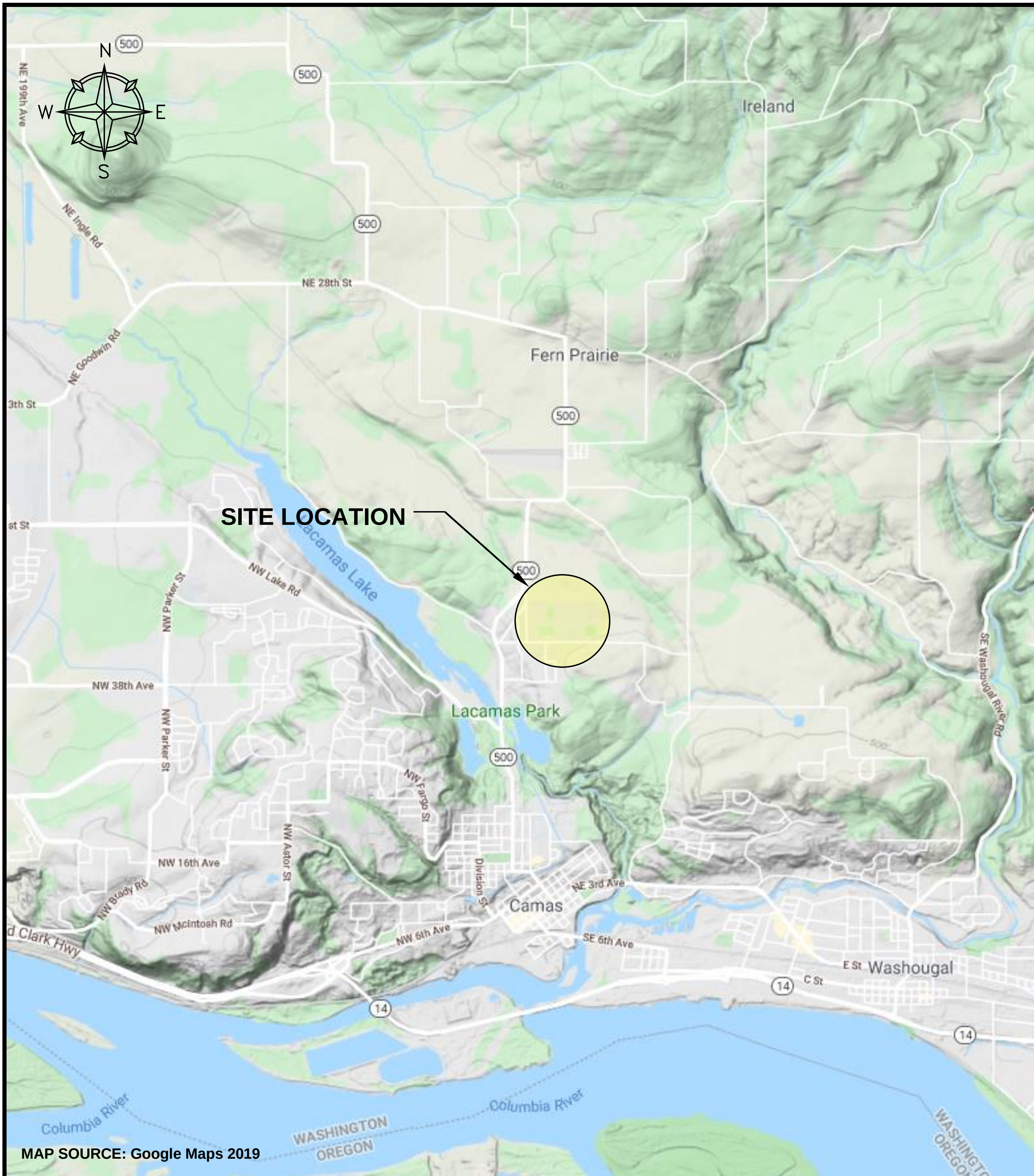
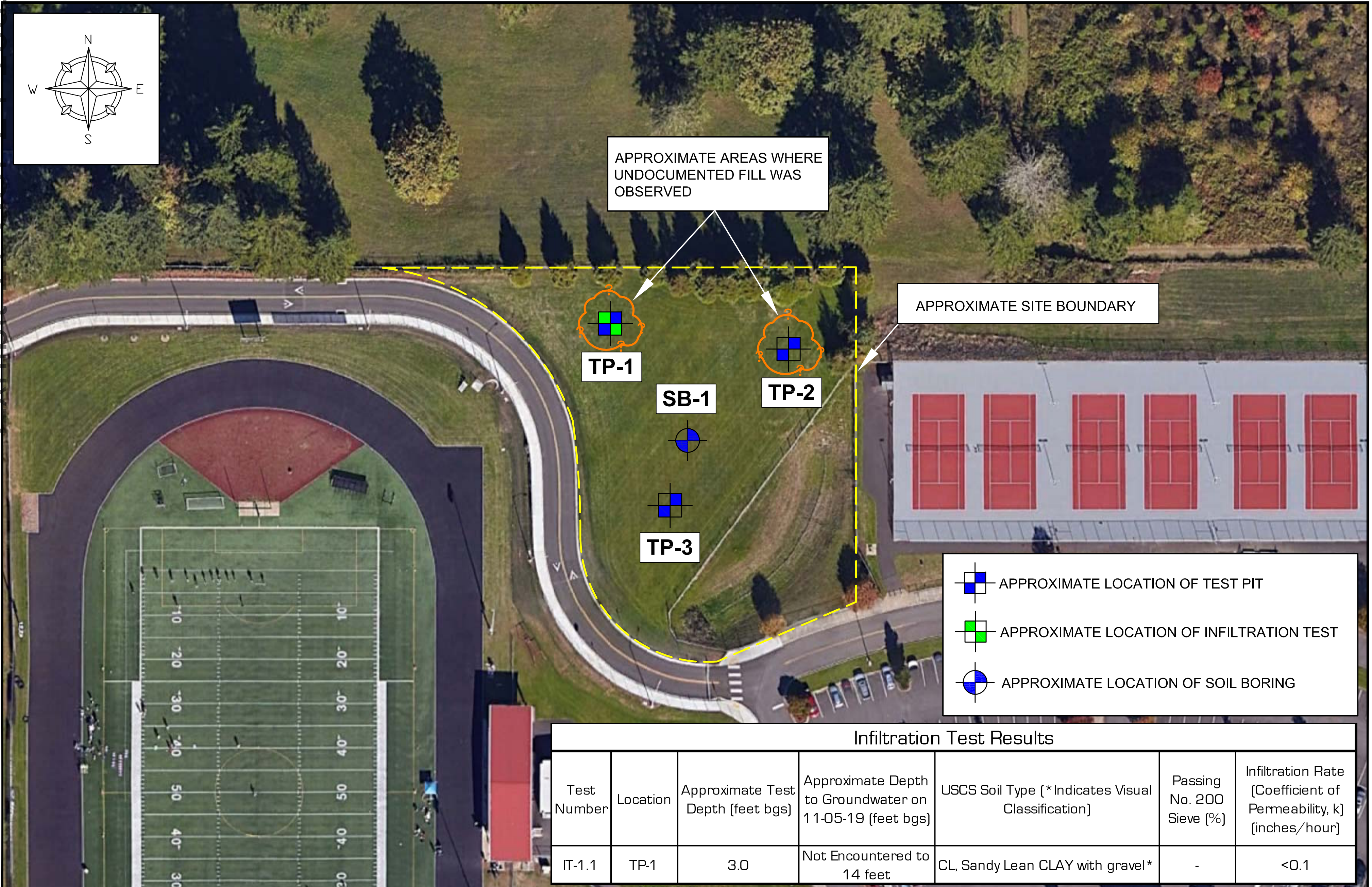
FIGURES

Exhibit 11 CUP24-1001



Geotechnical ▪ Environmental ▪ Special Inspections **Columbia West** Engineering, Inc 11917 NE 95th Street Vancouver, Washington 98682 Phone: 360-823-2900, Fax: 360-823-2901 www.columbiawestengineering.com	Design	Drawn: MCK	**SITE LOCATION MAP**	FIGURE 1
	Checked: GLW	Date: 11/18/19	CAMAS HIGH SCHOOL FIELD HOUSE CAMAS, WASHINGTON	
	Client: ROBERTSON	Rev / By / Date		
	Job No.: 19276			
	CAD File: FIGURE 1			
	Scale: NTS			



Infiltration Test Results

Test Number	Location	Approximate Test Depth (feet bgs)	Approximate Depth to Groundwater on 11-05-19 (feet bgs)	USCS Soil Type (*Indicates Visual Classification)	Passing No. 200 Sieve (%)	Infiltration Rate (Coefficient of Permeability, k) (inches/hour)
IT-1.1	TP-1	3.0	Not Encountered to 14 feet	CL, Sandy Lean CLAY with gravel*	-	<0.1

NOTES:
1. SITE LOCATION: 26600 SE 15TH STREET, CAMAS, WASHINGTON.
2. SITE CONSISTS OF A PORTION OF PARCEL 178111000 TOTALING APPROXIMATELY 1.15 ACRES.
3. DRAWING IS NOT TO SCALE.
4. AERIAL IMAGE SOURCED FROM GOOGLE EARTH.
5. EXPLORATION LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
6. TEST PITS BACKFILLED LOOSELY WITH ONSITE SOIL ON NOVEMBER 5, 2019.
7. SOIL BORING BACKFILLED WITH BENTONITE ON NOVEMBER 11, 2019.
8. INFILTRATION RATES ARE APPROXIMATE COEFFICIENTS OF PERMEABILITY AND DO NOT INCLUDE A FACTOR OF SAFETY.

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Design:
Checked: GLW
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Job No: 19276
CAD File: FIGURE 2
Scale: NONE
Drawn: MCK
Date: 11/11/19

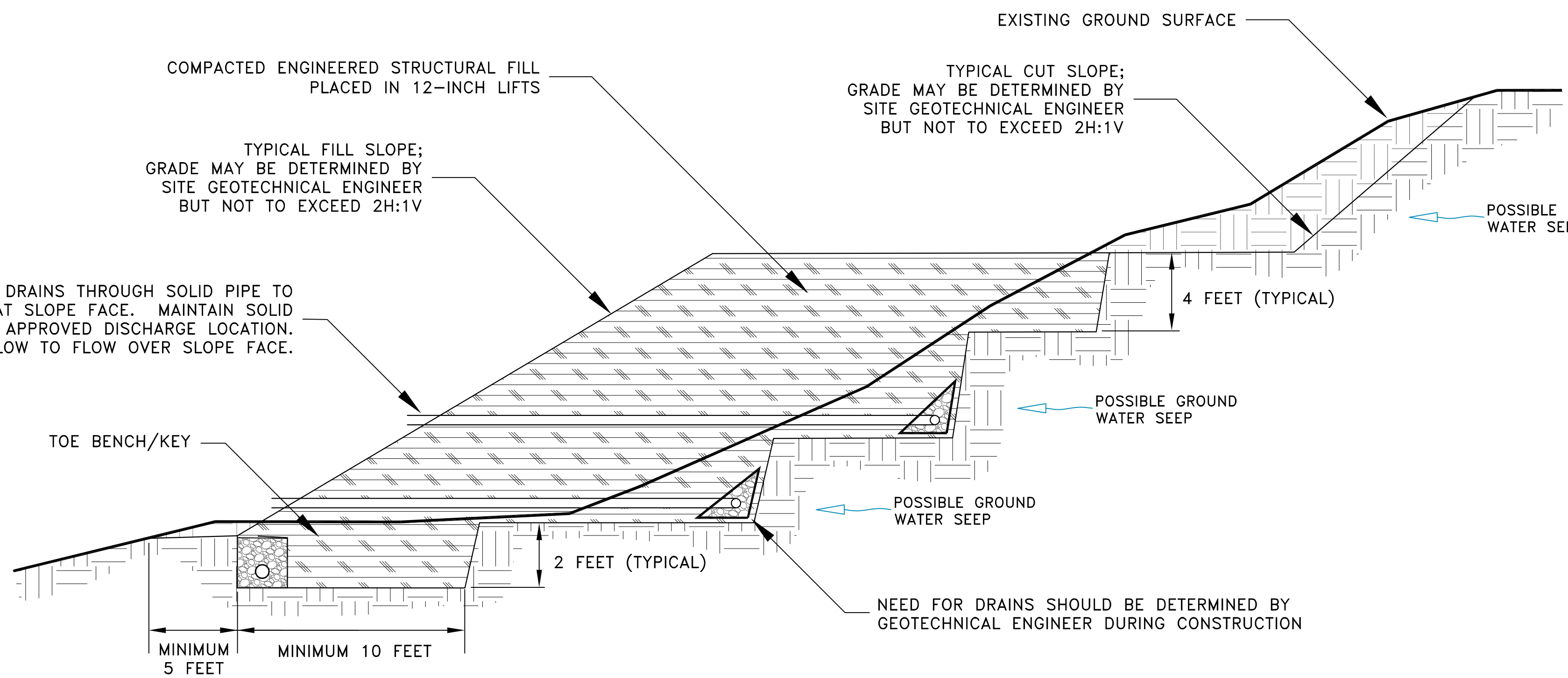
Rev	By	Date

EXPLORATION LOCATION MAP

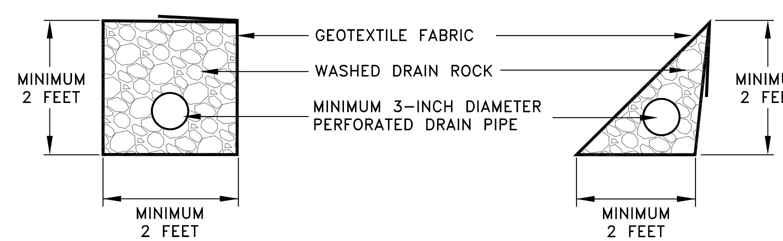
CAMAS HIGH SCHOOL FIELD HOUSE
CAMAS, WASHINGTON

FIGURE 2

TYPICAL CUT AND FILL SLOPE CROSS-SECTION



TYPICAL DRAIN SECTION DETAIL



DRAIN SPECIFICATIONS

GEOTEXTILE FABRIC SHALL CONSIST OF MIRAFI 140N OR APPROVED EQUIVALENT WITH AOS BETWEEN No. 70 AND No. 100 SIEVE.

WASHED DRAIN ROCK SHALL BE OPEN-GRADED ANGULAR DRAIN ROCK WITH LESS THAN 2 PERCENT PASSING THE No. 200 SIEVE AND A MAXIMUM PARTICLE SIZE OF 3 INCHES.

NOTES:
1. DRAWING IS NOT TO SCALE.
2. SLOPES AND PROFILES SHOWN ARE APPROXIMATE.
3. DRAWING REPRESENTS TYPICAL FILL AND CUT SLOPE SECTION, AND MAY NOT BE SITE-SPECIFIC.

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Design:	Drawn: MCK			TYPICAL CUT AND FILL SLOPE CROSS-SECTION
Checked: GLW	Date: 11/18/19			
Client: ROBERTSON	Rev	By	Date	
Job No: 19276				CAMAS HIGH SCHOOL FIELD HOUSE CAMAS, WASHINGTON
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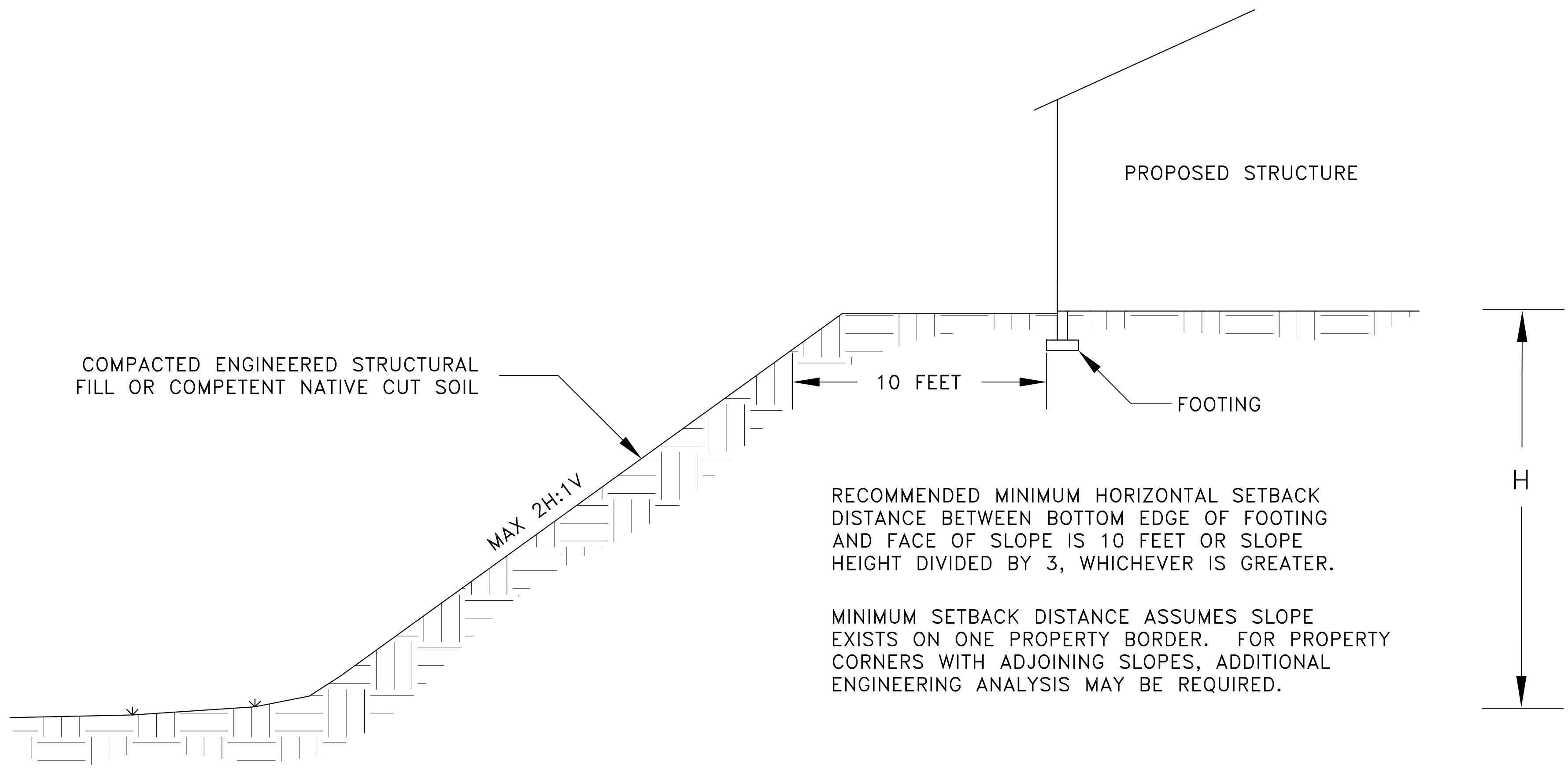
Scale: NONE				

FIGURE 3

MINIMUM FOUNDATION SLOPE SETBACK DETAIL



RECOMMENDED MINIMUM HORIZONTAL SETBACK DISTANCE BETWEEN BOTTOM EDGE OF FOOTING AND FACE OF SLOPE IS 10 FEET OR SLOPE HEIGHT DIVIDED BY 3, WHICHEVER IS GREATER.

MINIMUM SETBACK DISTANCE ASSUMES SLOPE EXISTS ON ONE PROPERTY BORDER. FOR PROPERTY CORNERS WITH ADJOINING SLOPES, ADDITIONAL ENGINEERING ANALYSIS MAY BE REQUIRED.

NOTES:
1. DRAWING IS NOT TO SCALE.
2. SLOPES AND PROFILES SHOWN ARE APPROXIMATE.
3. DRAWING REPRESENTS TYPICAL FOUNDATION SETBACK DETAIL, AND MAY NOT BE SITE-SPECIFIC.

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Design:	Drawn: MCK			
Checked: GLW	Date: 11/18/19			
Client: ROBERTSON	Rev	By	Date	
Job No: 19276				
CAD File: FIGURE 4				
Scale: NONE				

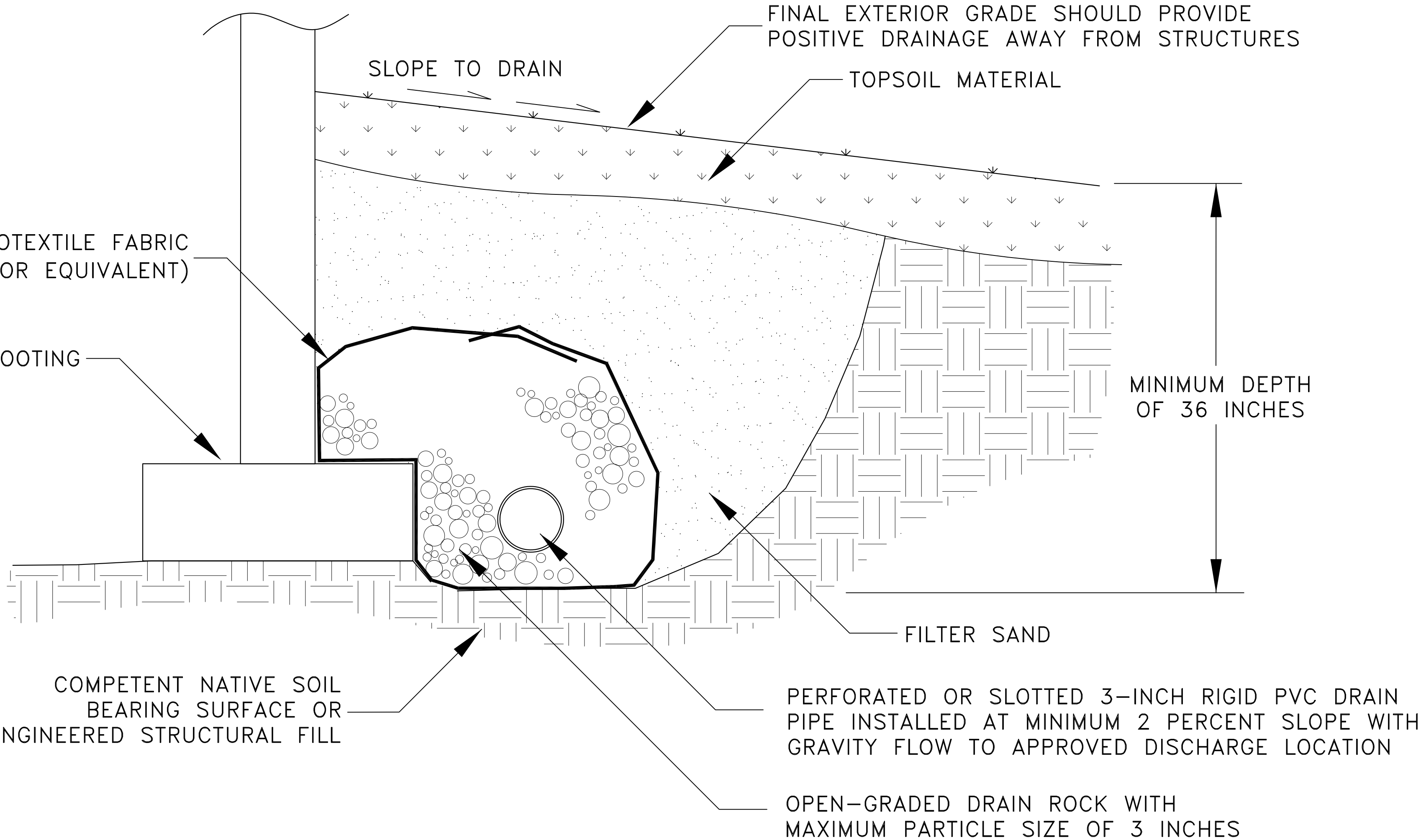
TYPICAL MINIMUM SLOPE SETBACK DETAIL

CAMAS HIGH SCHOOL FIELD HOUSE
CAMAS, WASHINGTON

FIGURE 4

Exhibit 11 CUP24-1001

TYPICAL PERIMETER FOOTING DRAIN DETAIL



NOTES:
1. DRAWING IS NOT TO SCALE.
2. DRAWING REPRESENTS TYPICAL FOOTING DRAIN DETAIL AND MAY NOT BE SITE–SPECIFIC.

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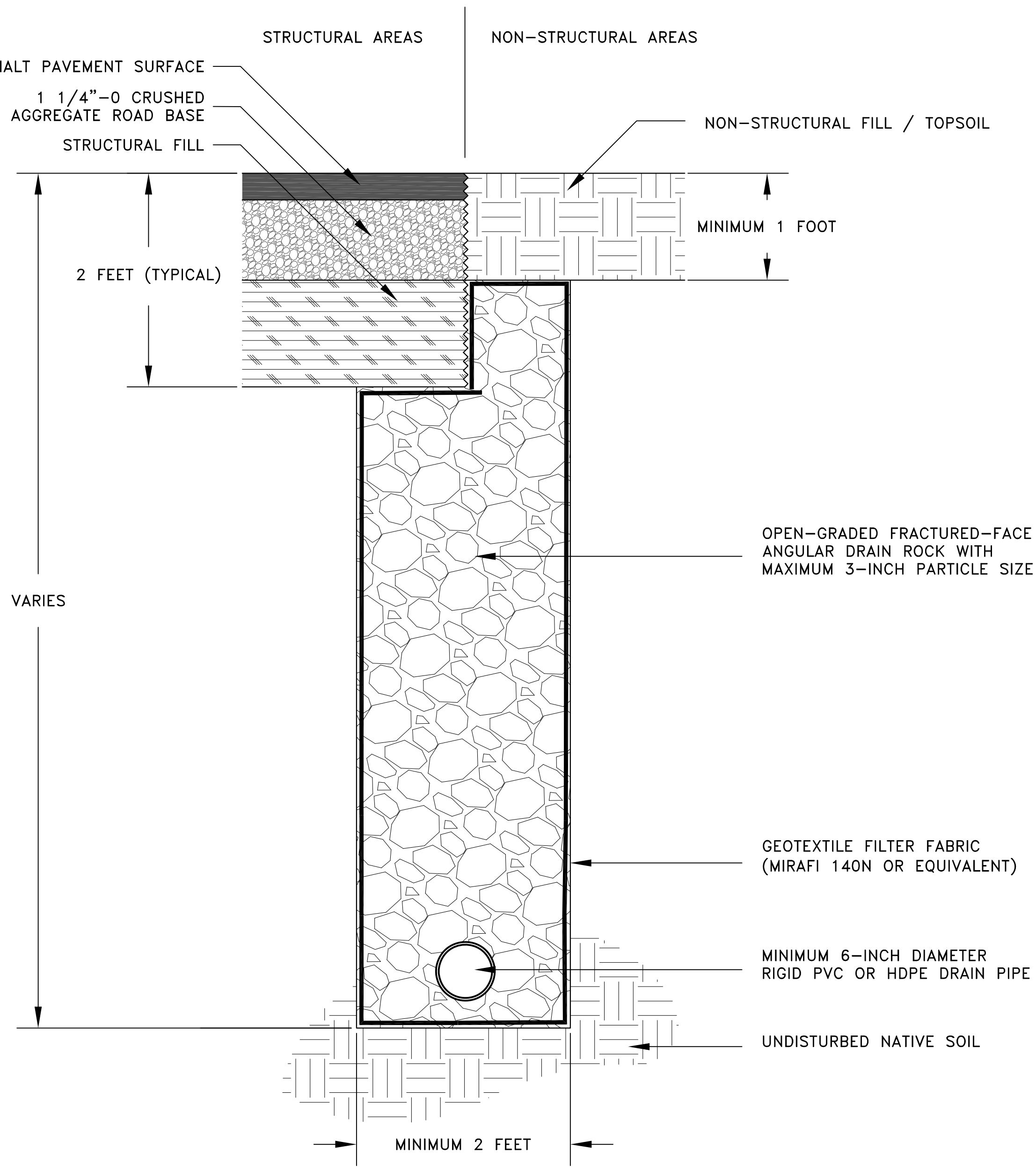
Design:	Drawn: MCK			TYPICAL PERIMETER FOOTING DRAIN DETAIL	FIGURE 5
Checked: GLW	Date: 11/18/19				
Client: ROBERTSON	Rev	By	Date		
Job No: 19276				CAMAS HIGH SCHOOL FIELD HOUSE CAMAS, WASHINGTON	
CAD File: FIGURE 5					
Scale: NONE					

TYPICAL PERFORATED DRAIN PIPE TRENCH DETAIL



NOTE: LOCATION, INVERT ELEVATION, DEPTH OF TRENCH, AND EXTENT OF PERFORATED PIPE REQUIRED MAY BE MODIFIED BY THE GEOTECHNICAL ENGINEER DURING CONSTRUCTION BASED UPON FIELD OBSERVATION AND SITE-SPECIFIC SOIL CONDITIONS.

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Design:	Drawn: MCK			TYPICAL PERFORATED DRAIN PIPE TRENCH DETAIL
Checked: GLW	Date: 11/18/19			
Client: ROBERTSON	Rev	By	Date	
Job No: 19276				CAMAS HIGH SCHOOL FIELD HOUSE CAMAS, WASHINGTON
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Scale: NONE				

FIGURE 6

APPENDIX A
LABORATORY TEST RESULTS

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Camas High School Field House 26600 SE 15th Street Camas, Washington	Robertson Engineering, PC 1101 Broadway Street, Suite 201 Vancouver, Washington 98660	19276	S19-1115
		REPORT DATE: 11/22/19	FIELD ID: TP2.1
		DATE SAMPLED: 11/05/19	SAMPLED BY: MCK

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Fat CLAY with Sand	Test Pit TP-02 depth = 2 feet	CH, Fat Clay with Sand

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-7-6(47)

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter 637	ASTM D6913

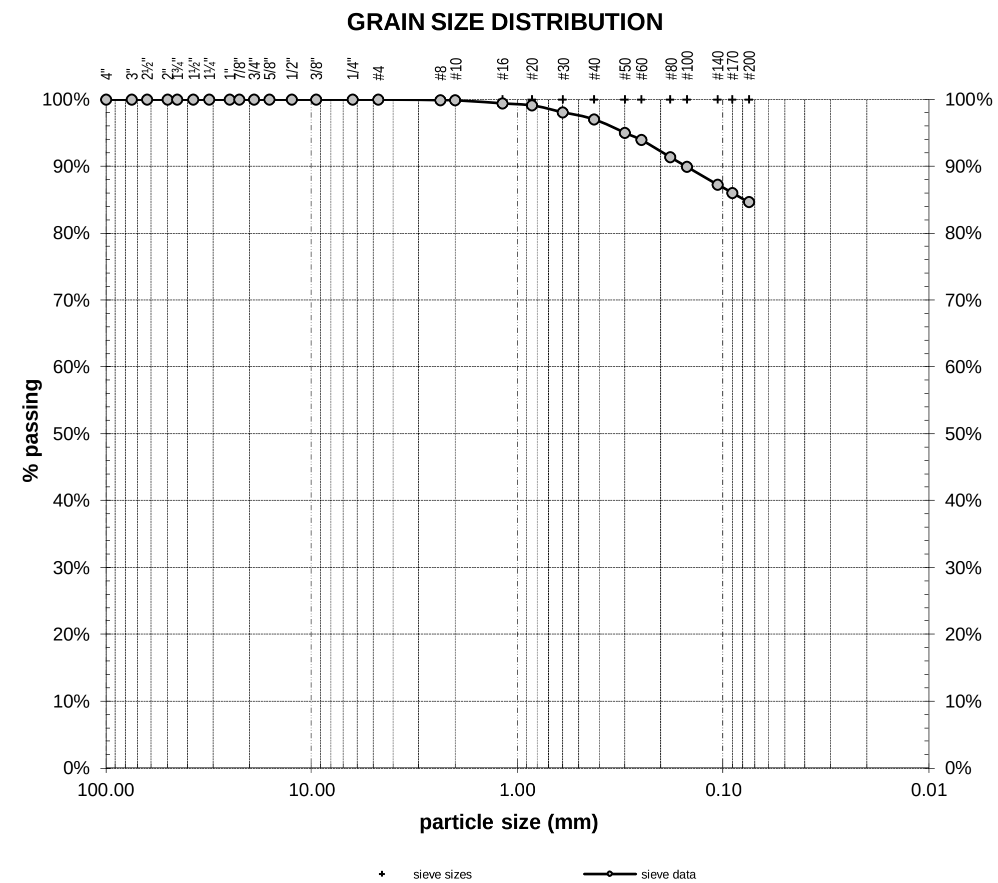
ADDITIONAL DATA

initial dry mass (g) =	159.83		
as-received moisture content =	40.1%	coefficient of curvature, C_C =	n/a
liquid limit =	76	coefficient of uniformity, C_U =	n/a
plastic limit =	26	effective size, $D_{(10)}$ =	n/a
plasticity index =	50	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	n/a

SIEVE DATA

% gravel = 0.0%
% sand = 15.3%
% silt and clay = 84.7%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0		100%		
	3.00"	75.0		100%		
	2.50"	63.0		100%		
	2.00"	50.0		100%		
	1.75"	45.0		100%		
	1.50"	37.5		100%		
	1.25"	31.5		100%		
	1.00"	25.0		100%		
	7/8"	22.4		100%		
	3/4"	19.0		100%		
	5/8"	16.0		100%		
	1/2"	12.5		100%		
	3/8"	9.50		100%		
	1/4"	6.30		100%		
	#4	4.75	100%			
SAND	#8	2.36		100%		
	#10	2.00	100%			
	#16	1.18		99%		
	#20	0.850	99%			
	#30	0.600		98%		
	#40	0.425	97%			
	#50	0.300		95%		
	#60	0.250	94%			
	#80	0.180		91%		
	#100	0.150	90%			
	#140	0.106		87%		
	#170	0.090		86%		
	#200	0.075	85%			



DATE TESTED	TESTED BY
11/19/19	BTT

COLUMBIA WEST ENGINEERING, INC. authorized signature

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Columbia West Engineering, Inc

ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Camas High School Field House 26600 SE 15th Street Camas, Washington	Robertson Engineering, PC 1101 Broadway Street, Suite 201 Vancouver, Washington 98660	19276	S19-1115
		REPORT DATE: 11/22/19	FIELD ID: TP2.1
		DATE SAMPLED: 11/05/19	SAMPLED BY: MCK

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Fat CLAY with Sand	Test Pit TP-02 depth = 2 feet	CH, Fat Clay with Sand

LABORATORY TEST DATA

LABORATORY EOUIPMENT	TEST PROCEDURE
Liquid Limit Machine, Hand Rolled	ASTM D4318

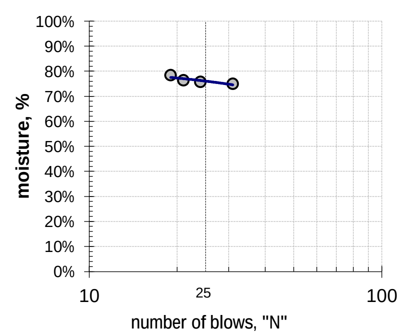
ATTERBERG LIMITS

liquid limit = 76
plastic limit = 26
plasticity index = 50

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	32.21	31.56	31.49	31.50
dry soil + pan weight, g =	27.37	26.91	26.78	26.85
pan weight, g =	20.91	20.77	20.61	20.92
N (blows) =	31	24	21	19
moisture, % =	74.9 %	75.7 %	76.3 %	78.4 %

LIQUID LIMIT



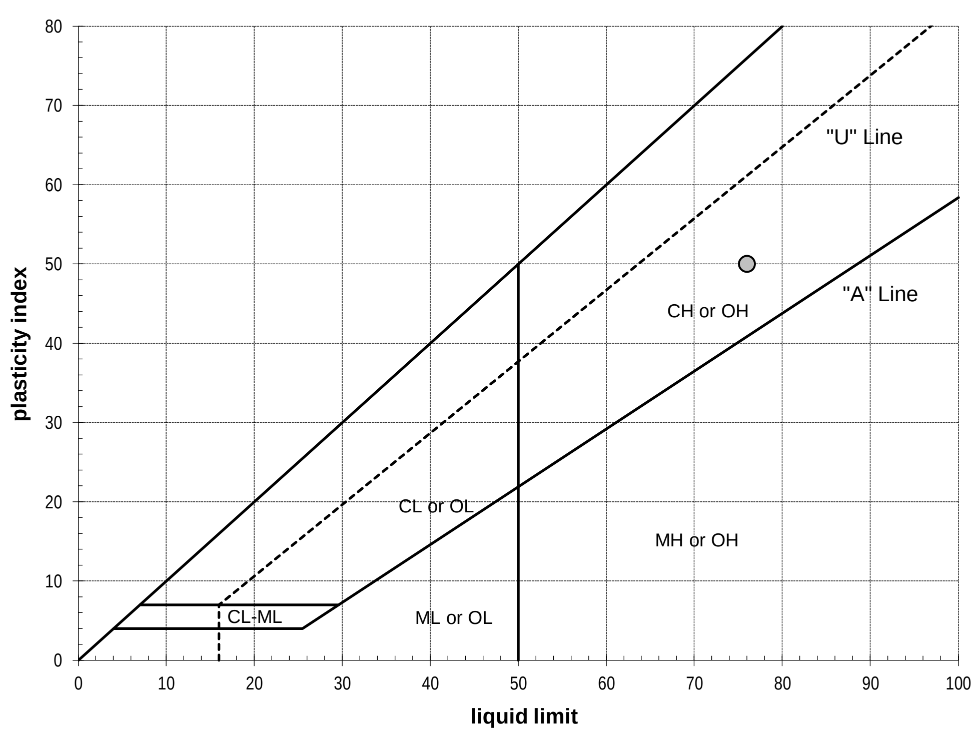
SHRINKAGE

shrinkage limit = n/a
shrinkage ratio = n/a

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.15	27.23		
dry soil + pan weight, g =	25.85	25.93		
pan weight, g =	20.74	20.87		
moisture, % =	25.4 %	25.7 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 0.0%
% sand = 15.3%
% silt and clay = 84.7%
% silt = n/a
% clay = n/a
moisture content = 40.1%

DATE TESTED	TESTED BY
11/21/19	KMS

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Camas High School Field House 26600 SE 15th Street Camas, Washington	Robertson Engineering, PC 1101 Broadway Street, Suite 201 Vancouver, Washington 98660	19276	S19-1116
		REPORT DATE: 11/22/19	FIELD ID: TP2.3
		DATE SAMPLED: 11/05/19	SAMPLED BY: MCK

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Poorly graded GRAVEL with Silt and Sand	Test Pit TP-02 depth = 9 feet	GP-GM, Poorly graded gravel with silt and sand

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-2-7(0)

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter 637	ASTM D6913

ADDITIONAL DATA

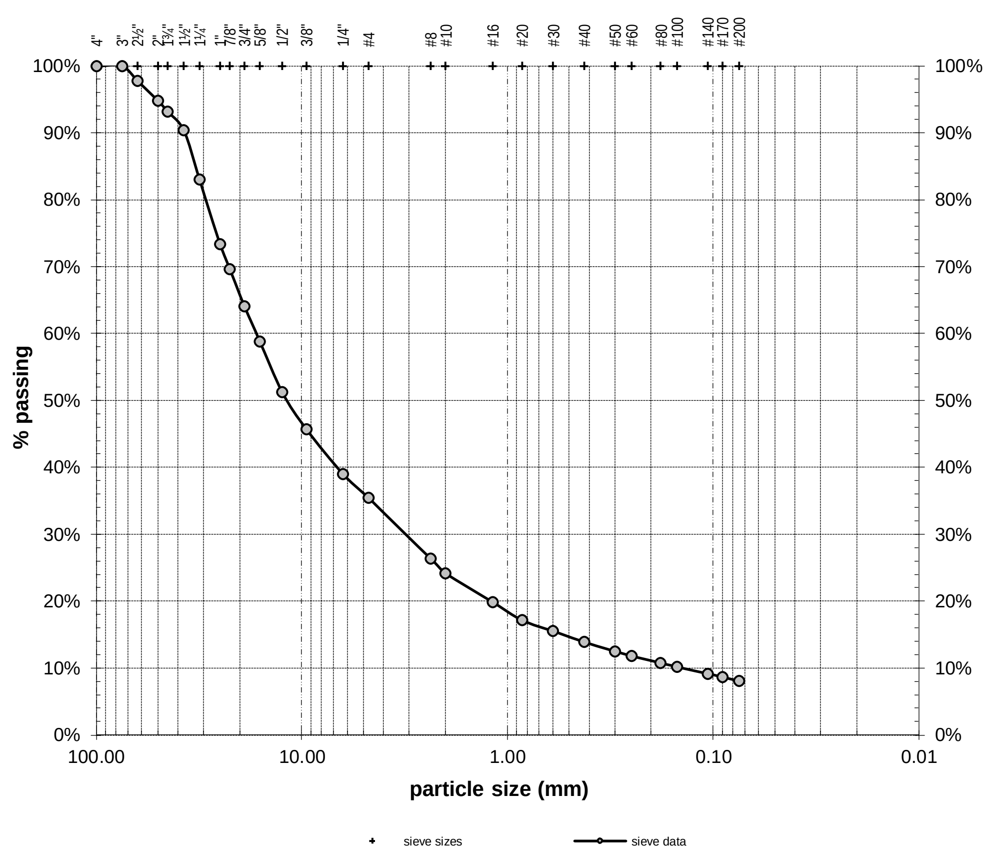
initial dry mass (g) =	17836.8		
as-received moisture content =	18.7%	coefficient of curvature, C_C =	4.19
liquid limit =	47	coefficient of uniformity, C_U =	118.56
plastic limit =	29	effective size, $D_{(10)}$ =	0.140 mm
plasticity index =	18	$D_{(30)}$ =	3.122 mm
fineness modulus =	n/a	$D_{(60)}$ =	16.612 mm

SIEVE DATA

% gravel = 64.6%
% sand = 27.3%
% silt and clay = 8.1%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0		100%		
	3.00"	75.0	100%			
	2.50"	63.0		98%		
	2.00"	50.0	95%			
	1.75"	45.0		93%		
	1.50"	37.5	90%			
	1.25"	31.5		83%		
	1.00"	25.0	73%			
	7/8"	22.4		70%		
	3/4"	19.0	64%			
	5/8"	16.0		59%		
	1/2"	12.5	51%			
	3/8"	9.50	46%			
	1/4"	6.30	39%			
	#4	4.75	35%			
SAND	#8	2.36		26%		
	#10	2.00	24%			
	#16	1.18		20%		
	#20	0.850	17%			
	#30	0.600		16%		
	#40	0.425	14%			
	#50	0.300		13%		
	#60	0.250	12%			
	#80	0.180		11%		
	#100	0.150	10%			
	#140	0.106		9%		
	#170	0.090		9%		
	#200	0.075	8%			

GRAIN SIZE DISTRIBUTION



DATE TESTED	TESTED BY
11/19/19	BTT

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Camas High School Field House 26600 SE 15th Street Camas, Washington	Robertson Engineering, PC 1101 Broadway Street, Suite 201 Vancouver, Washington 98660	19276	S19-1116
		REPORT DATE: 11/22/19	FIELD ID: TP2.3
		DATE SAMPLED: 11/05/19	SAMPLED BY: MCK

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Poorly graded GRAVEL with Silt and Sand	Test Pit TP-02 depth = 9 feet	GP-GM, Poorly graded gravel with silt and sand

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Liquid Limit Machine, Hand Rolled	ASTM D4318

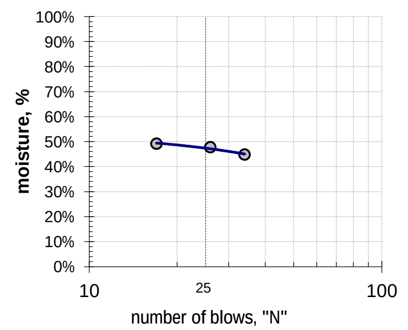
ATTERBERG LIMITS

liquid limit = 47
plastic limit = 29
plasticity index = 18

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	34.55	34.45	34.82	
dry soil + pan weight, g =	30.30	30.04	30.22	
pan weight, g =	20.80	20.79	20.86	
N (blows) =	34	26	17	
moisture, % =	44.7 %	47.7 %	49.2 %	

LIQUID LIMIT



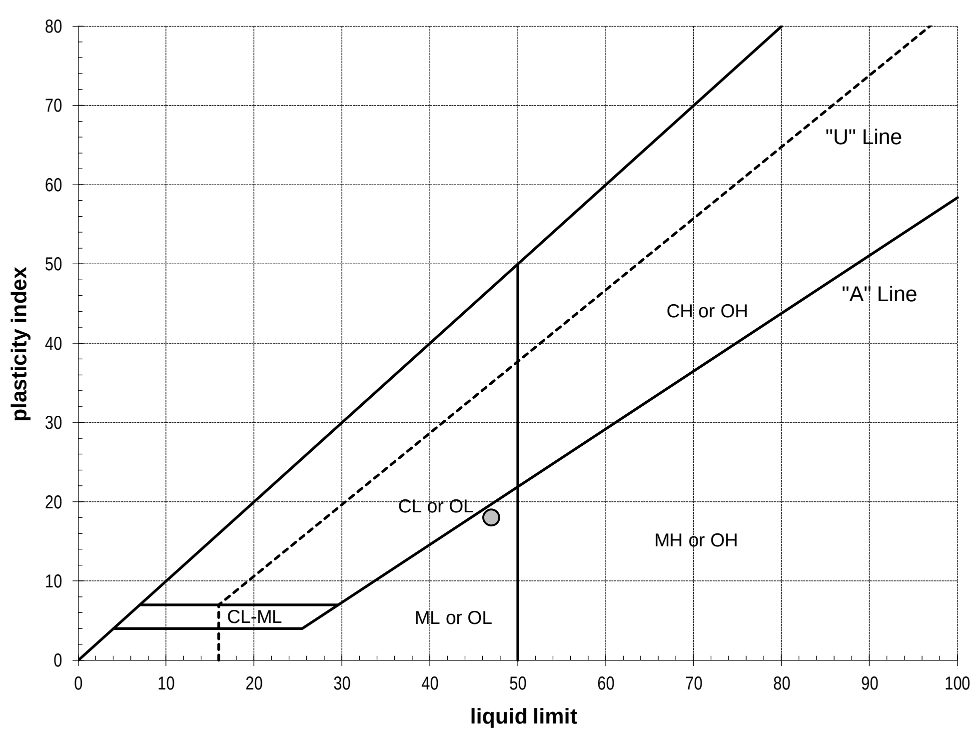
SHRINKAGE

shrinkage limit = n/a
shrinkage ratio = n/a

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.60	27.15		
dry soil + pan weight, g =	26.05	25.67		
pan weight, g =	20.75	20.60		
moisture, % =	29.3 %	29.2 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 64.6%
% sand = 27.3%
% silt and clay = 8.1%
% silt = n/a
% clay = n/a
moisture content = 18.7%

DATE TESTED	TESTED BY
11/21/19	KMS

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Camas High School Field House 26600 SE 15th Street Camas, Washington	Robertson Engineering, PC 1101 Broadway Street, Suite 201 Vancouver, Washington 98660	19276	S19-1109
		REPORT DATE: 11/20/19	FIELD ID: SB1.9
		DATE SAMPLED: 11/11/19	SAMPLED BY: MCK

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Silty SAND	Soil Boring SB-01 depth = 35 feet	SM, Silty Sand

SPECIFICATIONS	AASHTO SOIL TYPE
none	A-7-5(5)

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter 637	ASTM D6913

ADDITIONAL DATA

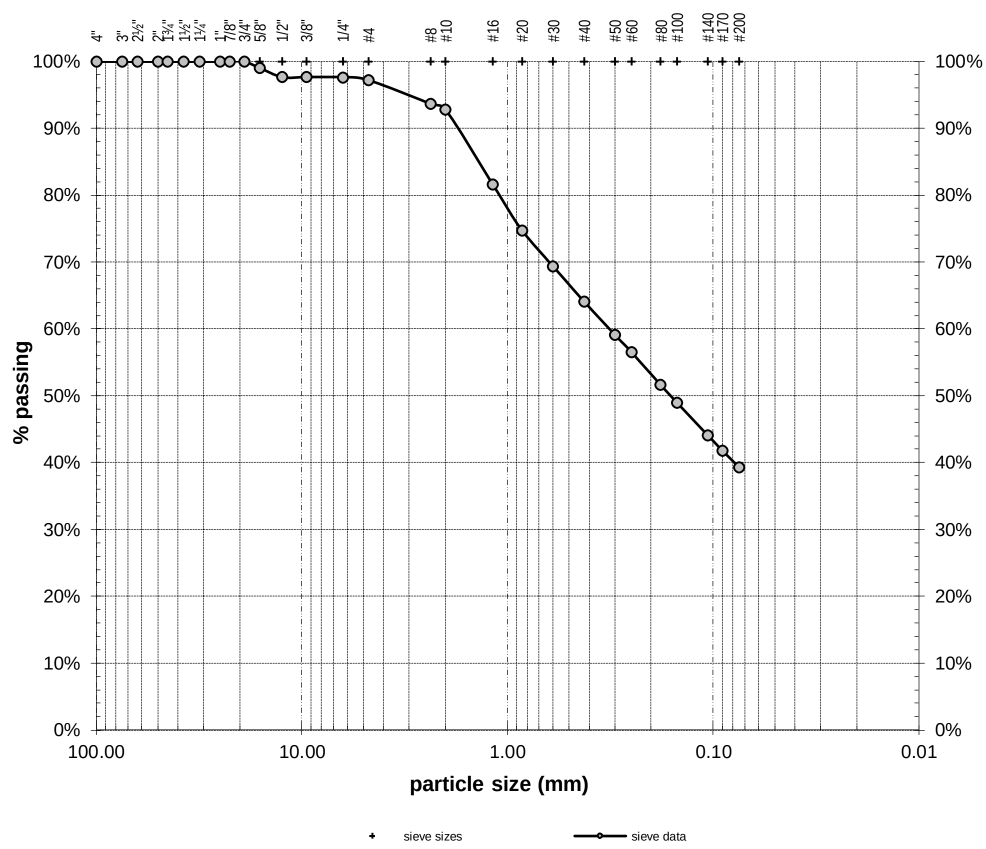
initial dry mass (g) =	112.40		
as-received moisture content =	56.0%	coefficient of curvature, C_C =	n/a
liquid limit =	57	coefficient of uniformity, C_U =	n/a
plastic limit =	33	effective size, $D_{(10)}$ =	n/a
plasticity index =	24	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	0.319 mm

SIEVE DATA

% gravel = 2.7%
% sand = 58.0%
% silt and clay = 39.3%

	SIEVE SIZE US	SIEVE SIZE mm	PERCENT PASSING SIEVE act.	PERCENT PASSING SIEVE interp.	SPECS max	SPECS min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0		100%		
	3.00"	75.0		100%		
	2.50"	63.0		100%		
	2.00"	50.0		100%		
	1.75"	45.0		100%		
	1.50"	37.5		100%		
	1.25"	31.5		100%		
	1.00"	25.0		100%		
	7/8"	22.4		100%		
	3/4"	19.0	100%			
	5/8"	16.0		99%		
	1/2"	12.5	98%			
	3/8"	9.50		98%		
	1/4"	6.30	98%			
	#4	4.75	97%			
SAND	#8	2.36		94%		
	#10	2.00	93%			
	#16	1.18		82%		
	#20	0.850	75%			
	#30	0.600		69%		
	#40	0.425	64%			
	#50	0.300		59%		
	#60	0.250	57%			
	#80	0.180		52%		
	#100	0.150	49%			
	#140	0.106		44%		
	#170	0.090		42%		
	#200	0.075	39%			

GRAIN SIZE DISTRIBUTION



DATE TESTED	TESTED BY
11/14/19	BTT

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Camas High School Field House 26600 SE 15th Street Camas, Washington	Robertson Engineering, PC 1101 Broadway Street, Suite 201 Vancouver, Washington 98660	19276	S19-1109
		REPORT DATE: 11/20/19	FIELD ID: SB1.9
		DATE SAMPLED: 11/11/19	SAMPLED BY: MCK

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
Silty SAND	Soil Boring SB-01 depth = 35 feet	SM, Silty Sand

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Liquid Limit Machine, Hand Rolled	ASTM D4318

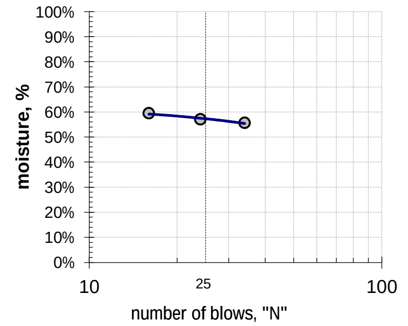
ATTERBERG LIMITS

liquid limit = 57
plastic limit = 33
plasticity index = 24

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	32.46	32.26	32.16	
dry soil + pan weight, g =	28.30	28.12	27.94	
pan weight, g =	20.82	20.86	20.84	
N (blows) =	34	24	16	
moisture, % =	55.6 %	57.1 %	59.4 %	

LIQUID LIMIT



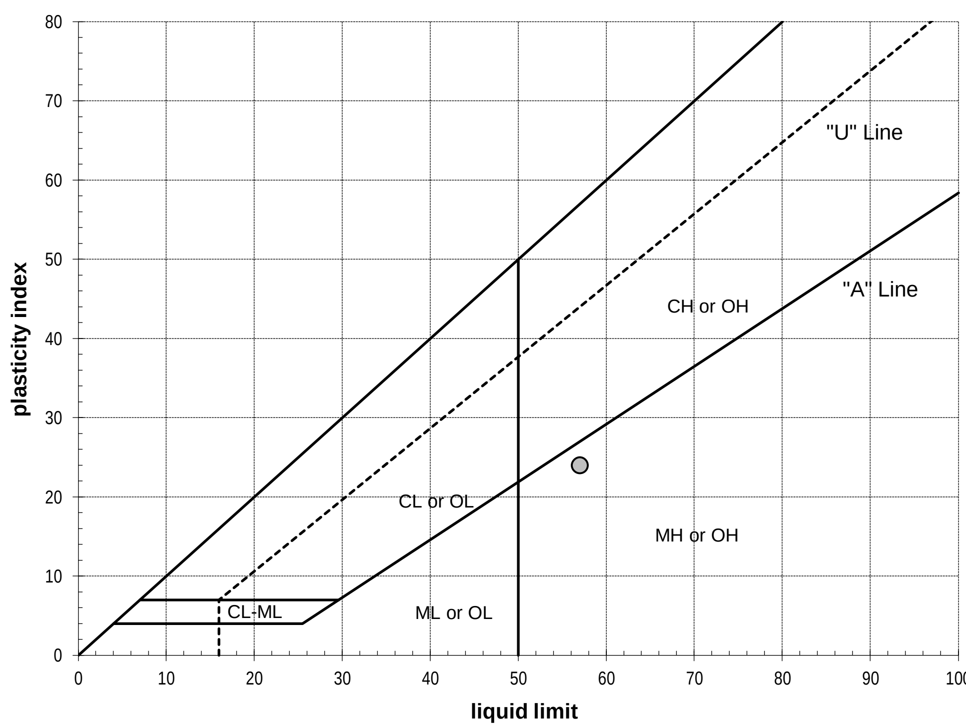
SHRINKAGE

shrinkage limit = n/a
shrinkage ratio = n/a

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.17	27.46		
dry soil + pan weight, g =	25.60	25.76		
pan weight, g =	20.87	20.68		
moisture, % =	33.2 %	33.5 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 2.7%
% sand = 58.0%
% silt and clay = 39.3%
% silt = n/a
% clay = n/a
moisture content = 56.0%

DATE TESTED	TESTED BY
11/19/19	KMS

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MOISTURE CONTENT

PROJECT	CLIENT	PROJECT NO.	REPORT DATE
Camas High School Field House 26600 SE 15th Street Camas, Washington	Robertson Engineering, PC 1101 Broadway Street, Suite 201 Vancouver, Washington 98660	19276	11/20/19
		DATE SAMPLED: 11/11/19	
		SAMPLED BY: MCK	

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Despatch LEB2	ASTM D2216, Method A

LAB ID	CONTAINER MASS	MOIST MASS + PAN	DRY MASS + PAN	MATERIAL DESCRIPTION	FIELD ID	SAMPLE DEPTH	MOISTURE CONTENT
S19-1105	86.83	350.94	283.13	sandy clay	SB1.1	2.5 feet	35%
S19-1106	87.70	308.23	260.08	sandy clay with gravel	SB1.3	7.5 feet	28%
S19-1107	87.20	370.48	324.81	clayey gravel with sand	SB1.4	15 feet	19%
S19-1108	87.37	313.29	264.70	sandy clay with gravel	SB1.6	25 feet	27%
S19-1109	87.61	276.89	208.95	Silty SAND weathered conglomerate	SB1.9	35 feet	56%
S19-1110	85.26	274.90	210.70	sandy silt/clay weathered conglomerate	SB1.11	45 feet	51%

NOTES:	DATE TESTED	TESTED BY
	11/13/19	KMS

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APPENDIX B
SUBSURFACE EXPLORATION LOGS

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Camas High School Field House	Robertson Engineering, PC		19276	TP-1
PROJECT LOCATION: Camas, Washington	CONTRACTOR: L&S Contractors	EQUIPMENT: Excavator	ENGINEER: MCK	DATE: 11/05/19
TEST PIT LOCATION: See Figure 2	APPROX. SURFACE ELEVATION: 378 ft amsl	GROUNDWATER DEPTH: Not Encountered	START TIME: 0923	FINISH TIME: 1145

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						FILL. Approximately 8 to 10 inches of grass and topsoil underlain by apprent reworked tan, mottled, moist, medium dense clayey sand with gravel [Soil Type 1].					
		Hesson clay loam	A-7	CL		Brown, moist, medium stiff sandy lean CLAY with gravel [Soil Type 2].					IT-1.1 D = 3.0-ft k = < 0.1 in/hr
5			A-7	GP-GM SM		Tan to orange-brown, mottled, weathered, moist, medium dense sedimentary CONGLOMERATE of poorly-graded gravel in a sand, silt, and clay matrix [Soil Type 4]. Soil may represent unconsolidated to cemented, thick-bedded, pebble to boulder sedimentary CONGLOMERATE of Evarts, 2008.					
10											
15						Bottom of test pit at 14 feet bgs. Groundwater not observed to 14 feet bgs on 11/05/19.					
20											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Camas High School Field House	Robertson Engineering, PC		19276	TP-2
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
Camas, Washington	L&S Contractors	Excavator	MCK	11/05/19
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	381 ft amsl	Not Encountered	0958	1029

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						FILL. Approximately 6 to 8 inches of grass and topsoil underlain by apparent reworked tan, mottled, moist, medium dense clayey sand with gravel [Soil Type 1].					
	TP2.1	Hesson clay loam	A-7-6(47)	CH		Gray, mottled, moist, stiff fat CLAY with sand [Soil Type 3].	40.1	84.7	76	50	
			A-7	CL		Brown, moist, medium stiff sandy lean CLAY with gravel [Soil Type 2].					
5				GP-GM SM		Tan to orange-brown, mottled, weathered, moist, medium dense sedimentary CONGLOMERATE of poorly-graded gravel in a sand, silt, and clay matrix [Soil Type 4]. Soil may represent unconsolidated to cemented, thick-bedded, pebble to boulder sedimentary CONGLOMERATE of Evarts, 2008.					
	TP2.3		A-2-7(0)				18.7	8.1	47	18	
10											
						Bottom of test pit at 13 feet bgs. Groundwater not observed to 13 feet bgs on 11/05/19.					
15											
20											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Camas High School Field House	Robertson Engineering, PC		19276	TP-3
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	ENGINEER	DATE
Camas, Washington	L&S Contractors	Excavator	MCK	11/05/19
TEST PIT LOCATION	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	START TIME	FINISH TIME
See Figure 2	378 ft amsl	Not Encountered	1031	1102

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 10 to 12 inches of grass and topsoil					
		Hesson clay loam	A-7	CH		Tan to gray, moist, stiff fat CLAY with sand [Soil Type 3].					
5			A-7	GP-GM SM		Tan to orange-brown, mottled, weathered, moist, medium dense sedimentary CONGLOMERATE of poorly-graded gravel in a sand, silt, and clay matrix [Soil Type 4]. Soil may represent unconsolidated to cemented, thick-bedded, pebble to boulder sedimentary CONGLOMERATE of Evarts, 2008.					
10											
15						Bottom of test pit at 14 feet bgs. Groundwater not observed to 14 feet bgs on 11/05/19.					
20											

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SOIL BORING LOG

PROJECT NAME	CLIENT		PROJECT NO.	BORING NO.
Camas High School Field House	Robertson Engineering, PC		19276	SB-1
PROJECT LOCATION	**DRILLING CONTRACTOR**	**DRILL RIG**	**ENGINEER**	**PAGE NO.**
Camas, Washington	Western States	CME Track-Rig	MCK	1 of 2
BORING LOCATION	**DRILLING METHOD**	**SAMPLING METHOD**	**START DATE**	**START TIME**
See Figure 2	Mud-rotary	SPT/SHELBY	11/11/19	0840
REMARKS	**APPROX. SURFACE ELEVATION**	**GROUNDWATER DEPTH**	**FINISH DATE**	**FINISH TIME**
None	379 ft amsl	Not Observed	11/11/19	1200

Depth (ft)	Elevation (ft amsl)	Field ID + Sample Type	SPT N-value (uncorrected) 0 20 40	USCS Soil Type	AASHTO Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Wet Density (PCF)	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index
0							Approximately 6 to 8 inches of grass and topsoil.					
				CL	A-7		Brown, mottled, moist, stiff sandy lean CLAY with gravel [Soil Type 2].					
2												
	376	SPT SB1.1	12						35.0			
4												
		SPT SB1.2	13	GP-GM SM	A-7-5(5)		Tan to orange-brown, mottled, moderately- to severly-weathered, moist, loose to dense sedimentary CONGLOMERATE of poorly-graded gravel in a sand, silt, and clay matrix [Soil Type 4].					
6												
	372											
8		SPT SB1.3	14						28.0			
							Soil may represent unconsolidated to cemented, thick-bedded, pebble to boulder sedimentary CONGLOMERATE of Evarts, 2008.					
10		SPT NR	17									
	368											
12												
14												
	364	SPT SB1.4	15						19.0			
16												
18												
	360											
20		SPT SB1.5	10									
22												
	356											
24												
26		SPT SB1.6	8						27.0			
	352	SHELBY SB1.7										
28		SPT SB1.8	11									
30												

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SOIL BORING LOG

PROJECT NAME	CLIENT		PROJECT NO.	BORING NO.
Camas High School Field House	Robertson Engineering, PC		19276	SB-1
PROJECT LOCATION	DRILLING CONTRACTOR	DRILL RIG	ENGINEER	PAGE NO.
Camas, Washington	Western States	CME Track-Rig	MCK	2 of 2
BORING LOCATION	DRILLING METHOD	SAMPLING METHOD	START DATE	START TIME
See Figure 2	Mud-rotary	SPT/SHELBY	11/11/19	0840
REMARKS	APPROX. SURFACE ELEVATION	GROUNDWATER DEPTH	FINISH DATE	FINISH TIME
None	379 ft amsl	Not Observed	11/11/19	1200

Depth (ft)	Elevation (ft amsl)	Field ID + Sample Type	SPT N-value (uncorrected)	USCS Soil Type	AASHTO Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Wet Density (PCF)	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index
30–51.5	349–327.5						Tan to orange-brown, mottled, moderately- to severly-weathered, moist, loose to dense sedimentary CONGLOMERATE of poorly-graded gravel in a sand, silt, and clay matrix [Soil Type 4].					
35–36.5		SPT SB1.9	15						56.0	39.3	57	24
40–41.5		SPT SB1.10	22									
45–46.5		SPT SB1.11	29						51.0			
50–51.5		SPT SB1.12	38									
51.5							Bottom of soil boring at 51.5 feet bgs. Groundwater not not measured due to mud-rotary drilling technique.					

APPENDIX C
SOIL CLASSIFICATION INFORMATION

SOIL DESCRIPTION AND CLASSIFICATION GUIDELINES

Particle-Size Classification

COMPONENT	ASTM/USCS		AASHTO	
	size range	sieve size range	size range	sieve size range
Cobbles	> 75 mm	greater than 3 inches	> 75 mm	greater than 3 inches
Gravel	75 mm – 4.75 mm	3 inches to No. 4 sieve	75 mm – 2.00 mm	3 inches to No. 10 sieve
Coarse	75 mm – 19.0 mm	3 inches to 3/4-inch sieve	-	-
Fine	19.0 mm – 4.75 mm	3/4-inch to No. 4 sieve	-	-
Sand	4.75 mm – 0.075 mm	No. 4 to No. 200 sieve	2.00 mm – 0.075 mm	No. 10 to No. 200 sieve
Coarse	4.75 mm – 2.00 mm	No. 4 to No. 10 sieve	2.00 mm – 0.425 mm	No. 10 to No. 40 sieve
Medium	2.00 mm – 0.425 mm	No. 10 to No. 40 sieve	-	-
Fine	0.425 mm – 0.075 mm	No. 40 to No. 200 sieve	0.425 mm – 0.075 mm	No. 40 to No. 200 sieve
Fines (Silt and Clay)	< 0.075 mm	Passing No. 200 sieve	< 0.075 mm	Passing No. 200 sieve

Consistency for Cohesive Soil

CONSISTENCY	SPT N-VALUE (BLOWS PER FOOT)	POCKET PENETROMETER (UNCONFINED COMPRESSIVE STRENGTH, tsf)
Very Soft	2	less than 0.25
Soft	2 to 4	0.25 to 0.50
Medium Stiff	4 to 8	0.50 to 1.0
Stiff	8 to 15	1.0 to 2.0
Very Stiff	15 to 30	2.0 to 4.0
Hard	30 to 60	greater than 4.0
Very Hard	greater than 60	-

Relative Density for Granular Soil

RELATIVE DENSITY	SPT N-VALUE (BLOWS PER FOOT)
Very Loose	0 to 4
Loose	4 to 10
Medium Dense	10 to 30
Dense	30 to 50
Very Dense	more than 50

Moisture Designations

TERM	FIELD IDENTIFICATION
Dry	No moisture. Dusty or dry.
Damp	Some moisture. Cohesive soils are usually below plastic limit and are moldable.
Moist	Grains appear darkened, but no visible water is present. Cohesive soils will clump. Sand will bulk. Soils are often at or near plastic limit.
Wet	Visible water on larger grains. Sand and silt exhibit dilatancy. Cohesive soil can be readily remolded. Soil leaves wetness on the hand when squeezed. Soil is much wetter than optimum moisture content and is above plastic limit.

AASHTO SOIL CLASSIFICATION SYSTEM

TABLE 1. Classification of Soils and Soil-Aggregate Mixtures

General Classification	Granular Materials (35 Percent or Less Passing .075 mm)			Silt-Clay Materials (More than 35 Percent Passing 0.075)			
Group Classification	A-1	A-3	A-2	A-4	A-5	A-6	A-7
Sieve analysis, percent passing:							
2.00 mm (No. 10)	-	-	-				
0.425 mm (No. 40)	50 max	51 min	-	-	-	-	-
0.075 mm (No. 200)	25 max	10 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40)							
Liquid limit				40 max	41 min	40 max	41 min
Plasticity index	6 max	N.P.		10 max	10 max	11 min	11 min
General rating as subgrade	Excellent to good			Fair to poor			

Note: The placing of A-3 before A-2 is necessary in the "left to right elimination process" and does not indicate superiority of A-3 over A-2.

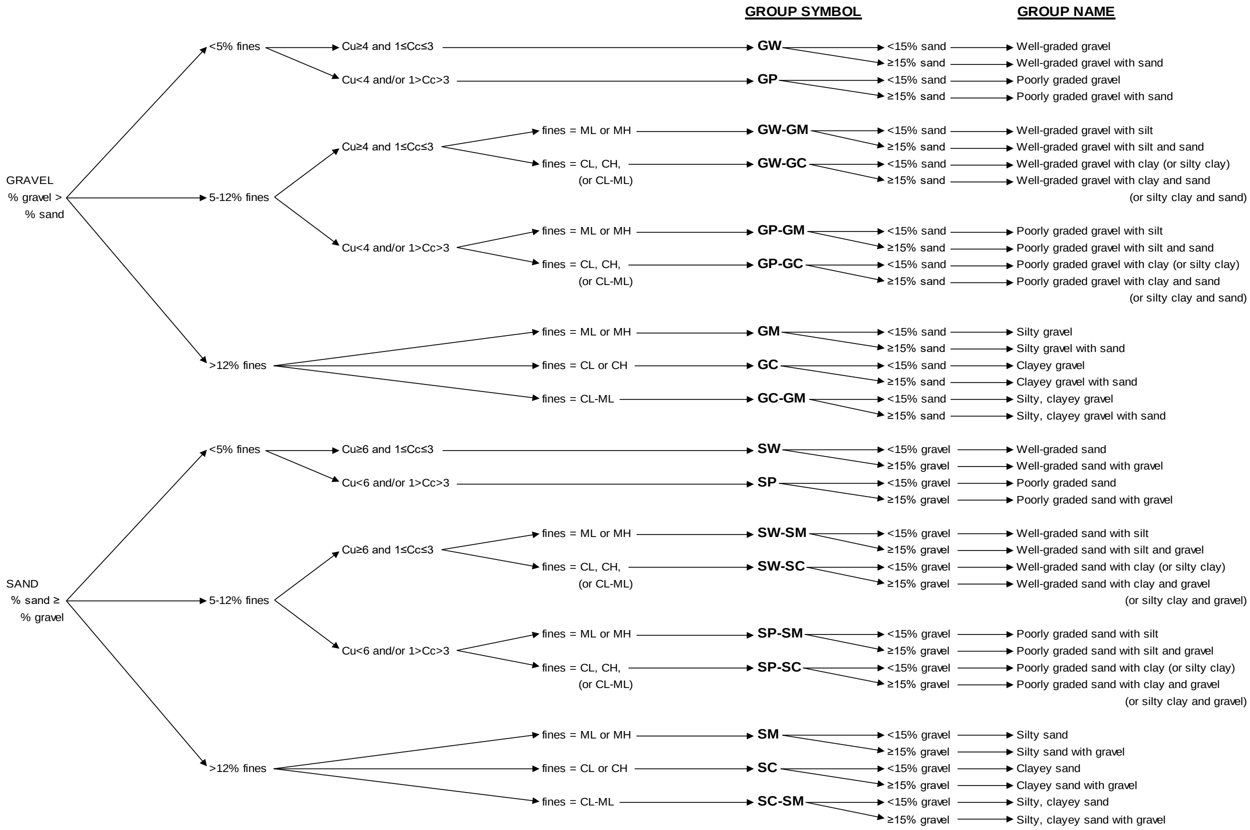
TABLE 2. Classification of Soils and Soil-Aggregate Mixtures

General Classification	Granular Materials (35 Percent or Less Passing 0.075 mm)							Silt-Clay Materials (More than 35 Percent Passing 0.075 mm)			
	A-1			A-2							A-7
Group Classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5, A-7-6
Sieve analysis, percent passing:											
2.00 mm (No. 10)	50 max	-	-	-	-	-	-	-	-	-	-
0.425 mm (No. 40)	30 max	50 max	51 min	-	-	-	-	-	-	-	-
0.075 mm (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40)											
Liquid limit				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11min
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General ratings as subgrade	Excellent to Good							Fair to poor			

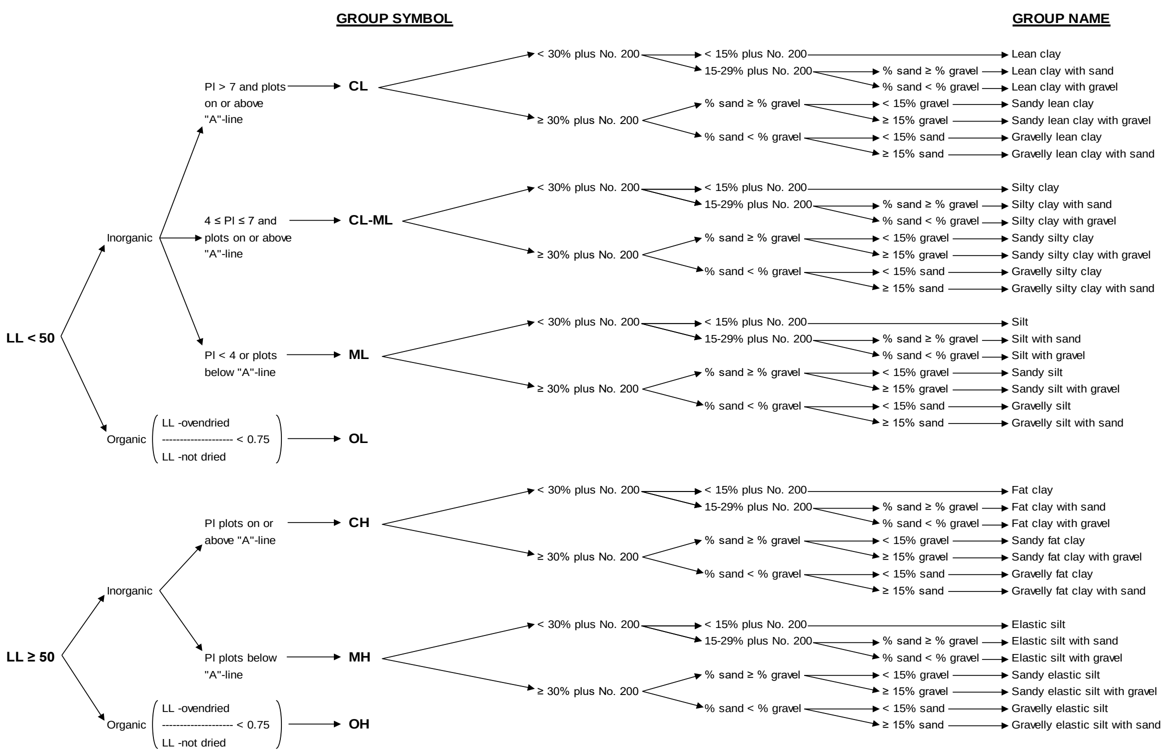
Note: Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30 (see Figure 2).

AASHTO = American Association of State Highway and Transportation Officials

USCS SOIL CLASSIFICATION SYSTEM



Flow Chart for Classifying Coarse-Grained Soils (More Than 50% Retained on No. 200 Sieve)



Flow Chart for Classifying Fine-Grained Soil (50% or More Passes No. 200 Sieve)

APPENDIX D
PHOTO LOG

Geotechnical ■ Environmental ■ Special Inspections
Columbia West Engineering, Inc

Camas High School Field House

November 2019

Camas, Washington

Central Site Area, Facing Southwest

Geotechnical ■ Environmental ■ Special Inspections
Columbia West Engineering, Inc

Camas High School Field House

November 2019

Camas, Washington

Eastern Site Area Facing South

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West Engineering, Inc

Camas High School Field House

November 2019

Camas, Washington

Test Pit Profile, TP-1

Geotechnical ■ Environmental ■ Special Inspections
Columbia West Engineering, Inc

Camas High School Field House

November 2019

Camas, Washington

Test Pit Profile, TP-2

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West Engineering, Inc

Camas High School Field House

November 2019

Camas, Washington

Test Pit Profile, TP-3

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West Engineering, Inc

Camas High School Field House

November 2019

Camas, Washington

Soil Boring, SB-1

APPENDIX E
REPORT LIMITATIONS AND IMPORTANT INFORMATION

Geotechnical ■ Environmental ■ Special Inspections
Columbia West
Engineering, Inc

Date: December 20, 2019
Project: Camas High School Field House
Camas, Washington

Geotechnical and Environmental Report Limitations and Important Information

Report Purpose, Use, and Standard of Care

This report has been prepared in accordance with standard fundamental principles and practices of geotechnical engineering and/or environmental consulting, and in a manner consistent with the level of care and skill typical of currently practicing local engineers and consultants. This report has been prepared to meet the specific needs of specific individuals for the indicated site. It may not be adequate for use by other consultants, contractors, or engineers, or if change in project ownership has occurred. It should not be used for any other reason than its stated purpose without prior consultation with Columbia West Engineering, Inc. (Columbia West). It is a unique report and not applicable for any other site or project. If site conditions are altered, or if modifications to the project description or proposed plans are made after the date of this report, it may not be valid. Columbia West cannot accept responsibility for use of this report by other individuals for unauthorized purposes, or if problems occur resulting from changes in site conditions for which Columbia West was not aware or informed.

Report Conclusions and Preliminary Nature

This geotechnical or environmental report should be considered preliminary and summary in nature. The recommendations contained herein have been established by engineering interpretations of subsurface soils based upon conditions observed during site exploration. The exploration and associated laboratory analysis of collected representative samples identifies soil conditions at specific discreet locations. It is assumed that these conditions are indicative of actual conditions throughout the subject property. However, soil conditions may differ between tested locations at different seasonal times of the year, either by natural causes or human activity. Distinction between soil types may be more abrupt or gradual than indicated on the soil logs. This report is not intended to stand alone without understanding of concomitant instructions, correspondence, communication, or potential supplemental reports that may have been provided to the client.

Because this report is based upon observations obtained at the time of exploration, its adequacy may be compromised with time. This is particularly relevant in the case of natural disasters, earthquakes, floods, or other significant events. Report conclusions or interpretations may also be subject to revision if significant development or other manmade impacts occur within or in proximity to the subject property. Groundwater conditions, if presented in this report, reflect observed conditions at the time of investigation. These conditions may change annually, seasonally or as a result of adjacent development.

Additional Investigation and Construction QA/QC

Columbia West should be consulted prior to construction to assess whether additional investigation above and beyond that presented in this report is necessary. Even slight variations in soil or site conditions may produce impacts to the performance of structural facilities if not adequately addressed. This underscores the importance of diligent QA/QC construction observation and testing to verify soil conditions do not differ materially or significantly from the interpreted conditions utilized for preparation of this report.

Therefore, this report contains several recommendations for field observation and testing by Columbia West personnel during construction activities. Actual subsurface conditions are more readily observed and discerned during the earthwork phase of construction when soils are exposed. Columbia West cannot accept responsibility for deviations from recommendations described in this report or future performance of structural facilities if another consultant is retained during the construction phase or Columbia West is not engaged to provide construction observation to the full extent recommended.

Collected Samples

Uncontaminated samples of soil or rock collected in connection with this report will be retained for thirty days. Retention of such samples beyond thirty days will occur only at client's request and in return for payment of storage charges incurred. All contaminated or environmentally impacted materials or samples are the sole property of the client. Client maintains responsibility for proper disposal.

Report Contents

This geotechnical or environmental report should not be copied or duplicated unless in full, and even then only under prior written consent by Columbia West, as indicated in further detail in the following text section entitled *Report Ownership*. The recommendations, interpretations, and suggestions presented in this report are only understandable in context of reference to the whole report. Under no circumstances should the soil boring or test pit excavation logs, monitor well logs, or laboratory analytical reports be separated from the remainder of the report. The logs or reports should not be redrawn or summarized by other entities for inclusion in architectural or civil drawings, or other relevant applications.

Report Limitations for Contractors

Geotechnical or environmental reports, unless otherwise specifically noted, are not prepared for the purpose of developing cost estimates or bids by contractors. The extent of exploration or investigation conducted as part of this report is usually less than that necessary for contractor's needs. Contractors should be advised of these report limitations, particularly as they relate to development of cost estimates. Contractors may gain valuable information from this report, but should rely upon their own interpretations as to how subsurface conditions may affect cost, feasibility, accessibility and other components of the project work. If believed necessary or relevant, contractors should conduct additional exploratory investigation to obtain satisfactory data for the purposes of developing adequate cost estimates. Clients or developers cannot insulate themselves from attendant liability by disclaiming accuracy for subsurface ground conditions without advising contractors appropriately and providing the best information possible to limit potential for cost overruns, construction problems, or misunderstandings.

Report Ownership

Columbia West retains the ownership and copyright property rights to this entire report and its contents, which may include, but may not be limited to, figures, text, logs, electronic media, drawings, laboratory reports, and appendices. This report was prepared solely for the client, and other relevant approved users or parties, and its distribution must be contingent upon prior express written consent by Columbia West. Furthermore, client or approved users may not use, lend, sell, copy, or distribute this document without express written consent by Columbia West. Client does not own nor have rights to electronic media files that constitute this report, and under no circumstances should said electronic files be distributed or copied. Electronic media is susceptible to unauthorized manipulation or modification, and may not be reliable.

Consultant Responsibility

Geotechnical and environmental engineering and consulting is much less exact than other scientific or engineering disciplines, and relies heavily upon experience, judgment, interpretation, and opinion often based upon media (soils) that are variable, anisotropic, and non-homogenous. This often results in unrealistic expectations, unwarranted claims, and uninformed disputes against a geotechnical or environmental consultant. To reduce potential for these problems and assist relevant parties in better understanding of risk, liability, and responsibility, geotechnical and environmental reports often provide definitive statements or clauses defining and outlining consultant responsibility. The client is encouraged to read these statements carefully and request additional information from Columbia West if necessary.

SECTION 5 – EXCERPT FROM PRIOR STORM REPORTS