GeoPacific
Engineering, Inc.
Real-World Geotechnical Solutions
Investigation • Design • Construction Support

Geotechnical Engineering Report

Old River Partition

18822 Old River Drive
West Linn, Oregon 97068

GeoPacific Engineering, Inc. Job No. 16-4408
December 30, 2016

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Real-World Geotechnical Solutions
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December 30, 2016
Project No. 16-4408

Mr. Jeff Parker
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West Linn, Oregon 97068
Phone: (503) 742-1942, Ext 3497

SUBJECT: GEOTECHNICAL ENGINEERING REPORT
OLD RIVER PARTITION
18822 OLD RIVER DRIVE
WEST LINN, OREGON 97068

PROJECT INFORMATION

This report presents the results of a geotechnical engineering study conducted by GeoPacific Engineering, Inc. (GeoPacific) for the above-referenced project. The purpose of our investigation was to evaluate subsurface conditions at the site, and to provide geotechnical recommendations for site development. This geotechnical study was performed in accordance with GeoPacific Agreement for Geotechnical Services P-5814, dated November 9, 2016, and your subsequent authorization of our proposal and *General Conditions for Geotechnical Services*.

Site Location:	18822 Old River Drive West Linn, Oregon 97068 Clackamas County Parcel No. 00297253 (see Figures 1 through 3)
Developer:	Mr. Jeff Parker 1800 Blankenship Road, Suite 200 West Linn, Oregon 97068
Jurisdictional Agency:	City of West Linn, Oregon
Geotechnical Engineer:	GeoPacific Engineering, Inc 14835 SW 72nd Avenue Portland, Oregon 97224 Tel (503) 598-8445 Fax (503) 941-9281

SITE AND PROJECT DESCRIPTION

The site is located at 18822 Old River Drive in West Linn, Oregon, is comprised of Clackamas County tax parcel 00297253 totaling approximately 0.76-acres in size, and is irregular in shape. The property is located adjacent to the west of 3588 Robin View Drive, and adjacent to the north of 18866 Old River Drive. The site is bordered to the west by Old River Drive, to the north by Robin View Drive, and to the south by a steep slope area which extends approximately 70 vertical feet to a northeast flowing creek. Historically the site contained a residential home in the relatively flat, central portion of the property, however, it was recently demolished. A paved driveway was present along the northern portion of the site with access from Robin View drive which has also been removed. The house contained a basement which was demolished and loosely backfilled with onsite soil and remnant building debris. The portions of the site adjacent to the remnant house are primarily vegetated with grasses and weeds. The steeply sloping southern portions of the property are heavily vegetated with coniferous trees and dense, understory vegetation.

GeoPacific understands that development at the site will consist of a 3-Lot partition to support construction of two-story, wood framed, residential homes, incorporating typical spread foundations. The client has indicated that the homes will be constructed in the northern portion of the site adjacent to Robin View Drive which where site gradients of less than 10 percent exist. We anticipate maximum applied bearing pressures from the proposed house foundations to be on the order of 1,500 psf. Cuts are anticipated to be on the order of five feet or less. We assume limited fill placement.

Topography in the northern portion of the property where the existing home was located is relatively flat with site gradients ranging from approximately 1 to 5 percent. To the south and southeast the lot slopes gently to moderately through the grassy area with gradients ranging from approximately 5 to 15 percent. The slope gradually becomes steeper to the southeast in the heavily vegetated portion of the site where it drops steeply to the creek below where site gradients are as steep as 150 percent extending to the river bank (see Figure 3). As indicated on Figure 3, an area containing slopes in exceedance of 15 percent is present extending northeast to southwest along the southeastern portion of the site. A riparian zone is present below which is measured at 100 feet to the northwest of the stream bank. An existing conservation easement is present near the creek. Based upon review of a topographic survey provided by the client, site elevations range from approximately 60 to 118 feet above mean sea level (amsl). The approximate site latitude and longitude is 45.395201, -122.639736, and the legal description is the NW ¼ of Section 13, T2S, R1E, Willamette Meridian. The regulatory jurisdictional agency is the City of West Linn, Oregon.

The City of West Linn building code requires a minimum building setback distance of 50 feet from areas containing slopes greater than 15 percent. The building code allows the 50-foot-distance to be reduced to 25 feet if a geotechnical study by a licensed engineer or similar accredited professional demonstrates that the slope is stable and not prone to erosion. Based upon our understanding of site planning and communication with the client, the house proposed at the eastern lot (Lot 3) will encroach into the 50-foot-setback distance by up to 25 feet. Figure 3 indicates the noted setback locations in relation to the slope.

REGIONAL GEOLOGIC SETTING

Regionally, the subject site lies within the Willamette Valley/Puget Sound lowland, a broad structural depression situated between the Coast Range on the west and the Cascade Range on the east. A series of discontinuous faults subdivide the Willamette Valley into a mosaic of fault-bounded, structural blocks (Yeats et al., 1996). Uplifted structural blocks form bedrock highlands, while down-warped structural blocks form sedimentary basins.

According to the *Geologic Map of the Lake Oswego Quadrangle, Clackamas, Multnomah, and Washington Counties, Oregon* (State of Oregon Department of Geology and Mineral Industries, Hull, Donald A. 1989), near-surface soils are expected to consist of Pleistocene-aged (approximately 2.6 million to 12,000 years ago), coarse sand to silt deposited by repeated catastrophic flood outbursts of Glacial Lake Missoula (Qff). Underlying the flood deposits, geologic mapping indicates that soils likely consist of the middle Miocene-aged (approximately 23 to 5.3 million years ago) Basalts of Sand Hollow (Tfsh). The basalts are generally composed of dense, finely crystalline rock that is commonly fractured along blocky and columnar vertical joints. *The Relative Earthquake Hazard Map of the Lake Oswego Quadrangle, Clackamas, Multnomah, and Washington Counties, Oregon* (State of Oregon Department of Geology and Mineral Industries, Hull, Donald, A., 1995), indicates that the subject site is located within Zone A. Zone A is indicates areas of the greatest hazard. *The Web Soil Survey (United States Department of Agriculture, Natural Resource Conservation Service (USDA NRCS 2016 Website)*, indicates that near-surface soils primarily consist of the Woodburn silt loam soils series. Woodburn series soils generally consist of very deep, moderately well drained silts and sands that formed in silty stratified, glacio-lacustrine deposits.

REGIONAL SEISMIC SETTING

At least four major fault zones capable of generating damaging earthquakes are thought to exist in the vicinity of the subject site. These include the Portland Hills Fault Zone, the Lacamas Creek/Sandy River Fault Zone, the Gales Creek-Newberg-Mt. Angel Structural Zone, and the Cascadia Subduction Zone.

Portland Hills Fault Zone

The Portland Hills Fault Zone is a series of NW-trending faults that include the central Portland Hills Fault, the western Oatfield Fault, and the eastern East Bank Fault. These faults occur in a northwest-trending zone that varies in width between 3.5 and 5.0 miles. The combined three faults reportedly vertically displace the Columbia River Basalt by 1,130 feet and appear to control thickness changes in late Pleistocene (approx. 780,000 years) sediment (Madin, 1990). The Portland Hills Fault occurs along the Willamette River at the base of the Portland Hills, and is located approximately 2.4 miles northeast of the site. The Oatfield Fault occurs along the western side of the Portland Hills, and is located approximately 1.2 miles southwest of the site. The East Bank Fault occurs along the eastern margin of the Willamette River, and is located approximately 6.1 miles northeast of the site. The accuracy of the fault mapping is stated to be within 500 meters (Wong, et al., 2000).

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 Columbia River Basalts, and Miocene to Pliocene sedimentary rocks of the Troutdale Formation. No fault scarps on surficial Quaternary deposits have been described along the fault trace, and the fault is mapped as buried by the Pleistocene aged Missoula flood deposits. No historical seismicity is correlated with the mapped portion of the Portland Hills Fault Zone, but in 1991 a M3.5 earthquake occurred on a NW-trending shear plane located 1.3 miles east of the fault (Yelin, 1992). Although there is no definitive evidence of recent activity, the Portland Hills Fault Zone is assumed to be potentially active (Geomatrix Consultants, 1995).

Lacamas Creek / Sandy River Fault Zone

The northwest trending Lacamas Creek Fault intersects the northeast trending Sandy River Fault north of Camas, Washington at Lacamas Lake, approximately 18.5 miles northeast of the subject 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 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.

Gales Creek-Newberg-Mt. Angel Structural Zone

The Gales Creek-Newberg-Mt. Angel Structural Zone is a 50-mile-long zone of discontinuous, NW-trending faults that lies about 17.4 miles southwest of the subject site. These faults are recognized in the subsurface by vertical separation of the Columbia River Basalt and offset seismic reflectors in the overlying basin sediment (Yeats et al., 1996; Werner et al., 1992). A geologic reconnaissance and photogeologic analysis study conducted for the Scoggins Dam site in the Tualatin Basin revealed no evidence of deformed geomorphic surfaces along the structural zone (Unruh et al., 1994). No seismicity has been recorded on the Gales Creek Fault or Newberg Fault (the fault closest to the subject site); however, these faults are considered to be potentially active because they may connect with the seismically active Mount Angel Fault and the rupture plane of the 1993 M5.6 Scotts Mills earthquake (Werner et al. 1992; 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 rocks of the Columbia River Basalts, and Miocene and Pliocene 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 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.

Cascadia Subduction Zone

The Cascadia Subduction Zone is a 680-mile-long zone of active tectonic convergence where oceanic crust of the Juan de Fuca Plate is subducting beneath the North American continent at a rate of 4 cm per year (Goldfinger et al., 1996). A growing body of geologic evidence suggests that prehistoric subduction zone earthquakes have occurred (Atwater, 1992; Carver, 1992; Peterson et al., 1993; Geomatrix Consultants, 1995). This evidence includes: (1) buried tidal marshes recording episodic, sudden subsidence along the coast of northern California, Oregon, and Washington, (2) burial of subsided tidal marshes by tsunami wave deposits, (3) paleoliquefaction features, and (4) geodetic uplift patterns on the Oregon coast. Radiocarbon dates on buried tidal marshes indicate a recurrence interval for major subduction zone earthquakes of 250 to 650 years with the last event occurring 300 years ago (Atwater, 1992; Carver, 1992; Peterson et al., 1993; Geomatrix Consultants, 1995). The inferred seismogenic portion of the plate interface lies approximately along the Oregon Coast at depths of between 20 and 40 kilometers below the surface.

FIELD EXPLORATION AND SUBSURFACE CONDITIONS

Our site-specific explorations for this report were conducted on December 6, 2016. Four exploratory test pits (TP-1 through TP-4) were excavated at the site to a maximum depth of 12 feet bgs using a track-mounted excavator subcontracted by GeoPacific. In March of 2016, GeoPacific conducted a geotechnical site investigation of the property and the slopes of the property adjacent to the east, at 3588 Robin View Drive. During our previous geotechnical site investigation one exploratory soil boring (B-1) was drilled approximately 50 feet to the east of the property boundary to a depth of 41.5 feet below the existing ground surface (bgs) using a track-mounted, solid-stem drill system subcontracted by GeoPacific. At the boring location, SPT (Standard Penetration Test) sampling was performed in general accordance with ASTM D1586 using a 2-inch outside diameter split-spoon sampler and a 140-pound hammer equipped with a rope and cathead mechanism. During the test, a sample is obtained by driving the sampler 18 inches into the soil with the hammer free-falling 30 inches. The number of blows for each 6 inches of penetration is recorded. The Standard Penetration Resistance ("N-value") of the soil is calculated as the number of blows required for the final 12 inches of penetration. If 50 or more blows are recorded within a single 6-inch interval, the test is terminated, and the blow count is recorded as 50 blows for the number of inches driven. This resistance, or N-value, provides a measure of the relative density of granular soils and the relative consistency of cohesive soils. The approximate locations of the test pit explorations are indicated on Figure 2. The boring log B-1 conducted on the property to the east is attached for reference.

It should be noted that the exploration locations were located in the field by pacing or taping distances from apparent property corners and other site features shown on the plans provided. As such, the locations of the explorations should be considered approximate. During the explorations, GeoPacific observed and recorded pertinent soil information such as color, stratigraphy, strength, and soil moisture content. Soils were classified in general accordance with the Unified Soil Classification System (USCS). At the completion of each test, the test pits were backfilled loosely with onsite soil. Exploration logs are presented in the appendix of this report. Soil and groundwater conditions encountered in the explorations are summarized below.

Soil Descriptions

Topsoil: Underlying the ground surface at the locations of test pits TP-1, TP-3, and TP-4, soils were observed to consist of dark brown, very moist, moderately organic SILT (OL-ML), containing fine grass roots. The topsoil horizon was observed to be approximately 8-inches-thick at the locations explored. It is likely that the thickness of the organic soil layers will increase in areas where trees, and dense understory vegetation are present.

Undocumented Fill: Underlying the ground surface at the location of test pit TP-2, which was conducted in the location of the remnant building basement, soils were observed to consist of a crushed aggregate/soil mixture, containing bricks, plastic, piping, rain gutters, and roofing material, extending to an approximately depth of 7 feet bgs. The apparent basement backfill was very moist, soft/loose, and easily excavated. The basement apparent basement backfill material is considered to be non-structural, and should be removed and replaced, preferably with reject rock or crushed aggregate, in areas proposed for new building foundations.

Sandy SILT/SILT with Sand: Underlying the topsoil at the location of locations of test pits TP-1, TP-3, and TP-4, and underlying the undocumented fill soils at the location of test pit TP-2, soils were observed to consist of brown, brown, very moist, medium stiff, moderately plastic, Sandy SILT/SILT with Sand (ML). The soil type classified as A-7-6(16), A-4(5), and A-4(6) according to AASHTO Standards. Sieve analysis indicated 60 to 94 percent by weight passing the U.S. No. 200 sieve, and moisture contents ranging from 33 to 35 percent. Atterberg limit testing indicated a liquid limit ranging from 37 to 45, and a plasticity index ranging from 8 to 17. Pocket penetrometer measurements conducted in the upper four feet of our test pits indicated unconfined compressive strengths ranging from approximately 1.0 to 3.5 tons/ft^2. The soil type was observed to extend to the maximum depth of exploration within our test pit explorations, and to a depth of 25 feet bgs in the soil boring conducted on the property adjacent to the east.

Groundwater and Soil Moisture

On December 6, 2016, observed soil moisture conditions were generally very moist. Groundwater was not encountered within our explorations which extended to a maximum depth of approximately 12 feet bgs. The creek at the bottom of the slope on the southern portion of the site was observed to contain water. According to the *Estimated Depth to Groundwater in the Portland, Oregon Area, (United States Geological Survey, Snyder, 2016 website)*, groundwater is expected to be present at an approximate depth of 49 feet below the ground surface. It is anticipated that groundwater conditions will vary depending on the season, local subsurface conditions, changes in site utilization, and other factors. Perched groundwater may be encountered in localized areas. Seeps and springs may exist in areas not explored, and may become evident during site grading.

SLOPE STABILITY STUDY

The City of West Linn building code requires a minimum building setback distance of 50 feet from areas containing slopes greater than 15 percent. The building code allows the 50-foot-distance to be reduced to 25 feet if a geotechnical study by a licensed engineer or similar accredited professional demonstrates that the slope is stable and not prone to erosion. Based upon our understanding of site planning and communication with the client, the southern portion of the house proposed at the eastern lot (Lot 3) will encroach into a 50-foot-setback distance from southern sloping portions of the property by up to 25 feet. Figure 3 indicates the noted setback locations in relation to the slope.

In order to determine the static and seismic factors of safety against slope instability associated with construction of house(s) with reduced setback distances located 25 feet from the portions of the site containing slopes greater than 15 percent, GeoPacific conducted a geologic hazard assessment and quantitative slope stability analysis of the south-facing slope extending to the creek below. For the purpose of evaluating global slope stability of the hillside with the proposed construction, we reviewed published geologic and hazard mapping, reviewed regional site topography and LIDAR imagery, reviewed legal property records, performed field reconnaissance, evaluated subsurface soil conditions in test pit explorations, created a geologic cross-section, and conducted quantitative slope stability modeling and analysis. The location of our geologic cross-section is shown on the attached Figures 2 and 3. A geologic cross-section of the subject property with the proposed development is presented in Figure 4. LIDAR images utilized in our site evaluation are presented in Figures 5 and 6. Slope stability calculations and cross-sections are presented in the appendix of this report. The results of our study are presented below.

Hazard Mapping Literature Review

The Relative Earthquake Hazard Map of the Lake Oswego Quadrangle, Clackamas, Multnomah, and Washington Counties, Oregon (State of Oregon Department of Geology and Mineral Industries, Hull, Donald, A., 1995), indicates that the subject site is located within Zone A. Zone A is indicates areas of the greatest hazard.

The Oregon Department of Geology and Mineral Industries (DOGAMI), Oregon HazVu: Statewide Geohazards Viewer indicates that the subject site is located in an area considered at risk for *severe* ground shaking, and *high* risk for soil liquefaction during an earthquake. LIDAR imagery reviewed of the site (Figures 5 and 6) indicates that the site is primarily located on a relatively broad, flat parcel, which is incised on the southeastern corner by a creek drainage ravine. The ravine is steep on the northwestern embankment, measuring approximately 70-feet-high, extending to a northwest flowing creek below. As can be seen in the LIDAR image in Figure 6, the creek bends to the northwest south of the subject site, where it has eroded into the slope on the outside (northwest) of the creek bend. The creek straightens adjacent to the subject site, and less erosion appears to have occurred. Published regional geologic mapping and the DOGAMI online landslide database indicate the presence of a landslide at the creek bend located south of the subject site. (Madin, 1990; Burns et al., 2011; DOGAMI SLIDO database, 2016). As shown on Figure 5, DOGAMI has identified landslide Lake Oswego #299 along the northwest side of the creek located approximately 100 feet to the south of the property. According to the DOGAMI

SLIDO website, Lake Oswego #299 consists of an earth flow slide with relative movement to the northeast.

Above the drainage ravine in the relatively flat portions of the property, no evidence of recent or prior earth movement, or disturbed soil conditions such as hummocky terrain or scarps is visible on the imagery. No clear evidence of recent or prior earth movement, soil erosion, or disturbed soil conditions such as hummocky terrain or scarps was observed of the north side of the drainage ravine along the frontage of the property.

Field Reconnaissance

We conducted field reconnaissance of the site to observe geomorphic features and assess the relative slope stability. In particular the steep slopes extending to the creek on the southern portion of the site were observed for recent landsliding, soil erosion, or general earth movements. During field exploration some large trees with trunks bowed towards the creek were observed to be present on the slope facing indicating that some shallow soil creep may be occurring. We did not observe geomorphic evidence of prior slope instability (such as hummocky topography, benches, or old scarps). No tension cracks, slumping, or areas of recent landsliding were observed. Vegetation appeared to be heavy and undisturbed down to the creek bank. However, the slope and creek bank contains gradients up to 150 percent, which should generally be considered susceptible to erosion by the creek, particularly if vegetation becomes disturbed in the future. Adjacent to the property boundary soil conditions appeared to be similar. At the time of our site visit water was flowing through the creek. It appears that the drainage was formed as the creek eroded through soft sediment, ultimately exposing basaltic bedrock at the creek bottom. The basaltic bedrock exposed in the creek is highly resistant to continued erosion, so it appears that erosion to the stream bank will primarily occur during periods of heavy rainfall and flooding. In general, the slope appeared to be relatively stable at the time of this report.

Subsurface Exploration

Test pit explorations were conducted at the site extending to a maximum depth of 12 feet bgs. The approximate locations of the explorations are indicated on the attached Figures 2 and 3. Subsurface exploration logs are attached in the appendix of this report. Subsurface conditions encountered within our explorations indicated that the upper 12 feet of soil at the test location consisted of medium stiff SILT, Sandy SILT, and SILT with Sand. Based upon the soil boring we drilled on the neighboring property approximately 50 feet to the east, and our observations of soil conditions at the stream bed, it appears that the fine-grained soil deposits extend to an approximate depth of 25 feet bgs at the subject site. Underlying the Silty soils, Silty SAND is present extending to an approximate depth of 37 feet bgs. The SAND is underlain by dense, Clayey GRAVEL, and basaltic bedrock.

Quantitative Slope Stability Modeling

GeoPacific conducted a quantitative slope stability analysis consisting of a geologic cross-section and Slope-W modeling through the proposed development area and the south-facing slope below, to model construction of the proposed houses with a reduced setback distance of 25 feet from the portions of the site contain slopes with gradients of 15 percent of greater. We modelled theoretical placement of a two-story house with a spread foundation, and a structural load from walls and footings up to 1,500 psf. The location of cross-section A to A', is indicated on the attached Figures 2 and 3. The geologic cross-section is presented on Figure 4. Slope-W cross-sections are presented in the appendix of this report.

Quantitative slope stability modeling and analyses were performed to evaluate slope stability on the site under post-construction conditions using the SLOPE/W computer program developed by Geo-Slope International of Calgary, Canada. This numerical analysis program utilizes a two-dimensional limiting equilibrium method to calculate the factor of safety of a potential slip surface, and incorporates search routines to identify the most critical potential failure surfaces for the cases analyzed. Factors of safety were calculated using the Morgenstern-Price method of slices, and Mohr-Coulomb soil parameters.

Existing subsurface conditions through the proposed development area were modeled as a three-layer system with layers consisting of SILT/Sandy SILT, Silty SAND, and Clayey GRAVEL, representative of subsurface soil conditions encountered within our subsurface explorations. Slope topography, subsurface geometry, and other conditions modeled in the analyses are based on a topographic map of the site prepared by All County Surveyors and Planners, Inc., dated October 21, 2016.

Slope Stability Calculations A to A'- Existing Soil Conditions with Proposed Houses

For stability calculations of the south-facing slope with the structural load of the proposed house, the potential failure model was considered primarily as circular or planar sliding along a basal shear surface. Shear strength parameters used in the models were selected based on soil conditions encountered within our subsurface explorations, and our local experience with similar soil and geologic conditions. The internal angle of friction of each soil type was estimated based on empirical correlations of soil stiffness, soil type, and vertical effective stress (adapted from DeMello, 1971, Coduto, 2001, Figure 4.11).

The slope stability analysis was conducted at the site in order to model placement of the proposed house with a reduced setback distance of 25 feet from the portions of the site contain slopes with gradients of 15 percent of greater, and assumed maximum bearing pressures from foundation columns and walls of 1,500 psf, in order to identify the factor of safety against sliding with the proposed development as we understand it. Existing conditions soil parameters assumed in the stability calculations are summarized in Table 1. The results of our analysis are summarized in Table 2. Slope stability analysis cross-sections are presented as attachments to this report. The location of the cross-section is indicated on Figures 2 and 3.

Table 1 - Summary of Estimated Soil Strength Parameters – A to A'

Geologic Unit	Wet Unit Weight (pcf)	Friction Angle	Cohesion (psf)
SILT/SANDY SILT	110	28°	50
SILTY SAND	110	28°	0
CLAYEY GRAVEL	125	40°	0

Table 2 - Summary of Slope Stability Analyses for Post-Development Soil Conditions A to A'

Condition Analyzed	Factor of Safety
Section A-A' South Facing Slope	1.6 Static 1.1 Seismic (1/2 PGA 0.452g = 0.22g)

The results of the quantitative slope stability modeling and analyses performed using the SLOPE/W computer program indicate that the factor of safety for slope stability under post-development soil conditions, with a reduced setback distance of 25 feet from the portions of the site contain slopes with gradients of 15 percent of greater, is greater than 1.5 for static conditions, and greater than 1.1 for seismic conditions. The seismic condition was modeled with ½ of the peak ground acceleration (PGA) in accordance with the requirements of the OSSC 2014. In our opinion, the calculated minimum factors of safety for placement are adequate for the proposed development as we understand it assuming a minimum footing-to-slope setback distance of 25 feet is maintained from portions of the site containing slope gradients of 15 percent or greater.

CONCLUSIONS AND RECOMMENDATIONS

Our site investigation indicated that the proposed construction is geotechnically feasible, provided that the recommendations of this report are incorporated into the design and construction phases of the project. The primary geotechnical concerns associated with development of the site are:

1) Undocumented fill soils placed in the basement demolition of the removed house are considered non-structural, and unsuitable to provide sufficient bearing support for the proposed structures. The soils should be thoroughly removed and replaced with engineered fill.
2) A minimum footing-to-slope setback distance of 25 feet shall be maintained from portions of the site containing slope gradients of 15 percent or greater for all structures.

Site Preparation Recommendations

Areas of proposed construction and areas to receive fill should be cleared of vegetation, stockpiled soils, and any organic and inorganic debris. Inorganic debris and organic materials from clearing should be removed from the site. Organic-rich soils and root zones should then be stripped from construction areas of the site or where engineered fill is to be placed. Undocumented fill associated with demolition of the house at the site, and non-structural backfill of the basement shall be excavated and removed. Depth of stripping of organic soils is estimated to be approximately 8 inches across the majority of the site, however depth of organic soil layers may increase in areas where trees are present. Undocumented fill at the location of test pit TP-2 extended to approximately 7 feet bgs. The final depth of soil removal will be determined on the basis of a site inspection after the stripping/excavation has been performed. Stripped topsoil should be removed from the site. Any remaining topsoil should be stockpiled only in designated areas and stripping operations should be observed and documented by the geotechnical engineer or his representative. If encountered, all undocumented fills and any subsurface structures (dry wells, basements, driveway and landscaping fill, old utility lines, septic leach fields, etc.) should be completely removed and the excavations backfilled with engineered fill.

Engineered Fill

All grading for the proposed construction should be performed as engineered grading in accordance with the applicable building code at the time of construction with the exceptions and additions noted herein. Areas proposed for fill placement should be prepared as described in the site preparation section. Surface soils should then be scarified and recompacted prior to placement of structural fill. Proper test frequency and earthwork documentation usually requires daily observation and testing during stripping, rough grading, and placement of engineered fill. Imported fill material must be approved by the geotechnical engineer prior to being imported to the site. Oversize material greater than 6 inches in size should not be used within 3 feet of foundation footings, and material greater than 12 inches in diameter should not be used in engineered fill.

Engineered fill should be compacted in horizontal lifts not exceeding 8 inches using standard compaction equipment. We recommend that engineered fill be compacted to at least 95 percent of the maximum dry density determined by ASTM D698 (Standard Proctor) or equivalent. Field density testing should conform to ASTM D2922 and D3017, or D1556. All engineered fill should be observed and tested by the project geotechnical engineer or his representative. Typically, one density test is performed for at least every 2 vertical feet of fill placed or every 500 yd^3, whichever requires more testing. Because testing is performed on an on-call basis, we recommend that the earthwork contractor be held contractually responsible for test scheduling and frequency. Site earthwork will be impacted by soil moisture and shallow groundwater conditions.

Excavating Conditions and Utility Trench Backfill

We anticipate that on-site soils can be excavated using conventional heavy equipment. Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor. Actual slope inclinations at the time of construction should be determined based on safety requirements and actual soil and groundwater conditions. All temporary cuts in excess of 4 feet in height should be sloped in accordance with U.S. Occupational Safety and Health Administration (OSHA) regulations (29 CFR Part 1926), or be shored. The existing soils classify as Type B Soil and temporary excavation side slope inclinations as steep as 1H:1V may be assumed for planning purposes. This cut slope inclination is applicable to excavations above the water table only.

Shallow, perched groundwater may be encountered during the wet weather season and should be anticipated in excavations and utility trenches. Vibrations created by traffic and construction equipment may cause some caving and raveling of excavation walls. In such an event, lateral support for the excavation walls should be provided by the contractor to prevent loss of ground support and possible distress to existing or previously constructed structural improvements.

PVC pipe should be installed in accordance with the procedures specified in ASTM D2321 and City of West Linn standards. We recommend that structural trench backfill be compacted to at least 95 percent of the maximum dry density obtained by the Modified Proctor (ASTM D1557) or equivalent. Initial backfill lift thicknesses for a ¾"-0 crushed aggregate base may need to be as great as 4 feet to reduce the risk of flattening underlying flexible pipe. Subsequent lift thickness should not exceed 1 foot. If imported granular fill material is used, then the lifts for large vibrating plate-compaction equipment (e.g. hoe compactor attachments) may be up to 2 feet, provided that proper compaction is being achieved and each lift is tested. Use of large vibrating compaction equipment should be carefully monitored near existing structures and improvements due to the potential for vibration-induced damage.

Adequate density testing should be performed during construction to verify that the recommended relative compaction is achieved. Typically, at least one density test is taken for every 4 vertical feet of backfill on each 200-lineal-foot section of trench.

Erosion Control Considerations

During our field exploration program, we did not observe soil conditions that would be considered highly susceptible to erosion. In our opinion, the primary concern regarding erosion potential will occur during construction in areas that have been stripped of vegetation. Erosion at the site during construction can be minimized by implementing the project erosion control plan, which should include judicious use of straw waddles, fiber rolls, and silt fences. If used, these erosion control devices should remain in place throughout site preparation and construction.

Erosion and sedimentation of exposed soils can also be minimized by quickly re-vegetating exposed areas of soil, and by staging construction such that large areas of the project site are not denuded and exposed at the same time. Areas of exposed soil requiring immediate and/or temporary protection against exposure should be covered with either mulch or erosion control netting/blankets. Areas of exposed soil requiring permanent stabilization should be seeded with an approved grass seed mixture, or hydroseeded with an approved seed-mulch-fertilizer mixture.

Wet Weather Earthwork

Soils underlying the site are likely to be moisture sensitive and may be difficult to handle or traverse with construction equipment during periods of wet weather. Earthwork is typically most economical when performed under dry weather conditions. Earthwork performed during the wet-weather season will probably require expensive measures such as cement treatment or imported granular material to compact areas where fill may be proposed to the recommended engineering specifications. If earthwork is to be performed or fill is to be placed in wet weather or under wet conditions when soil moisture content is difficult to control, the following recommendations should be incorporated into the contract specifications.

- Earthwork should be performed in small areas to minimize exposure to wet weather. Excavation or the removal of unsuitable soils should be followed promptly by the placement and compaction of clean engineered fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance. Under some circumstances, it may be necessary to excavate soils with a backhoe to minimize subgrade disturbance caused by equipment traffic;

- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water;
- Material used as engineered fill should consist of clean, granular soil containing less than 5 percent passing the No. 200 sieve. The fines should be non-plastic. Alternatively, cement treatment of on-site soils may be performed to facilitate wet weather placement;
- The ground surface within the construction area should be sealed by a smooth drum vibratory roller, or equivalent, and under no circumstances should be left uncompacted and exposed to moisture. Soils which become too wet for compaction should be removed and replaced with clean granular materials;
- Excavation and placement of fill should be observed by the geotechnical engineer to verify that all unsuitable materials are removed and suitable compaction and site drainage is achieved; and
- Geotextile silt fences, straw waddles, and fiber rolls should be strategically located to control erosion.

If cement or lime treatment is used to facilitate wet weather construction, GeoPacific should be contacted to provide additional recommendations and field monitoring.

Spread Foundations

The proposed residential structures may be supported on a shallow foundation bearing on stiff, competent undisturbed, native soils and/or engineered fill, appropriately designed and constructed as recommended in this report. Foundation design, construction, and setback requirements should conform to the applicable building code at the time of construction. For maximization of bearing strength and protection against frost heave, spread footings should be embedded at a minimum depth of 18 inches below exterior grade. Foundations should be designed by a licensed structural engineer. Due to the presence of soft to medium stiff soil identified within the upper two to three feet of the ground surface, we recommend either extending the footings to a minimum depth of 2 to 2½ feet bgs, or removal of the soft soils to the noted depth, and replacement with compacted crushed aggregate.

The anticipated allowable soil bearing pressure is 1,500 lbs/ft^2 for footings bearing on competent, native soil and/or engineered fill. The anticipated allowable soil bearing pressure is 2,000 lbs/ft^2 for footings bearing on a minimum of 12 inches of compacted crushed aggregate. The recommended maximum allowable bearing pressure may be increased by 1/3 for short-term transient conditions such as wind and seismic loading. For heavier loads, the geotechnical engineer should be consulted. If heavier loads than described above are proposed, it may be necessary to over-excavate point load areas and replace with compacted crushed aggregate. The coefficient of friction between on-site soil and poured-in-place concrete may be taken as 0.42, which includes no factor of safety. The maximum anticipated total and differential footing movements (generally from soil expansion and/or settlement) are 1 inch and ¾ inch over a span of 20 feet, respectively. We anticipate that the majority of the estimated settlement will occur during construction, as loads are applied. Excavations near structural footings should not extend within a 1H:1V plane projected downward from the bottom edge of footings.

Footing excavations should penetrate through topsoil and any disturbed soil to competent subgrade that is suitable for bearing support. All footing excavations should be trimmed neat, and all loose or softened soil should be removed from the excavation bottom prior to placing reinforcing steel bars. Due to the moisture sensitivity of on-site native soils, foundations constructed during the wet weather season may require over-excavation of footings and backfill with compacted, crushed aggregate.

Our recommendations are for residential construction incorporating raised wood floors and conventional spread footing foundations. After site development, a Final Soil Engineer's Report should either confirm or modify the above recommendations.

Concrete Slab-on-Grade

Preparation of areas beneath concrete slab-on-grade floors should be performed as recommended in the *Site Preparation Recommendations* section. Care should be taken during excavation for foundations and floor slabs, to avoid disturbing subgrade soils. If subgrade soils have been adversely impacted by wet weather or otherwise disturbed, the surficial soils should be scarified to a minimum depth of 8 inches, moisture conditioned to within about 3 percent of optimum moisture content, and compacted to engineered fill specifications. Alternatively, disturbed soils may be removed and the removal zone backfilled with additional crushed rock.

For evaluation of the concrete slab-on-grade floors using the beam on elastic foundation method, a modulus of subgrade reaction of 150 kcf (87 pci) should be assumed for the medium-stiff, fine-grained soils anticipated to be present in the upper four feet at the site. This value assumes the concrete slab system is designed and constructed as recommended herein, with a minimum thickness of 12 inches of 1½"-0 crushed aggregate beneath the slab. The total thickness of crushed aggregate will be dependent on the subgrade conditions at the time of construction, and should be verified visually by proof-rolling. Under-slab aggregate should be compacted to at least 95 percent of its maximum dry density as determined by ASTM D1557 (Modified Proctor) or equivalent.

In areas where moisture will be detrimental to floor coverings or equipment inside the proposed structure, appropriate vapor barrier and damp-proofing measures should be implemented. A commonly applied vapor barrier system consists of a 10-mil polyethylene vapor barrier placed directly over the capillary break material. Other damp/vapor barrier systems may also be feasible. Appropriate design professionals should be consulted regarding vapor barrier and damp proofing systems, ventilation, building material selection and mold prevention issues, which are outside GeoPacific's area of expertise.

Footing and Roof Drains

Construction should include typical measures for controlling subsurface water beneath the structures, including positive crawlspace drainage to an adequate low-point drain exiting the foundation, visqueen covering the expose ground in the crawlspace, and crawlspace ventilation (foundation vents). Some slow flowing water in the crawlspaces is considered normal and not necessarily detrimental to the home given these other design elements incorporated into its construction. Appropriate design professionals should be consulting regarding crawlspace ventilation, building material selection and mold prevention issues, which are outside GeoPacific's area of expertise.

Down spouts and roof drains should collect roof water in a system separate from the footing drains to reduce the potential for clogging. Roof drain water should be directed to an appropriate discharge point and storm system well away from structural foundations. Grades should be sloped downward and away from buildings to reduce the potential for ponded water near structures.

If the proposed structures will have a raised floor, and no concrete slab-on-grade floors in living spaces are used, perimeter footing drains would not be required based on soil conditions encountered at the site and experience with standard local construction practices. Where it is desired to reduce the potential for moist crawl spaces, footing drains may be installed. If concrete slab-on-grade floors are used, perimeter footing drains should be installed as recommended below.

Where necessary, perimeter footing drains should consist of 3 or 4-inch diameter, perforated plastic pipe embedded in a minimum of 1 ft^3 per lineal foot of clean, free-draining drain rock. The drain pipe and surrounding drain rock should be wrapped in non-woven geotextile (Mirafi 140N, or approved equivalent) to minimize the potential for clogging and/or ground loss due to piping. A minimum 0.5 percent fall should be maintained throughout the drain and non-perforated pipe outlet. Figure 7 presents a typical perimeter footing drain detail. In our opinion, footing drains may outlet at the curb, or on the back sides of lots where sufficient fall is not available to allow drainage to meet the street.

Permanent Below-Grade Walls

Lateral earth pressures against below-grade retaining walls will depend upon the inclination of any adjacent slopes, type of backfill, degree of wall restraint, method of backfill placement, degree of backfill compaction, drainage provisions, and magnitude and location of any adjacent surcharge loads. At-rest soil pressure is exerted on a retaining wall when it is restrained against rotation. In contrast, active soil pressure will be exerted on a wall if its top is allowed to rotate or yield a distance of roughly 0.001 times its height or greater.

If the subject retaining walls will be free to rotate at the top, they should be designed for an active earth pressure equivalent to that generated by a fluid weighing 35 pcf for level backfill against the wall. For restrained wall, an at-rest equivalent fluid pressure of 55 pcf should be used in design, again assuming level backfill against the wall. These values assume that the recommended drainage provisions are incorporated, and hydrostatic pressures are not allowed to develop against the wall.

During a seismic event, lateral earth pressures acting on below-grade structural walls will increase by an incremental amount that corresponds to the earthquake loading. Based on the Mononobe-Okabe equation and peak horizontal accelerations appropriate for the site location, seismic loading should be modeled using the active or at-rest earth pressures recommended above, plus an incremental rectangular-shaped seismic load of magnitude 6.5H, where H is the total height of the wall.

We assume relatively level ground surface below the base of the walls. As such, we recommend passive earth pressure of 300 pcf for use in design, assuming wall footings are cast against competent native soils or engineered fill. If the ground surface slopes down and away from the base of any of the walls, a lower passive earth pressure should be used and GeoPacific should be contacted for additional recommendations.

A coefficient of friction of 0.42 may be assumed along the interface between the base of the wall footing and subgrade soils. The recommended coefficient of friction and passive earth pressure values do not include a safety factor, and an appropriate safety factor should be included in design. The upper 12 inches of soil should be neglected in passive pressure computations unless it is protected by pavement or slabs on grade.

The above recommendations for lateral earth pressures assume that the backfill behind the subsurface walls will consist of properly compacted structural fill, and no adjacent surcharge loading. If the walls will be subjected to the influence of surcharge loading within a horizontal distance equal to or less than the height of the wall, the walls should be designed for the additional horizontal pressure. For uniform surcharge pressures, a uniformly distributed lateral pressure of 0.3 times the surcharge pressure should be added. Traffic surcharges may be estimated using an additional vertical load of 250 psf (2 feet of additional fill), in accordance with local practice.

The recommended equivalent fluid densities assume a free-draining condition behind the walls so that hydrostatic pressures do not build-up. This can be accomplished by placing a 12 to 18-inch wide zone of sand and gravel containing less than 5 percent passing the No. 200 sieve against the walls. A 3-inch minimum diameter perforated, plastic drain pipe should be installed at the base of the walls and connected to a suitable discharge point to remove water in this zone of sand and gravel. The drain pipe should be wrapped in filter fabric (Mirafi 140N or other as approved by the geotechnical engineer) to minimize clogging.

Wall drains and perimeter drains are recommended to prevent detrimental effects of surface water runoff on foundations – not to dewater groundwater. Drains should not be expected to eliminate all potential sources of water entering a basement or beneath a slab-on-grade. An adequate grade to a low point outlet drain in the crawlspace is required by code. Underslab drains are sometimes added beneath the slab when placed over soils of low permeability and shallow, perched groundwater. At present we are not recommending underslab drains.

Water collected from the wall drains and perimeter drains should be directed into the local storm drain system or other suitable outlet. A minimum 0.5 percent fall should be maintained throughout the drain and non-perforated pipe outlet. Down spouts and roof drains should not be connected to the wall drains in order to reduce the potential for clogging. The drains should include clean-outs to allow periodic maintenance and inspection. Grades around the proposed structure should be sloped such that surface water drains away from the building.

GeoPacific should be contacted during construction to verify subgrade strength in wall keyway excavations, to verify that backslope soils are in accordance with our assumptions, and to take density tests on the wall backfill materials.

Structures should be located a horizontal distance of at least 1.5H away from the back of the retaining wall, where H is the total height of the wall. GeoPacific should be contacted for additional foundation recommendations where structures are located closer than 1.5H to the top of any wall.

Seismic Design

The Oregon Department of Geology and Mineral Industries (DOGAMI), Oregon HazVu: 2015 Statewide GeoHazards Viewer indicates that the site is located in an area considered to be at risk for *severe* shaking during a seismic event. Structures should be designed to resist earthquake loading in accordance with the methodology described in the 2015 International Building Code (IBC) with applicable Oregon Structural Specialty Code (OSSC) revisions (current 2014). We recommend Site Class D be used for design per the OSSC, Table 1613.5.2 and as defined in ASCE 7, Chapter 20, Table 20.3-1. Design values determined for the site using the USGS (United States Geological Survey) *2016 Seismic Design Maps Summary Report* are summarized in Table 3.

Table 3 - Recommended Earthquake Ground Motion Parameters (USGS 2016)

Parameter	Value
Location (Lat, Long), degrees	45.394, -122.639
Probabilistic Ground Motion Values, 2% Probability of Exceedance in 50 yrs	
Peak Ground Acceleration PGA_M	0.452 g
Short Period, S_s	0.969 g
1.0 Sec Period, S_1	0.415 g
Soil Factors for Site Class D:	
F_a	1.113
F_v	1.585
$SD_s = 2/3 \times F_a \times S_s$	0.718 g
$SD_1 = 2/3 \times F_v \times S_1$	0.438 g
Seismic Design Category	D

Soil Liquefaction

The Oregon Department of Geology and Mineral Industries (DOGAMI), Oregon HazVu: 2015 Statewide GeoHazards Viewer indicates that the site is in an area considered to be at *high* risk for soil liquefaction during an earthquake. Soil liquefaction is a phenomenon wherein saturated soil deposits temporarily lose strength and behave as a liquid in response to ground shaking caused by strong earthquakes. Soil liquefaction is generally limited to loose, sands and granular soils located below the water table, and fine-grained soils with a plasticity index less than 15. The site was observed to be underlain by medium stiff, fine-grained, low to moderately plastic soils consisting of Sandy SILT, Silty SAND, and Clayey GRAVEL, located above the static groundwater table. Based upon the lack of groundwater observed within the subsurface profile at the time of our site investigation, it is our opinion that the risk of soil liquefaction during a seismic event at the subject site should be considered to be low.

UNCERTAINTIES AND LIMITATIONS

We have prepared this report for the owner and his/her consultants for use in design of this project only. The conclusions and interpretations presented in this report should not be construed as a warranty of the subsurface conditions. Experience has shown that soil and groundwater conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations that may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, GeoPacific should be notified for review of the recommendations of this report, and revision of such if necessary.

Sufficient geotechnical monitoring, testing, and consultation should be provided during construction to confirm that the conditions encountered are consistent with those indicated by subsurface explorations. The checklist attached to this report outlines recommended geotechnical observations and testing for the project. Recommendations for design changes will be provided should conditions revealed during construction differ from those anticipated, and to verify that the geotechnical aspects of construction comply with the contract plans and specifications.

Within the limitations of scope, schedule and budget, GeoPacific executed these services in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology at the time the report was prepared. No warranty, express or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or groundwater at this site.

We appreciate this opportunity to be of service.

Sincerely,

GEOPACIFIC ENGINEERING, INC.

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OREGON
BENJAMIN L COOK
2245
GEOLOGIST
EXP 12/31/17

REGISTERED PROFESSIONAL
ENGINEER
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JAN. 23, 1990
JAMES D. IMBRIE
EXPIRES: 06/30/2017

Benjamin L. Cook, R.G.
Senior Geologist

James D. Imbrie, G.E., C.E.G.
Principal Geotechnical Engineer

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FIGURES

GeoPacific Engineering, Inc.
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VICINITY MAP

Base map: Google Maps, 2016

Date: 12/27/2016
Drawn by: BLC

Project: Old River Partition 18822 Old River Drive West Linn, Oregon 97068	Project No. 16-4408	FIGURE 1

GeoPacific Engineering, Inc.
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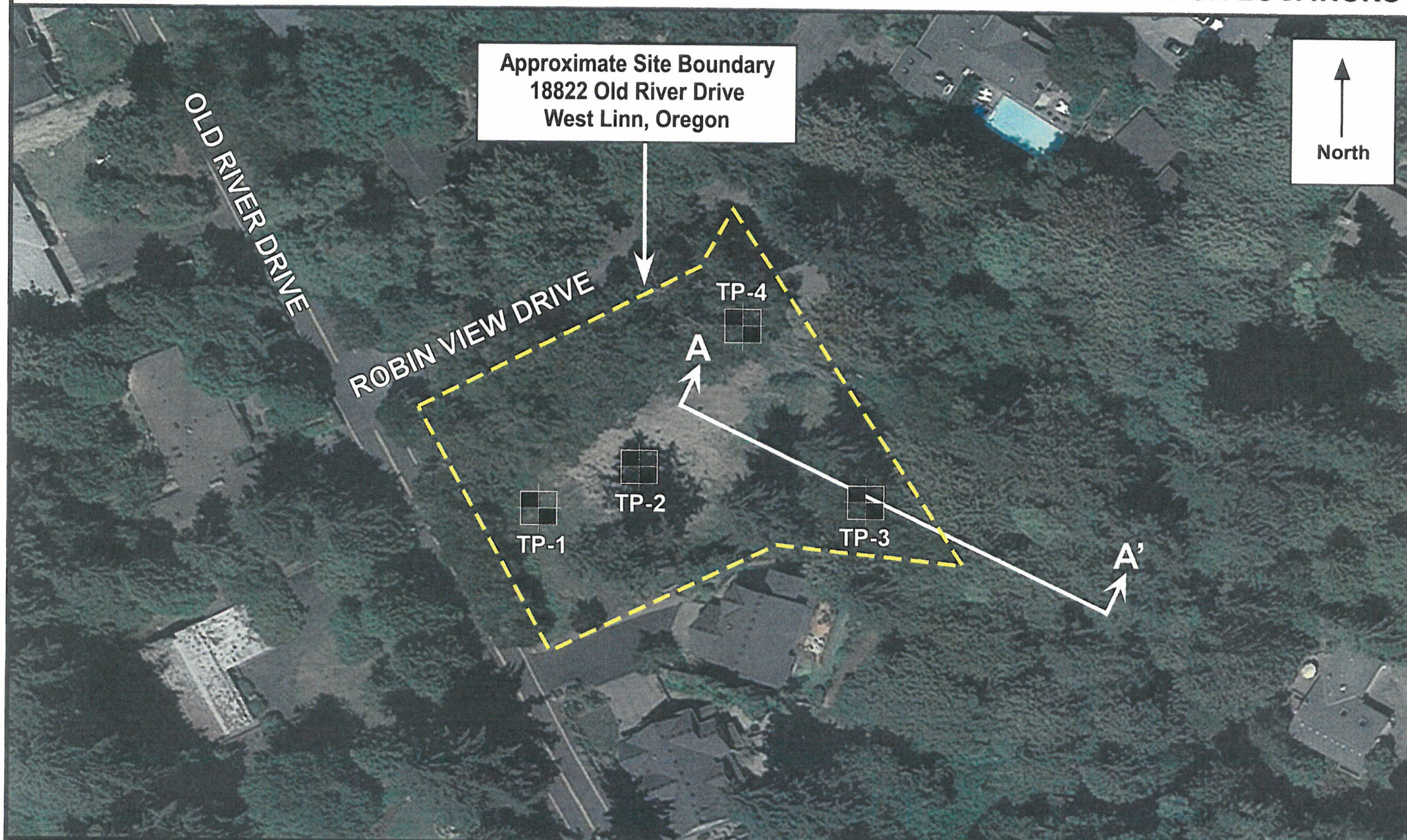
SITE AERIAL AND EXPLORATION LOCATIONS



Legend:
Test Pit Exploration Approximate Location

Slope Cross Section

APPROXIMATE SCALE (FEET)
30 0 30 60

Date:12/27/2016
Drawn by: BLC

Project: Old River Partition 18822 Old River Drive, West Linn, Oregon 97068	Project No. 16-4408	FIGURE 2

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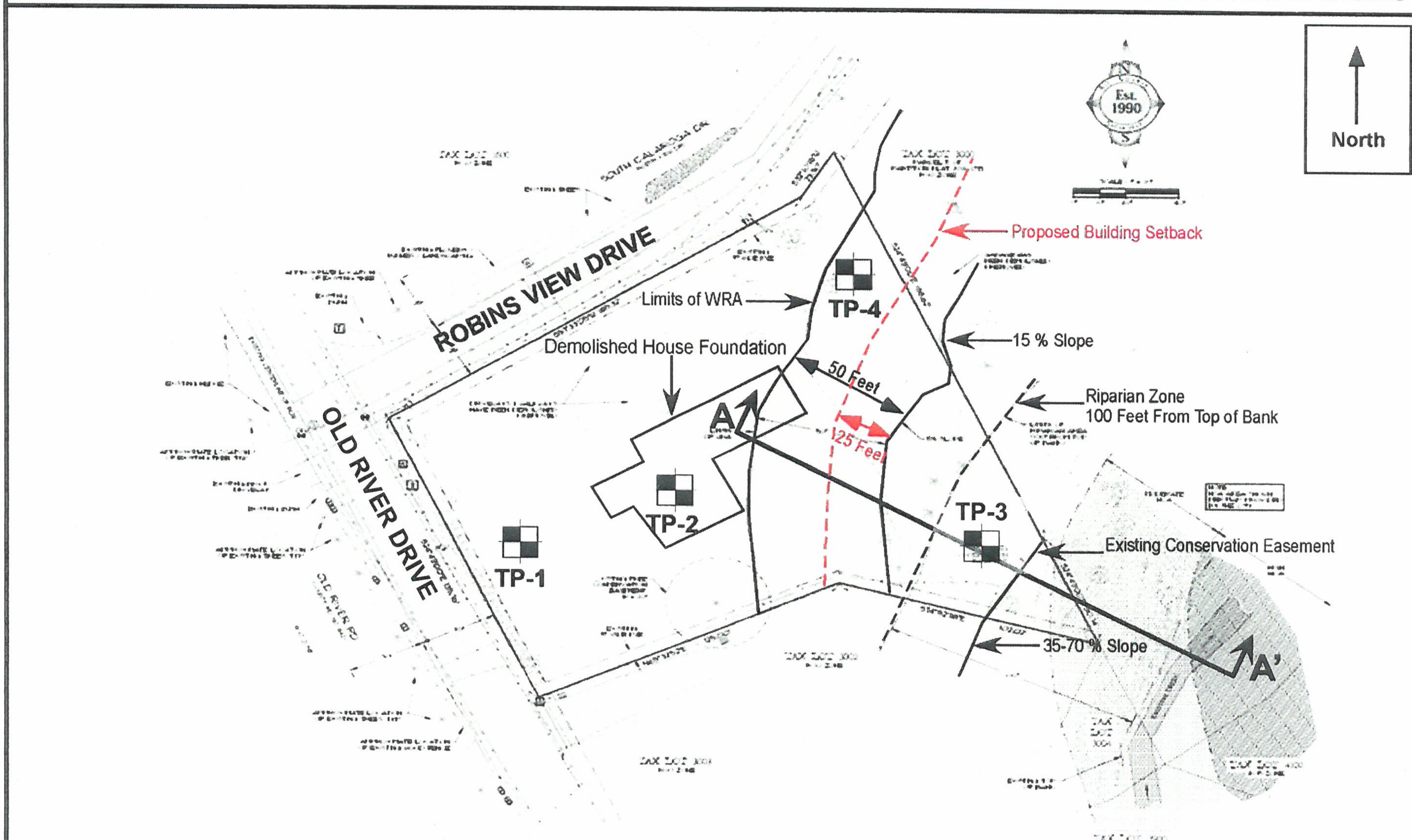
SITE PLAN AND EXPLORATION LOCATIONS



Legend:
Test Pit Exploration Approximate Location

Slope Cross Section

APPROXIMATE SCALE (FEET)
35 0 35 70

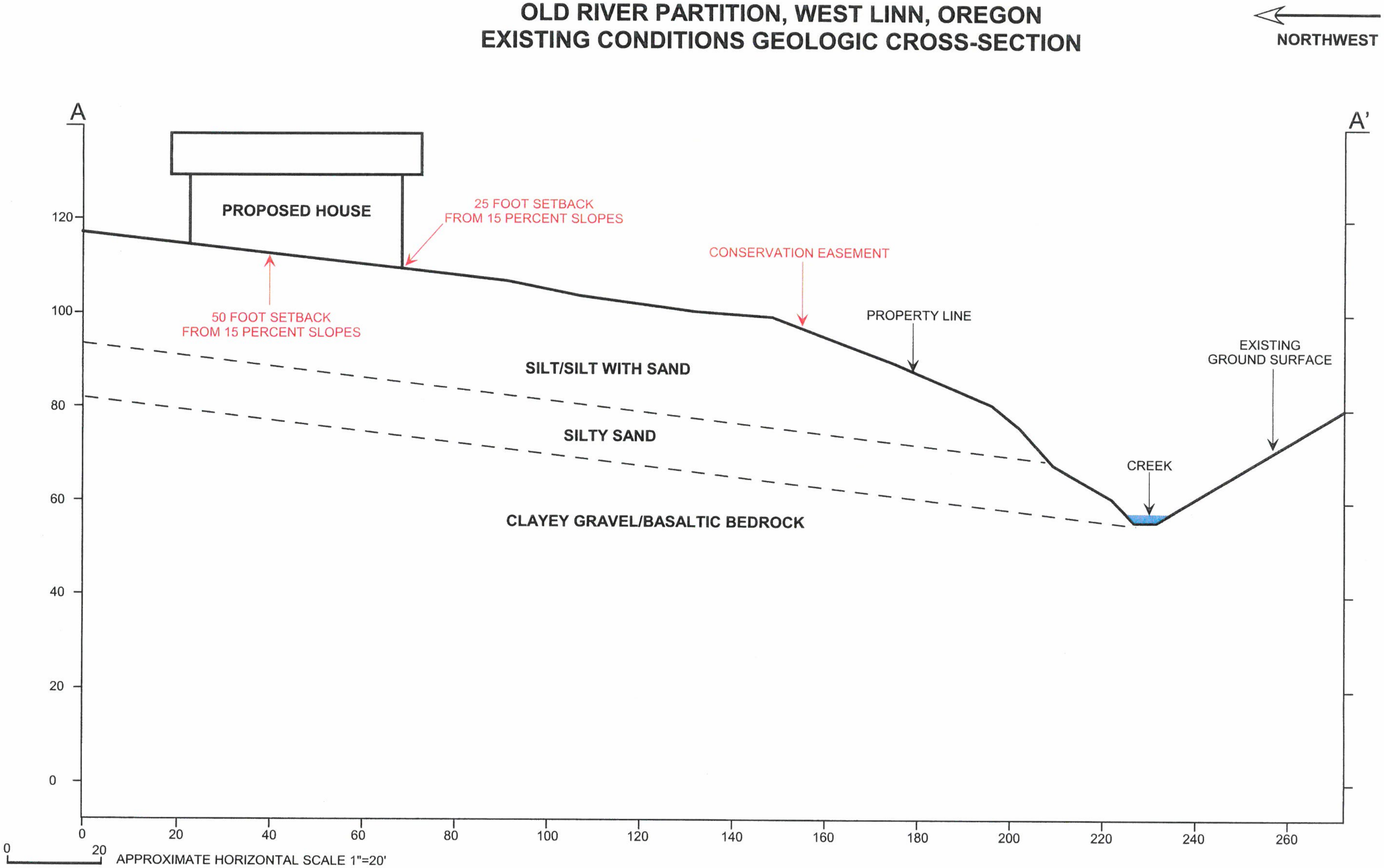
Date: 12/27/2016
Drawn by: BLC

Project: Old River Partition
18822 Old River Drive, West Linn, Oregon 97068

Project No. 16-4408

FIGURE 3

OLD RIVER PARTITION, WEST LINN, OREGON
EXISTING CONDITIONS GEOLOGIC CROSS-SECTION



Note: Location of all geotechnical information is approximate.
Topographic data is based on topographic site survey dated October 21, 2016.
Soil conditions are based upon subsurface exploration conducted 12/6/2016

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GEOLOGIC CROSS-SECTION - A to A'

Project: Old River Partition 18822 Old River Drive, West Linn, OR	Project No. 16-4408	Drawn By:BLC 12/27/16	FIGURE 4

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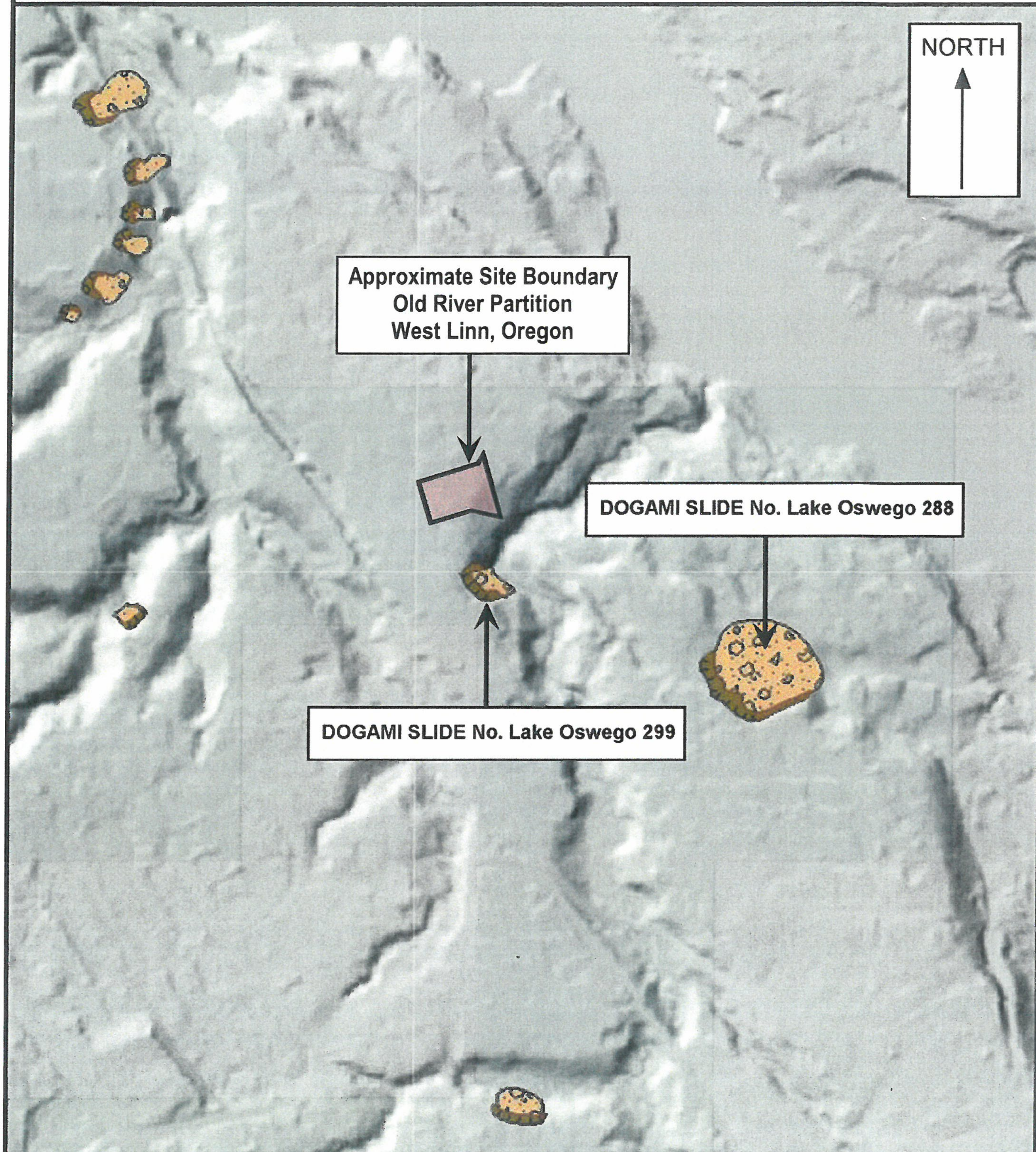
LANDSLIDE HAZARD MAP



Legend: Mapped Landslide

0 500'
APPROXIMATE SCALE 1"=500'

Date: 12/27/16
Drawn by: BLC

Map Source: *Oregon Department of Geology and Mineral Industries*
SLIDO: Statewide Landslide Information 2016 Website

Project: Old River Partition 18822 Old River Drive West Linn, Oregon 97068	Project No. 16-4408	FIGURE 5

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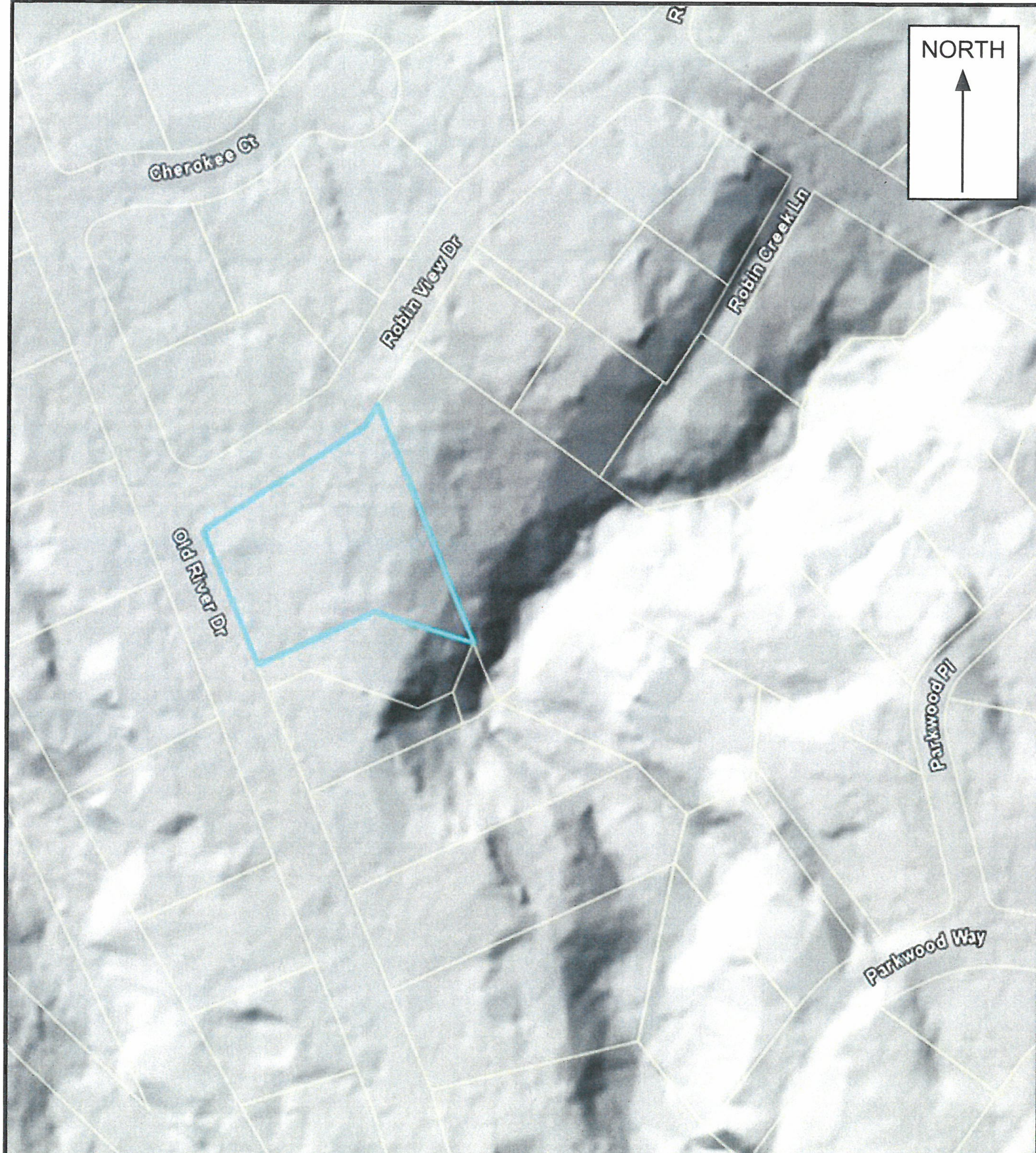
LIDAR PROPERTY MAP



Map Source: *Clackamas County Maps Online, 2016*

Date: 12/27/16
Drawn by: BLC

Project: Old River Partition 18822 Old River Drive West Linn, Oregon 97068	Project No. 16-4408	FIGURE 6

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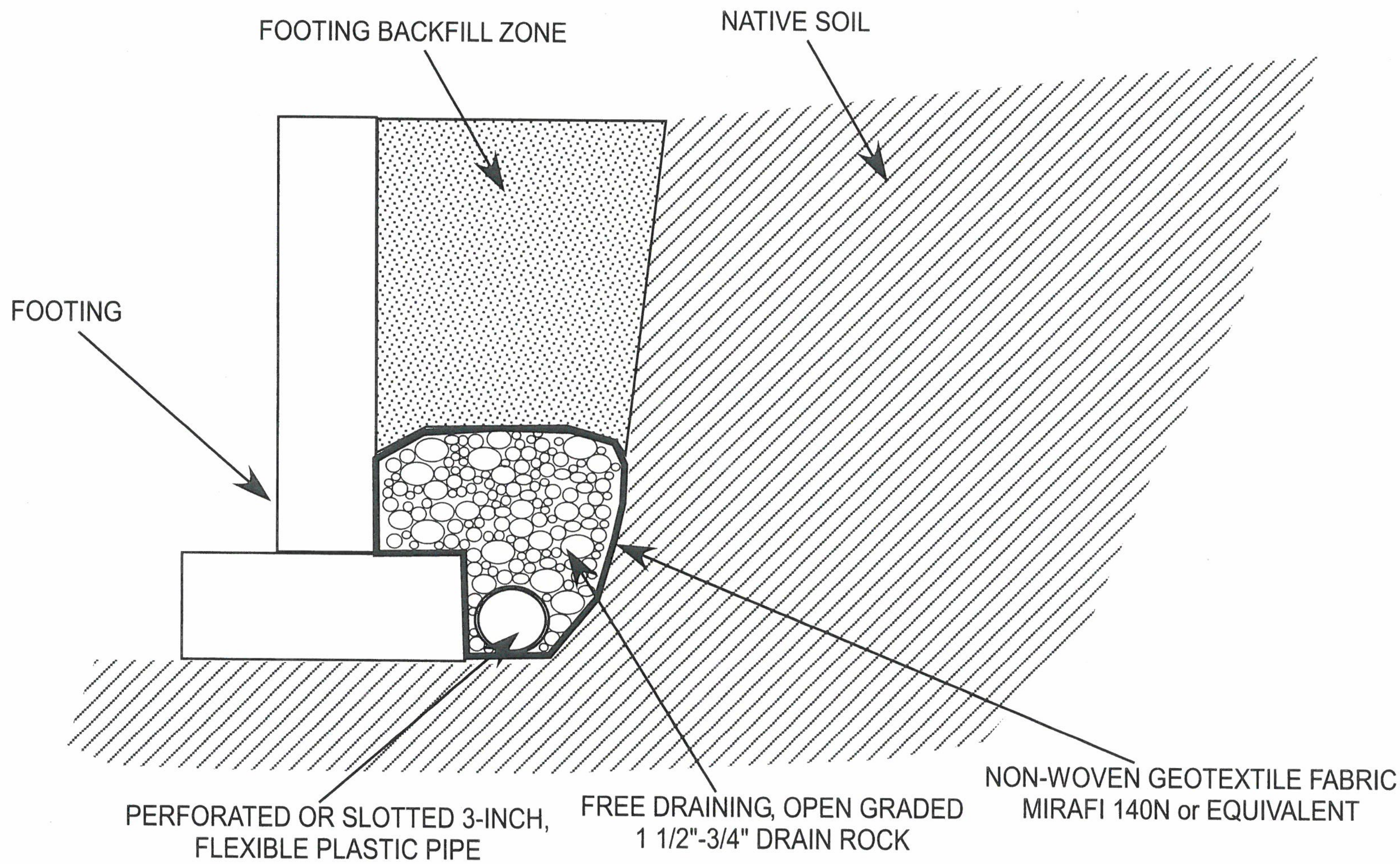
TYPICAL PERIMETER FOOTING DRAIN DETAIL



Notes:
1) Drain rock should contain no more than 5 percent fines passing the U.S. No. 200 Sieve.
2) Trench bottom and drain pipe should be sloped to drain to approved discharge location.

Date: 12/27/2016
Drawn by: BLC

Project: Old River Partition 18822 Old River Drive, West Linn, Oregon 97068	Project No. 16-4408	FIGURE 7

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EXPLORATION LOGS

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TEST PIT LOG

Project: Old River Partition West Linn, Oregon	Project No. 16-4408	Test Pit No. **TP-1**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Type	% Passing No. 200 Sieve	Moisture Content (%)	Water Bearing Zone	Material Description
0 – 0.7						Topsoil. Grassy area underlain by approximately 8-inches dark brown, wet, moderately organic SILT (OL-ML).
1	1.5					SILT/SILT with Sand (ML), brown, very moist, medium stiff, moderately plastic.
2	2.0					
3	2.5	Bag Sample (100 to 1,000 g)	85.5	33.6		
4	3.5					Sandy SILT (ML), brown with orange mottling, very moist, medium stiff, low to moderately plasticity.
5						
6		Bag Sample (100 to 1,000 g)	60.6	35.2		
7						
8						SILT with Sand (ML), brown, very moist, medium stiff, low to moderate plasticity.
9		Bag Sample (100 to 1,000 g)	93.9	33.5		
10						
11						Test pit terminated at 11 feet bgs. No groundwater seepage observed.
12						
13						
14						
15						
16						
17						

LEGEND

100 to 1,000 g — Bag Sample
5 Gal. Bucket — Bucket Sample
Shelby Tube Sample
Seepage
Water Bearing Zone
Water Level at Abandonment

Date Excavated: 12/6/2016
Logged By: B. Cook
Surface Elevation: 117 feet

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TEST PIT LOG

Project: Old River Partition West Linn, Oregon	Project No. 16-4408	Test Pit No. TP-2

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Type	% Passing No. 200 Sieve	Moisture Content (%)	Water Bearing Zone	Material Description
0–7						FILL. Basement backfill. Crushed aggregate and soil mixture, very moist, loose, containing bricks, plastic, piping, rain gutters, and roofing. Easy excavation. Non-structural material.
7–8						Sandy SILT (ML), brown with orange mottling, very moist, medium stiff, low to moderately plasticity.
8–12						SILT with Sand (ML), brown, very moist, medium stiff, low to moderate plasticity.
12						Test pit terminated at 12 feet bgs. No groundwater seepage observed.

LEGEND

Bag Sample (100 to 1,000 g)	Bucket Sample (5 Gal. Bucket)	Shelby Tube Sample	Seepage	Water Bearing Zone	Water Level at Abandonment

Date Excavated: 12/6/2016
Logged By: B. Cook
Surface Elevation: 117 feet

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: Old River Partition
West Linn, Oregon

Project No. 16-4408

Test Pit No. **TP-3**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	% Passing No. 200 Sieve	Moisture Content (%)	Water Bearing Zone	**Material Description**
						Topsoil. Grassy area underlain by approximately 8-inches dark brown, wet, moderately organic SILT (OL-ML).
1	1.0					
						SILT/SILT with Sand (ML), brown, very moist, medium stiff, moderately plastic. AASHTO Classification = A-7-6(16), Liquid Limit = 45.5, Plasticity Index = 17.6
2	1.5					
3	2.5	100 to 1,000 g	80.3	33.2		
						Sandy SILT (ML), brown with orange mottling, very moist, medium stiff, low to moderately plasticity. AASHTO Classification = A-4(5), Liquid Limit = 37.4, Plasticity Index = 8.2
4	3.5					
5						
6		100 to 1,000 g	60.6	35.2		
7						
8						
						SILT with Sand (ML), brown, very moist, medium stiff, low to moderate plasticity. AASHTO Classification = A-4(6), Liquid Limit = 37.0, Plasticity Index = 8.8
9		100 to 1,000 g	93.9	33.5		
10						
11						
12						Test pit terminated at 12 feet bgs. No groundwater seepage observed.
13						
14						
15						
16						
17						

LEGEND

100 to 1,000 g — Bag Sample
5 Gal. Bucket — Bucket Sample
Shelby Tube Sample
Seepage
Water Bearing Zone
Water Level at Abandonment

Date Excavated: 12/6/2016
Logged By: B. Cook
Surface Elevation: 107 feet

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TEST PIT LOG

Project: Old River Partition
West Linn, Oregon

Project No. 16-4408

Test Pit No. **TP-4**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Type	% Passing No. 200 Sieve	Moisture Content (%)	Water Bearing Zone	Material Description
						Topsoil. Grassy area underlain by approximately 8-inches dark brown, wet, moderately organic SILT (OL-ML).
1	1.0					SILT/SILT with Sand (ML), brown, very moist, medium stiff, moderately plastic.
2	2.0					
3	2.5					
4	3.0					Sandy SILT (ML), brown with orange mottling, very moist, medium stiff, low to moderately plasticity.
5						
6						
7						
8						SILT with Sand (ML), brown, very moist, medium stiff, low to moderate plasticity.
9						
10						
11						Test pit terminated at 11 feet bgs. No groundwater seepage observed.
12						
13						
14						
15						
16						
17						

LEGEND

100 to 1,000 g — Bag Sample
5 Gal. Bucket — Bucket Sample
Shelby Tube Sample
Seepage
Water Bearing Zone
Water Level at Abandonment

Date Excavated: 12/6/2016
Logged By: B. Cook
Surface Elevation: 115 feet

GeoPacific Engineering, Inc.
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Portland, Oregon 97224
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BORING LOG

Project: Robins View Drive West Linn, Oregon | Project No. 16-4131 | Boring No. **B-1**

Depth (ft)	Sample Type	N-Value	% Passing the No. 200 Sienve	Moisture Content (%)	Water Bearing Zone	Material Description
						Asphalt Drive. 3 Inches A/C, 10 inches of 3/4"-0 crushed aggregate.
						Lean CLAY (CL), brown, very moist, soft to medium stiff, low to moderate plasticity.
	Split-Spoon	6				
5	Split-Spoon	3	62.2	36.0		
	Split-Spoon	3				
10	Split-Spoon	4	69.5	30.3		Sandy SILT (ML), brown, very moist, soft to medium stiff, fine sand portion, low plasticity.
15	Split-Spoon	6	53.7	23.6		
20	Split-Spoon	7	41.9	19.9		
25	Split-Spoon	9				Silty SAND (SM), light brown, damp to moist, loose to medium dense, fine to medium sand portion, low plasticity.
30	Split-Spoon	11	41.9	16.3		
35	Split-Spoon	6				**Soil boring terminated at -41.5 feet bgs. No groundwater encountered. Drill Rig: Trailer-mounted, solid-stem drill rig, Dan Fischer Excavating**
40	Split-Spoon	70				Clayey GRAVEL (GC), red brown to orange and black, Lean CLAY matrix with angular basalt, and subgrounded andesite, partial cementation, moist, dense,

LEGEND

- Bag Sample (100 to 1,000 g)
- Split-Spoon
- Shelby Tube Sample
- Static Water Table at Drilling
- Static Water Table (10-20-99)
- Water Bearing Zone

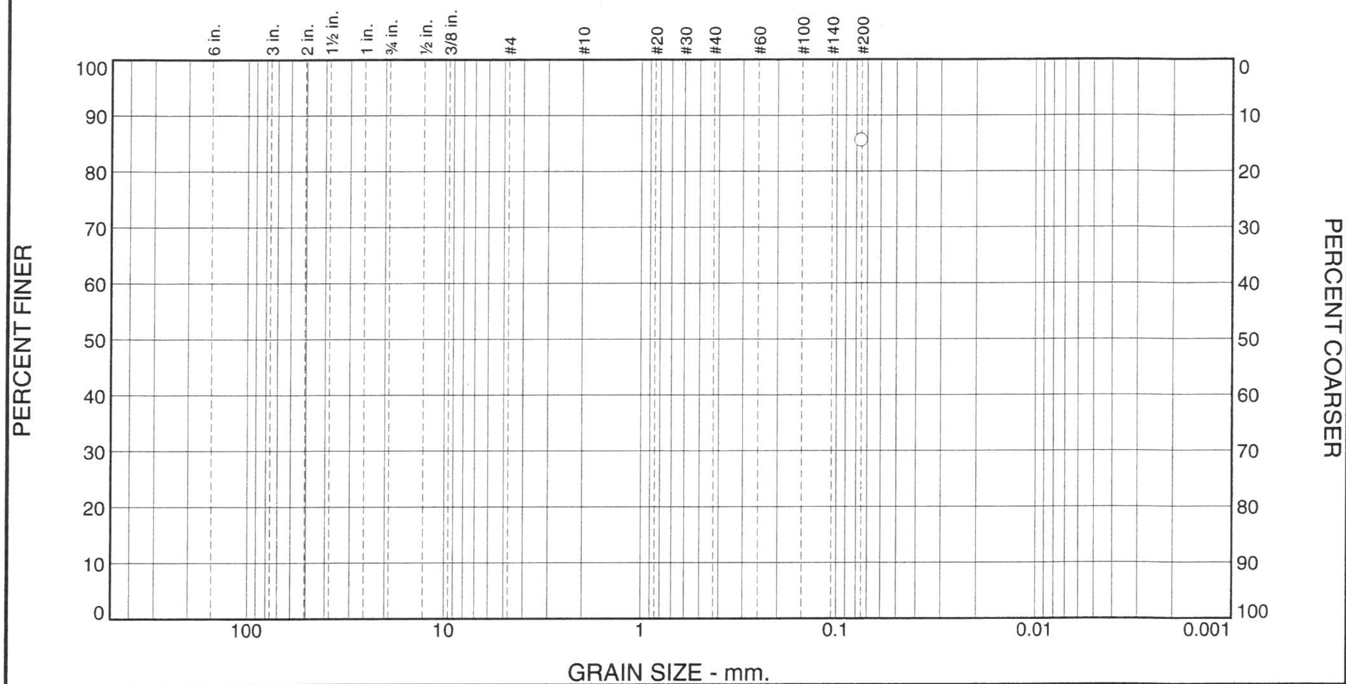
Date Drilled: 3/7/2016

Logged By: B. Cook

Surface Elevation: 106 Feet

LABORATORY TEST RESULTS

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						85.5	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	85.5		

* (no specification provided)

Material Description

Silt

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Moisture 33.6%

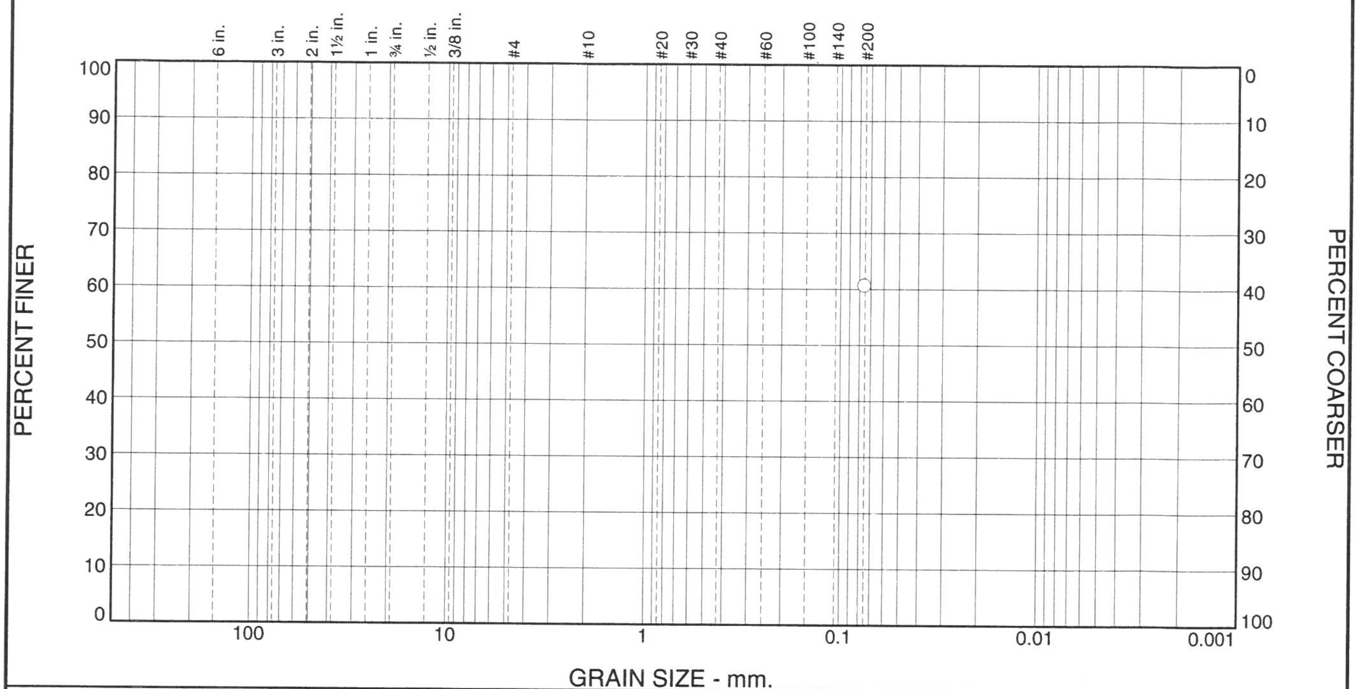
Date Received: **Date Tested:** 12/12/2016
Tested By: SJC
Checked By:
Title:

Location: TP-1 Sample 1.1
Sample Number: S16-350 **Depth:** 3'
Date Sampled: 12/6/2016

GEOPACIFIC ENGINEERING, INC.

Client: Mr. Jeff Parker
Project: Old River Partition
Project No: 16-4408 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						60.6	

TEST RESULTS			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	60.6		

* (no specification provided)

Material Description

Sandy Silt

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Moisture 35.2%

Date Received: **Date Tested:** 12/12/2016
Tested By: SJC
Checked By:
Title:

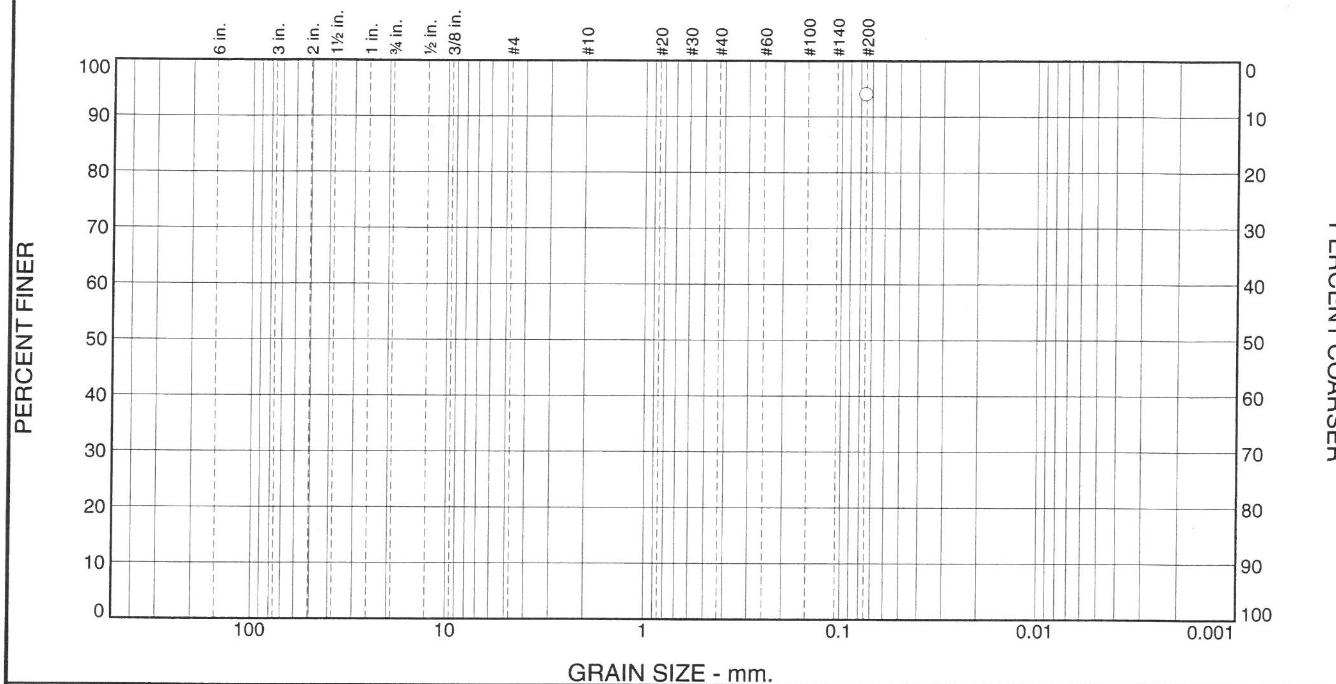
Location: TP-1 Sample 1.2
Sample Number: S16-351 **Depth:** 6' **Date Sampled:** 12/6/2016

GEOPACIFIC ENGINEERING, INC.

Client: Mr. Jeff Parker
Project: Old River Partition

Project No: 16-4408 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
						93.9	

TEST RESULTS			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
#200	93.9		

* (no specification provided)

Material Description

Silt

Atterberg Limits (ASTM D 4318)

PL= LL= PI=

Classification

USCS (D 2487)= AASHTO (M 145)=

Coefficients

D_{90}= D_{85}= D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Moisture 33.5%

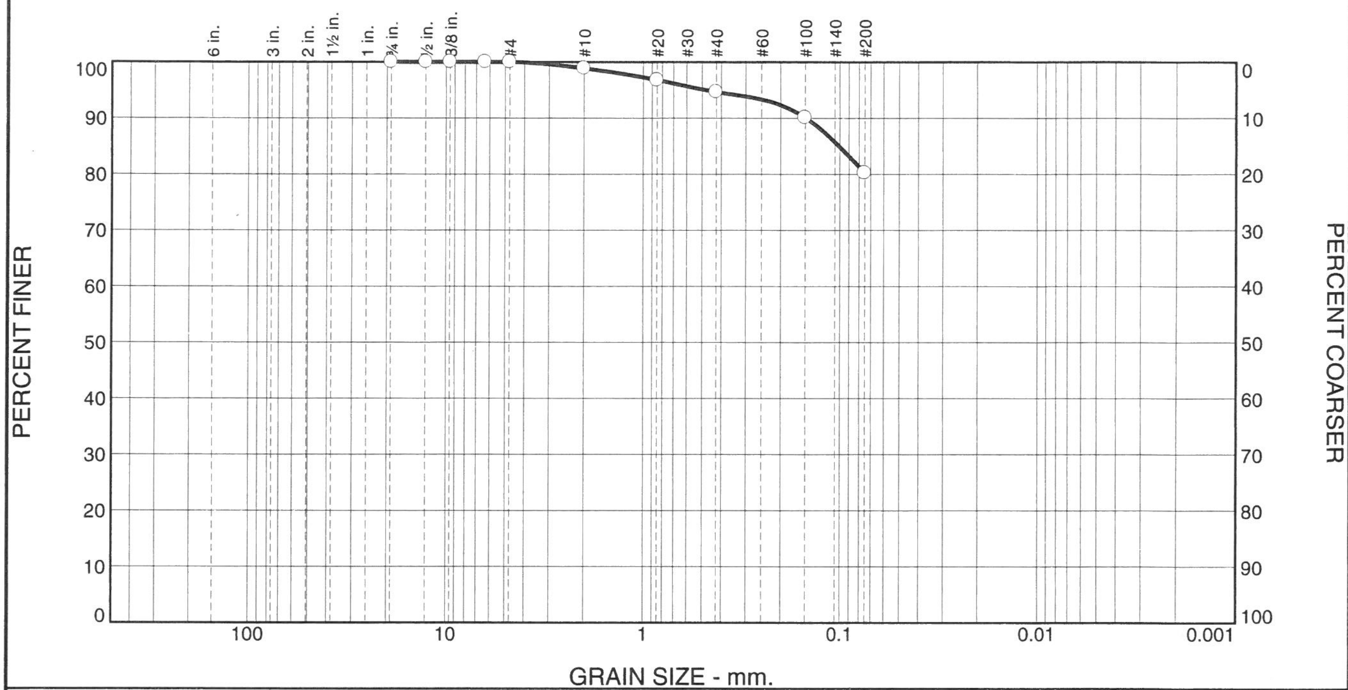
Date Received: **Date Tested:** 12/12/2016
Tested By: SJC
Checked By:
Title:

Location: TP-1 Sample 1.3
Sample Number: S16-352 **Depth:** 9' **Date Sampled:** 12/6/2016

GEOPACIFIC ENGINEERING, INC.

Client: Mr. Jeff Parker
Project: Old River Partition
Project No: 16-4408 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	1.1	4.2	14.4	80.3	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
.75	100.0		
.5	100.0		
.375	100.0		
.25	100.0		
#4	100.0		
#10	98.9		
#20	96.8		
#40	94.7		
#100	90.1		
#200	80.3		

* (no specification provided)

Material Description

Silt with Sand

Atterberg Limits (ASTM D 4318)

PL= 27.9 **LL=** 45.5 **PI=** 17.6

Classification

USCS (D 2487)= ML **AASHTO (M 145)=** A-7-6(16)

Coefficients

D_{90}= 0.1483 D_{85}= 0.1009 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Moisture 33.2%

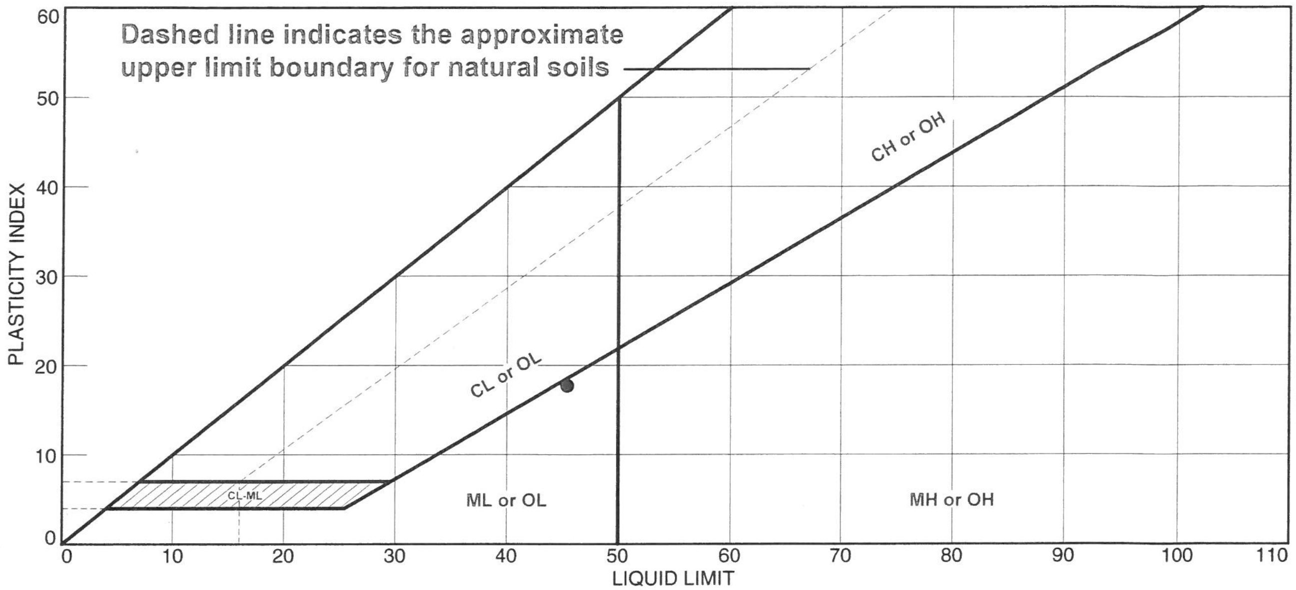
Date Received: **Date Tested:** 12/12/2016
Tested By: SJC
Checked By:
Title:

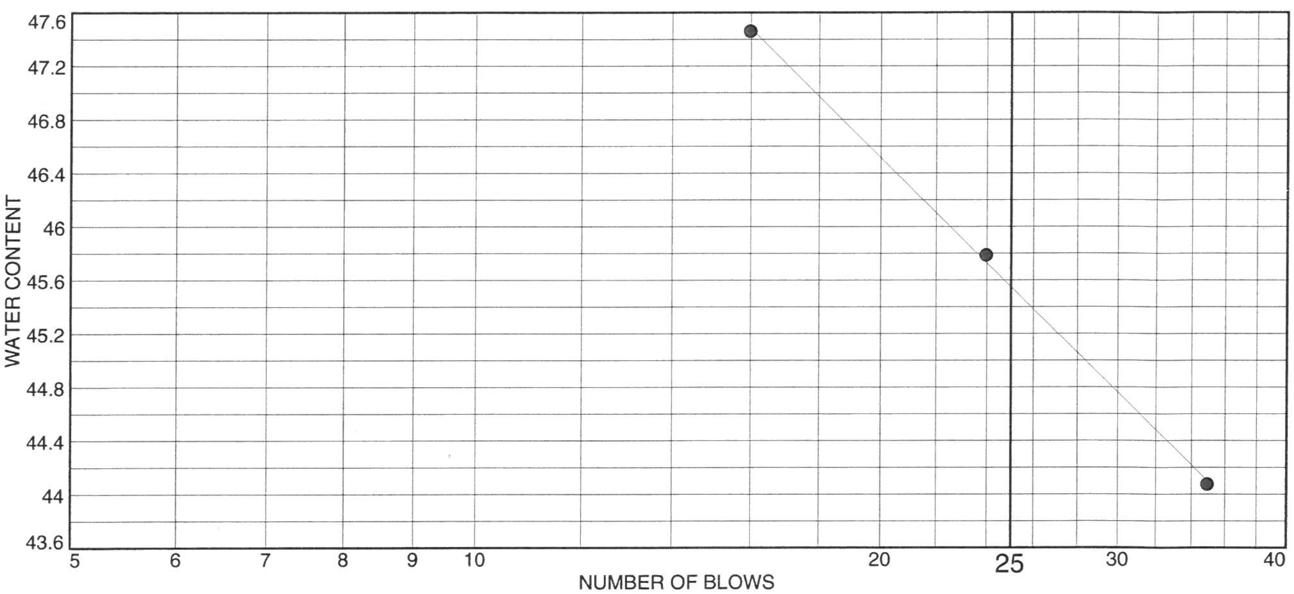
Location: TP-3 Sample 3.1
Sample Number: S16-353 **Depth:** 3'
Date Sampled: 12/6/2016

GEOPACIFIC ENGINEERING, INC.

Client: Mr. Jeff Parker
Project: Old River Partition
Project No: 16-4408 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silt with Sand	45.5	27.9	17.6	94.7	80.3	ML

Project No. 16-4408 **Client:** Mr. Jeff Parker

Project: Old River Partition

Location: TP-3 Sample 3.1
Sample Number: S16-353 **Depth:** 3'

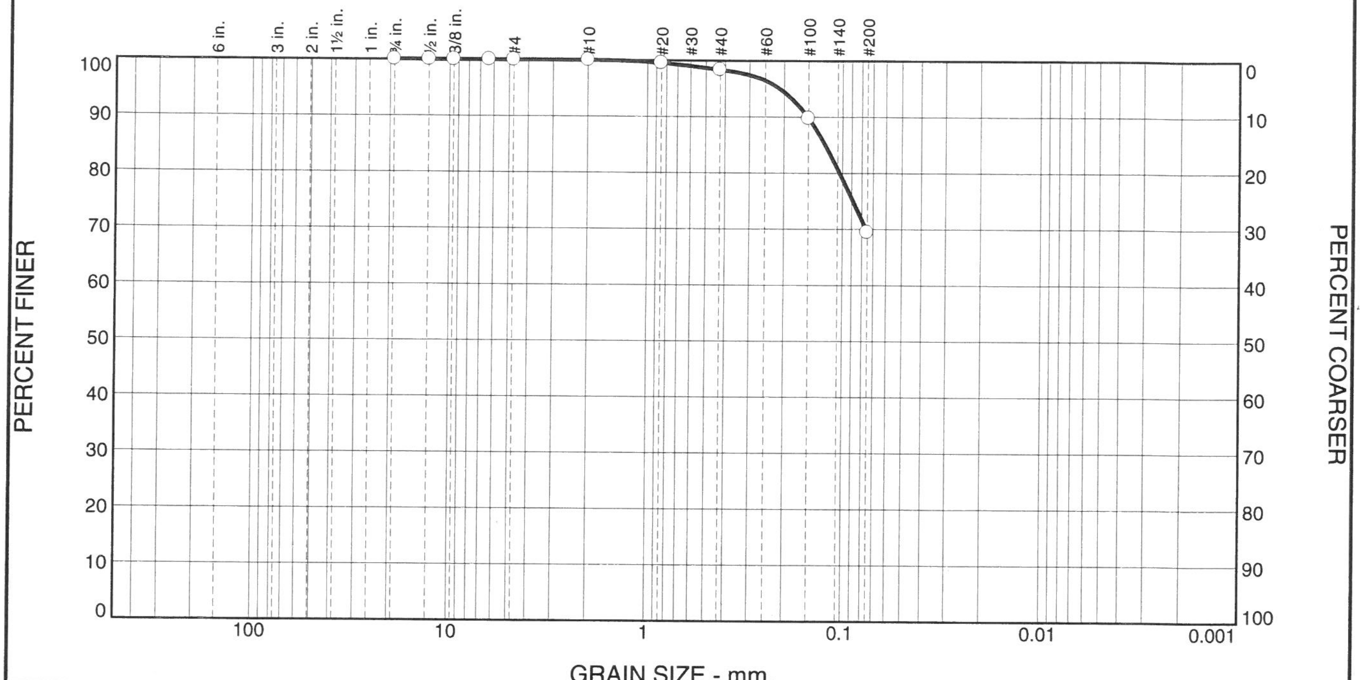
Remarks:

GEOPACIFIC ENGINEERING, INC.

Figure

Tested By: SJC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	1.7	28.8	69.5	

TEST RESULTS			
Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
.75	100.0		
.5	100.0		
.375	100.0		
.25	100.0		
#4	100.0		
#10	100.0		
#20	99.5		
#40	98.3		
#100	89.8		
#200	69.5		

* (no specification provided)

Material Description

Sandy Silt

Atterberg Limits (ASTM D 4318)

PL= 29.2 LL= 37.4 PI= 8.2

Classification

USCS (D 2487)= ML AASHTO (M 145)= A-4(5)

Coefficients

D_{90}= 0.1514 D_{85}= 0.1230 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Moisture 32.7%

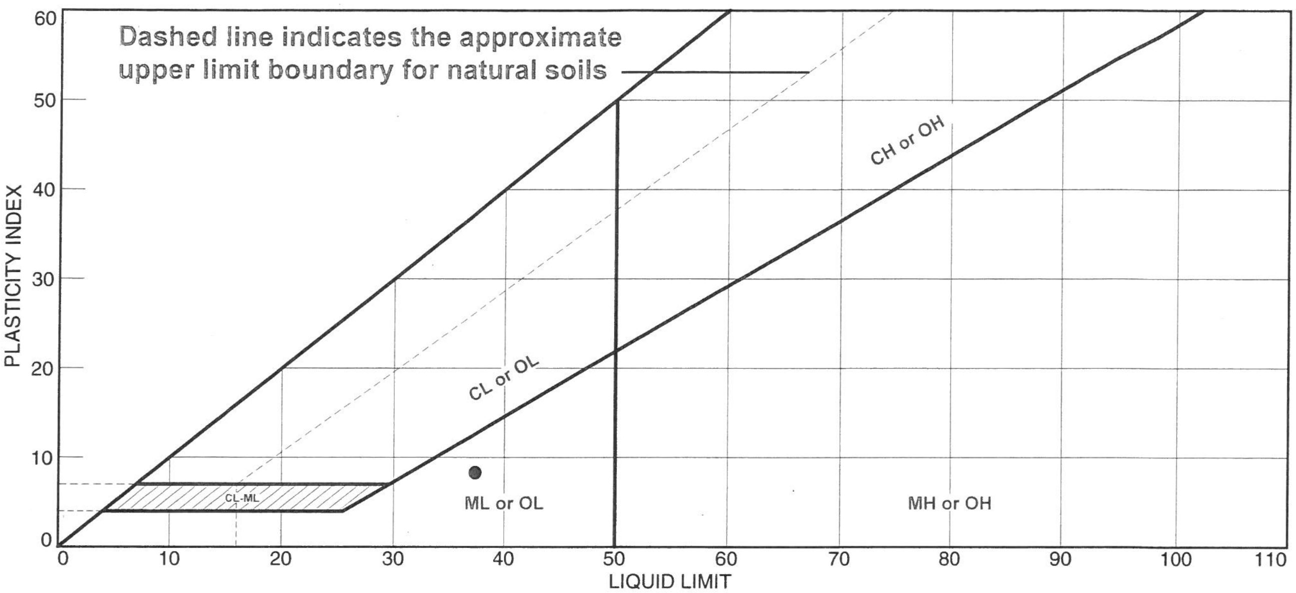
Date Received: **Date Tested:** 12/12/2016
Tested By: SJC
Checked By:
Title:

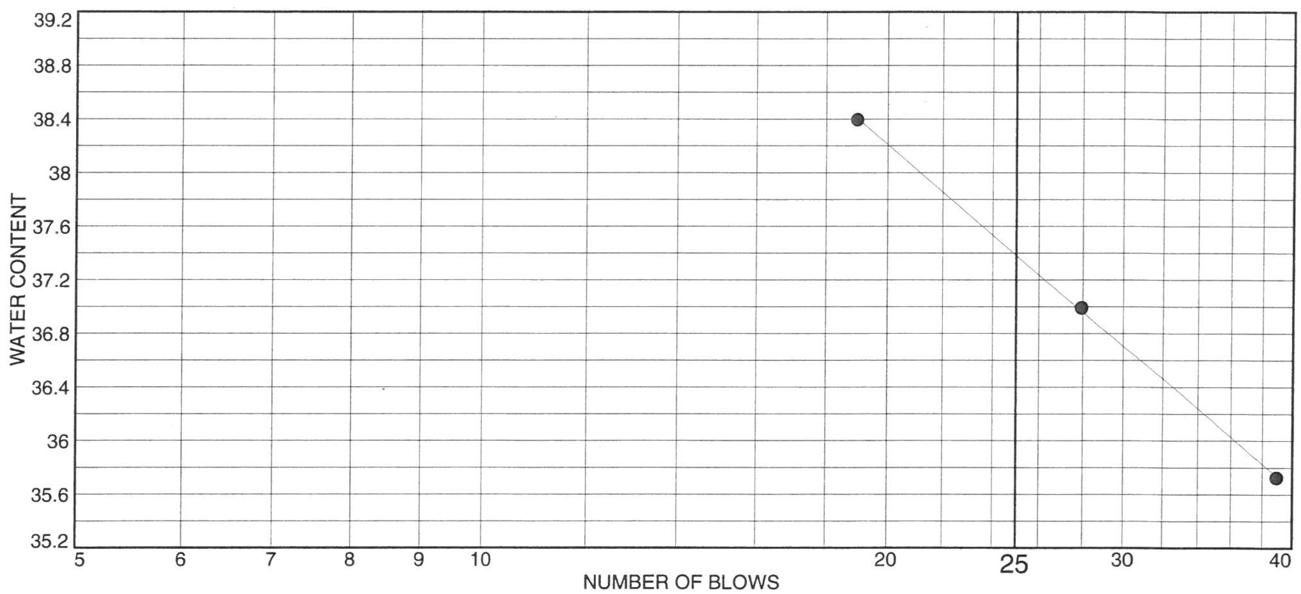
Location: TP-3 Sample 3.2
Sample Number: S16-354 **Depth:** 6' **Date Sampled:** 12/6/2016

GEOPACIFIC ENGINEERING, INC.

Client: Mr. Jeff Parker
Project: Old River Partition
Project No: 16-4408 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Sandy Silt	37.4	29.2	8.2	98.3	69.5	ML

Project No. 16-4408 **Client:** Mr. Jeff Parker

Project: Old River Partition

Location: TP-3 Sample 3.2

Sample Number: S16-354 **Depth:** 6'

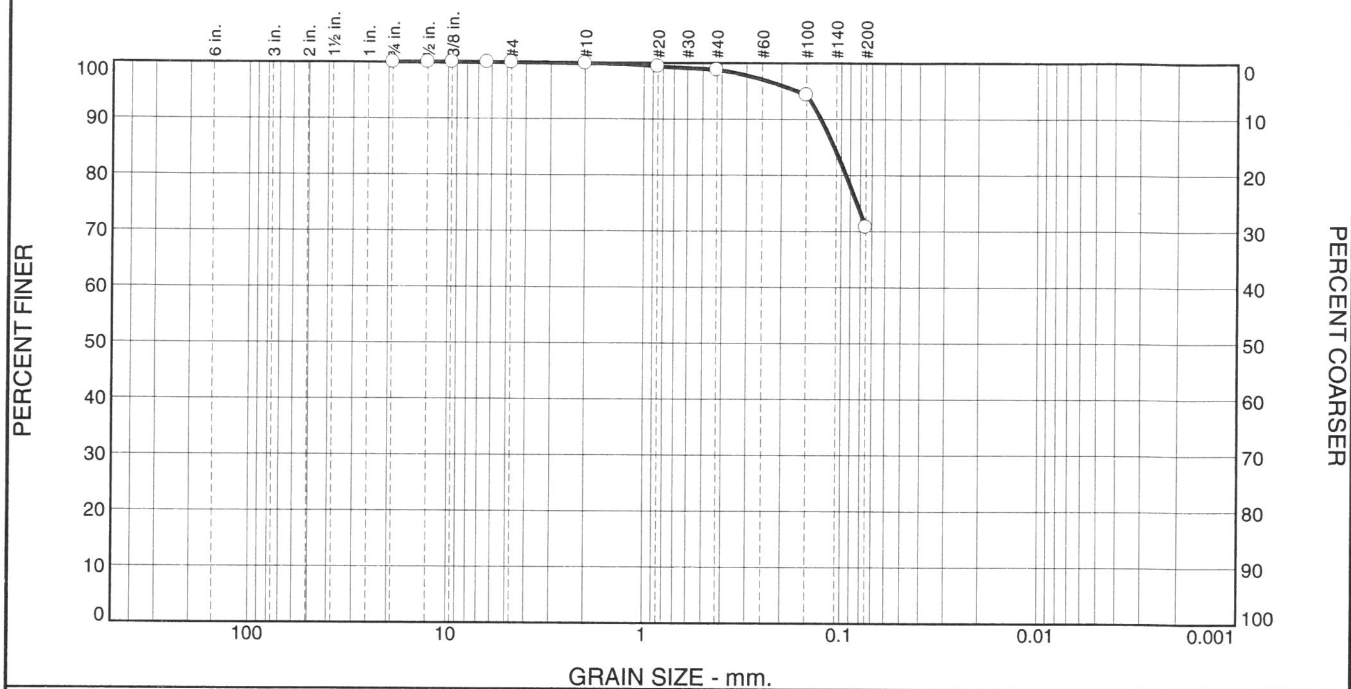
Remarks:

GEOPACIFIC ENGINEERING, INC.

Figure

Tested By: SJC

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.1	1.1	28.0	70.8	

TEST RESULTS

Opening Size	Percent Finer	Spec.* (Percent)	Pass? (X=Fail)
.75	100.0		
.5	100.0		
.375	100.0		
.25	100.0		
#4	100.0		
#10	99.9		
#20	99.4		
#40	98.8		
#100	94.3		
#200	70.8		

* (no specification provided)

Material Description

Silt with Sand

Atterberg Limits (ASTM D 4318)

PL= 28.2 **LL=** 37.0 **PI=** 8.8

Classification

USCS (D 2487)= ML **AASHTO (M 145)=** A-4(6)

Coefficients

D_{90}= 0.1267 D_{85}= 0.1085 D_{60}=
D_{50}= D_{30}= D_{15}=
D_{10}= C_u= C_c=

Remarks

Moisture 36.5%

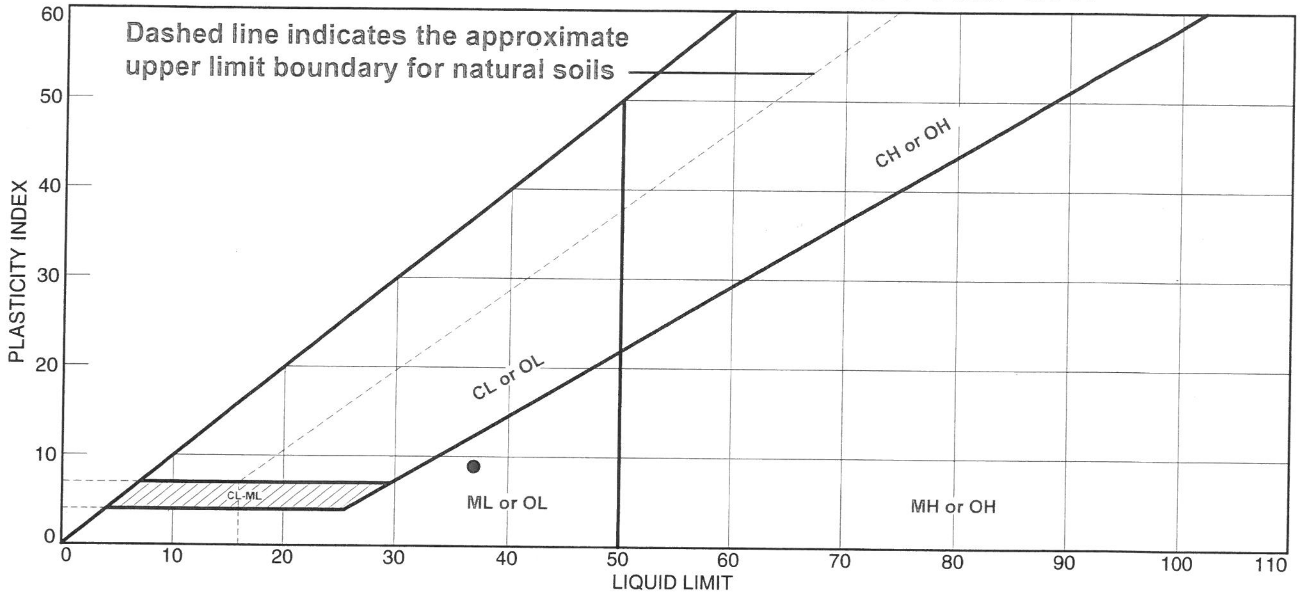
Date Received: **Date Tested:** 12/12/2016
Tested By: SJC
Checked By:
Title:

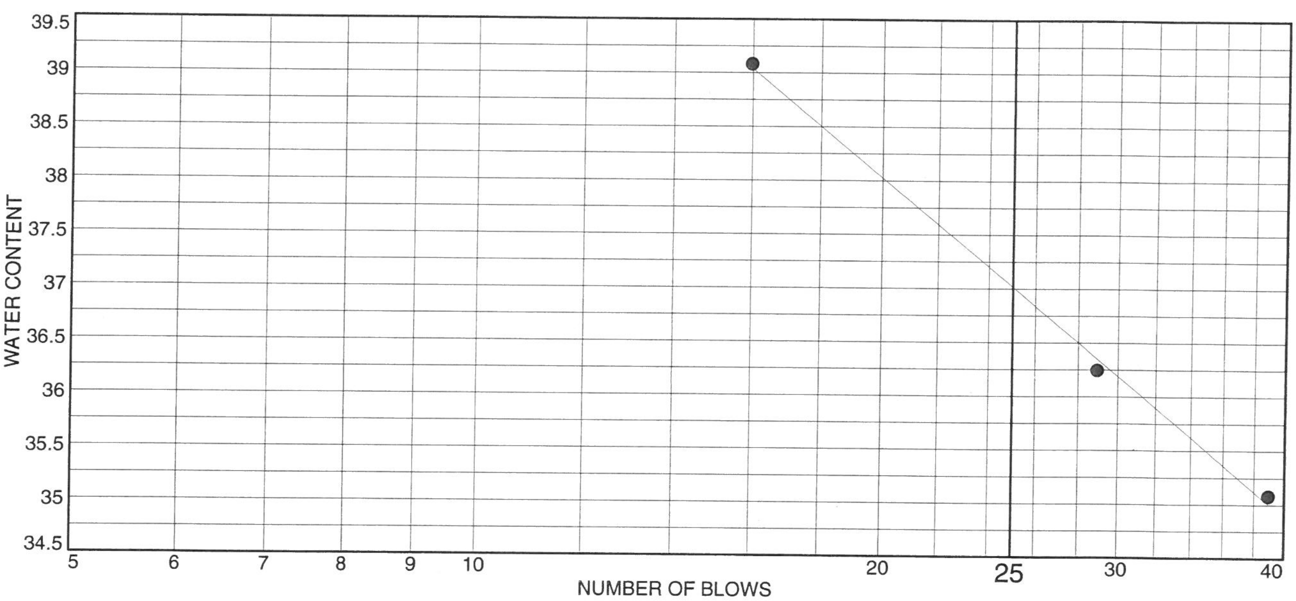
Location: TP-3 Sample 3.3
Sample Number: S16-355 **Depth:** 9' **Date Sampled:** 12/6/2016

GEOPACIFIC ENGINEERING, INC.

Client: Mr. Jeff Parker
Project: Old River Partition
Project No: 16-4408 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silt with Sand	37.0	28.2	8.8	98.8	70.8	ML

Project No. 16-4408 **Client:** Mr. Jeff Parker

Project: Old River Partition

Location: TP-3 Sample 3.3

Sample Number: S16-355 **Depth:** 9'

Remarks:

GEOPACIFIC ENGINEERING, INC.

Figure

Tested By: SJC

SOIL DESCRIPTION AND CLASSIFICATION GUIDELINES

Particle-Size Classification

COMPONENT	ASTM/USCS size range	ASTM/USCS sieve size range	AASHTO size range	AASHTO 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			

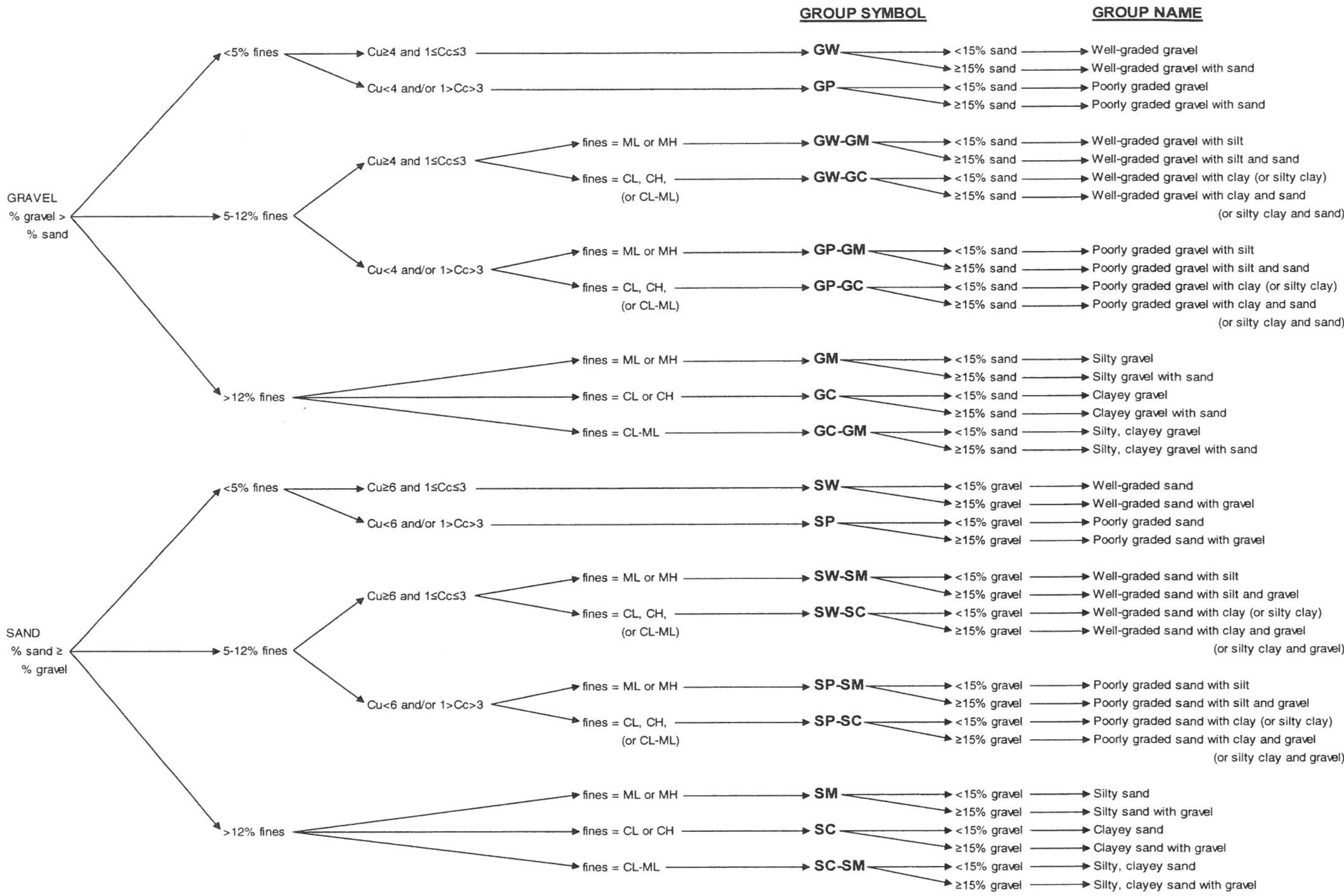
Note: The placing of A-3 before A-2 is necessary in the "left to right elimination process" and does not indicate superiority of A-3 over A-2.

TABLE 2. Classification of Soils and Soil-Aggregate Mixtures

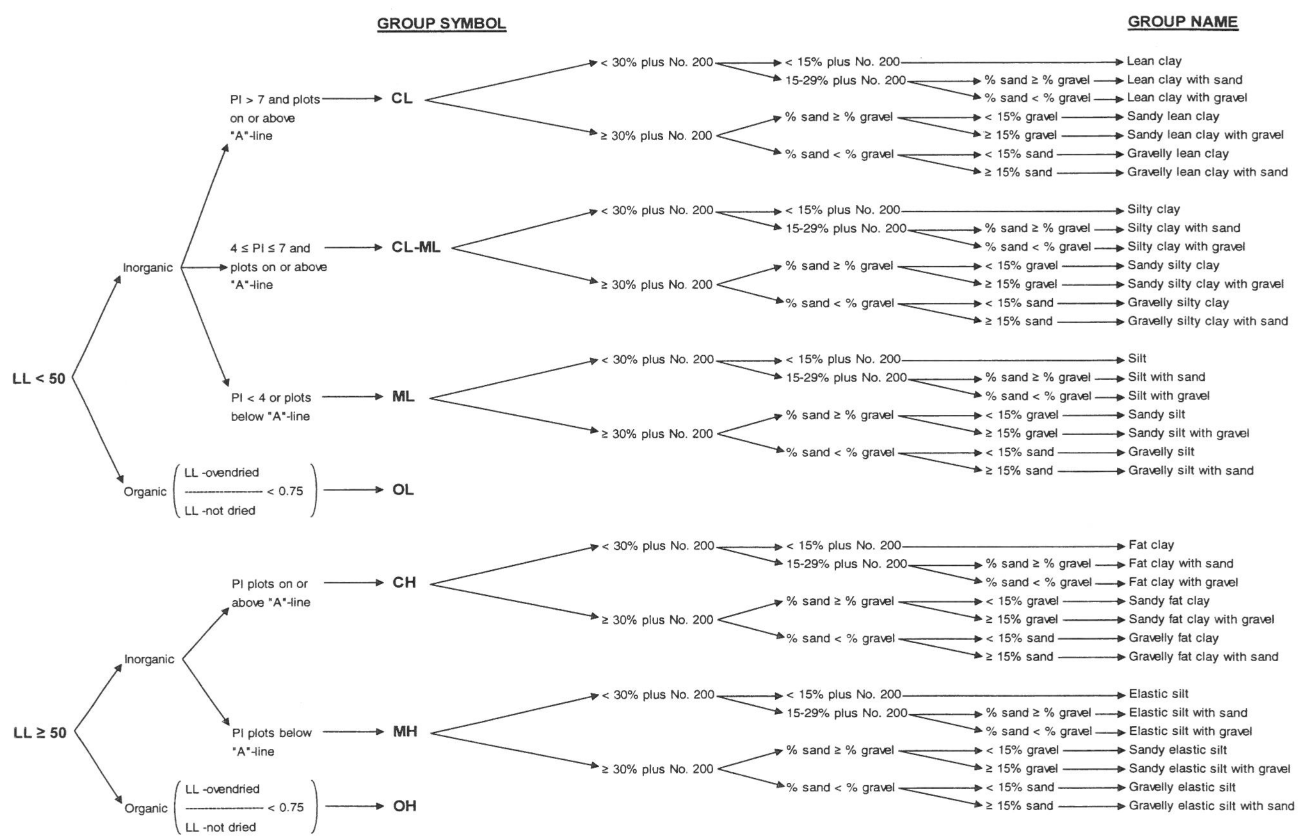
General Classification	Granular Materials (35 Percent or Less Passing 0.075 mm)							Silt-Clay Materials (More than 35 Percent Passing 0.075 mm)			
	A-1			A-2							A-7
Group Classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5, A-7-6
Sieve analysis, percent passing:											
2.00 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



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)

GeoPacific Engineering, Inc.
Real-World Geotechnical Solutions
Investigation • Design • Construction Support

SLOPE STABILITY CROSS-SECTIONS

14835 SW 72nd Avenue
Portland, Oregon 97224

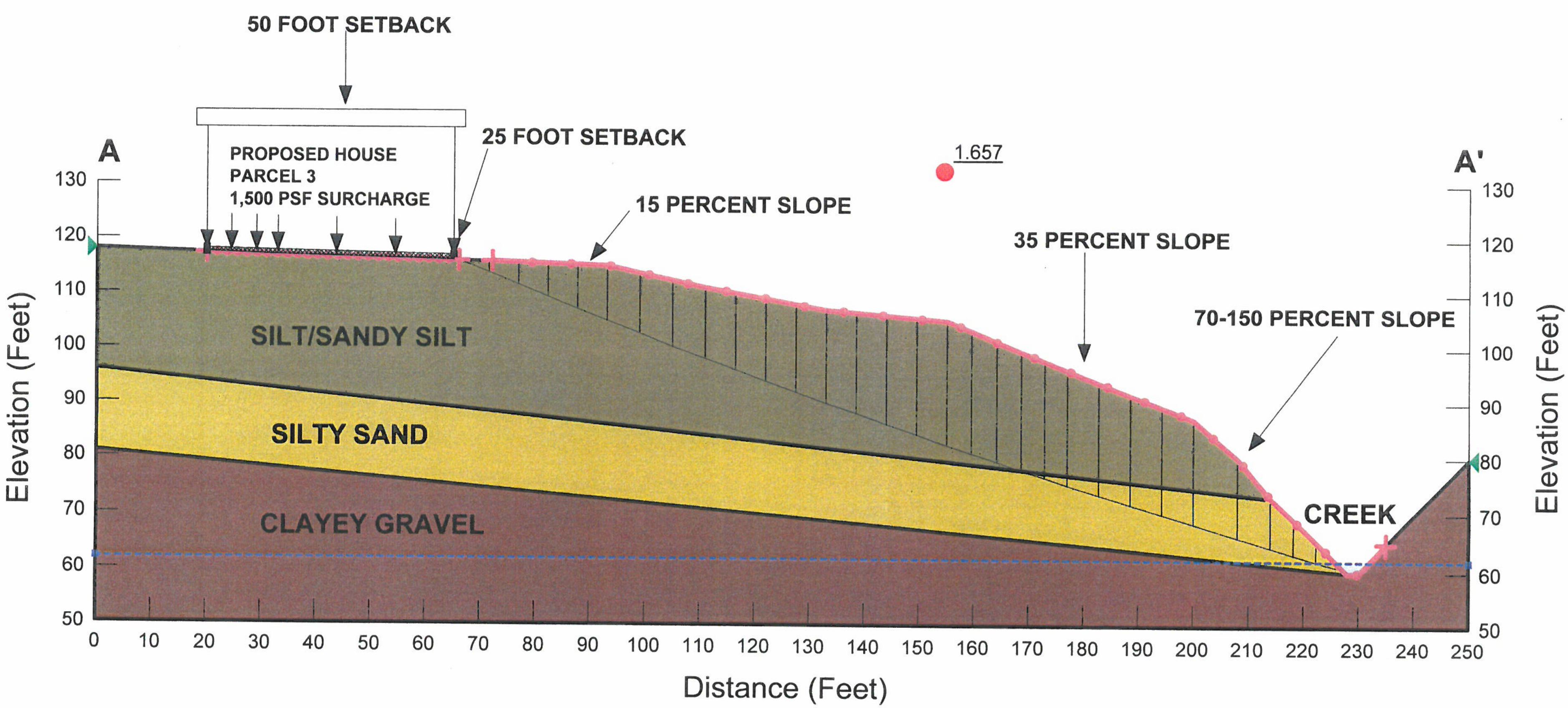
Tel (503) 598-8445
Fax (503) 941-9281

NORTHWEST

16-4408, OLD RIVER PARTITION
Geologic Cross-Section A to A'
12/29/2016 Scale = 1:1

Name: SILT Model: Mohr-Coulomb Unit Weight: 110 pcf Cohesion': 50 psf Phi': 28 ° Piezometric Line: 1
Name: Silty SAND Model: Mohr-Coulomb Unit Weight: 110 pcf Cohesion': 0 psf Phi': 28 ° Piezometric Line: 1
Name: Clayey GRAVEL Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion': 0 psf Phi': 40 ° Piezometric Line: 1

Static Factor of Safety: 1.657

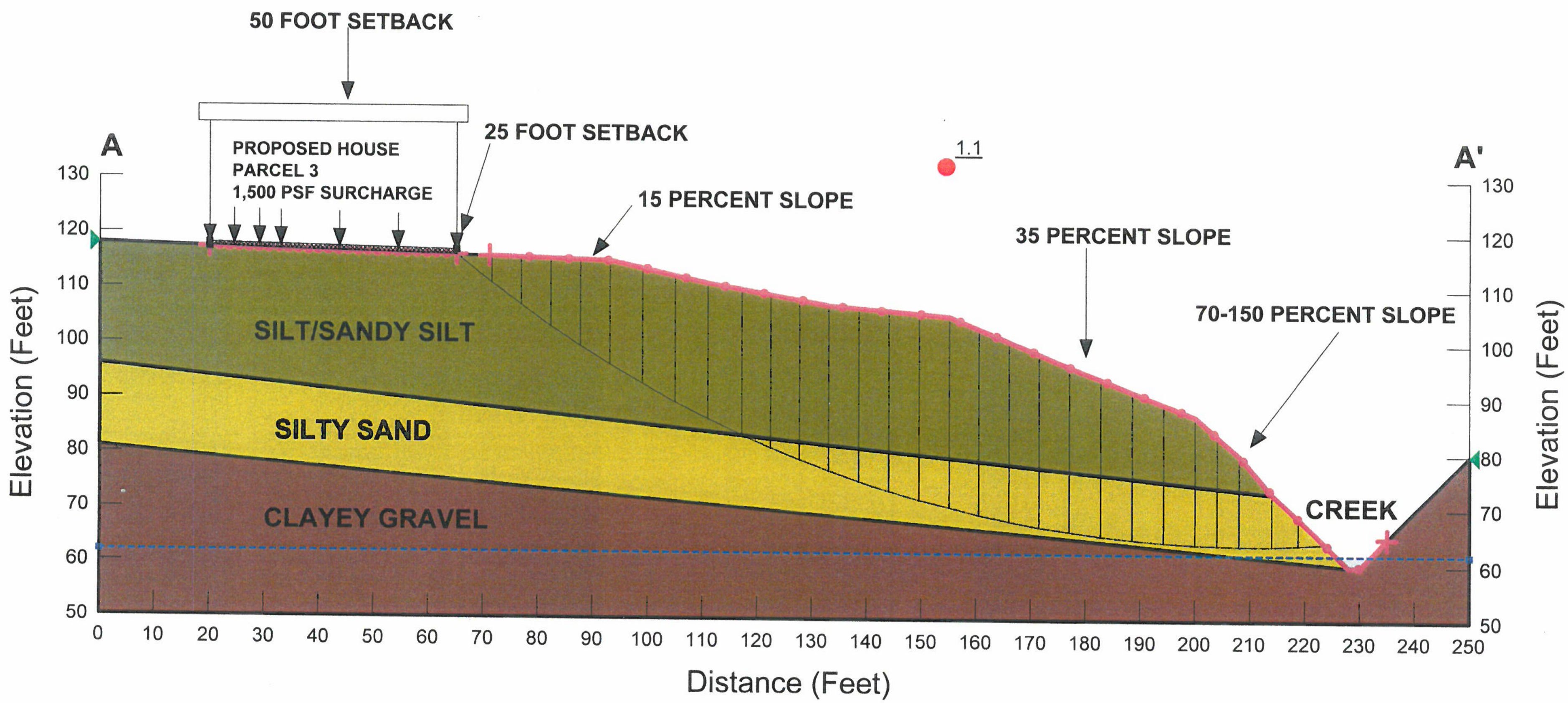


← NORTHWEST

16-4408, OLD RIVER PARTITION
Geologic Cross-Section A to A'
12/29/2016 Scale = 1:1

Name: SILT Model: Mohr-Coulomb Unit Weight: 110 pcf Cohesion': 50 psf Phi': 28 ° Piezometric Line: 1
Name: Silty SAND Model: Mohr-Coulomb Unit Weight: 110 pcf Cohesion': 0 psf Phi': 28 ° Piezometric Line: 1
Name: Clayey GRAVEL Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion': 0 psf Phi': 40 ° Piezometric Line: 1

Pseudostatic Factor of Safety: 1.1
Peak Ground Acceleration (PGA) = 0.452g
1/2 PGA = 0.22



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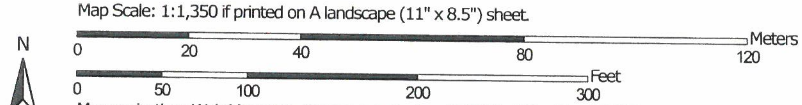
Investigation • Design • Construction Support

SITE RESEARCH

Map Scale: 1:1,350 if printed on A landscape (11" x 8.5") sheet.



Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 10N WGS84

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Map Unit Polygons

Soil Map Unit Lines

Soil Map Unit Points

Special Point Features

Blowout

Borrow Pit

Clay Spot

Closed Depression

Gravel Pit

Gravelly Spot

Landfill

Lava Flow

Marsh or swamp

Mine or Quarry

Miscellaneous Water

Perennial Water

Rock Outcrop

Saline Spot

Sandy Spot

Severely Eroded Spot

Sinkhole

Slide or Slip

Sodic Spot

Spoil Area

Stony Spot

Very Stony Spot

Wet Spot

Other

Special Line Features

Water Features

Streams and Canals

Transportation

Rails

Interstate Highways

US Routes

Major Roads

Local Roads

Background

Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:20,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Clackamas County Area, Oregon
Survey Area Data: Version 11, Sep 16, 2016

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Jul 26, 2014—Sep 5, 2014

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

USGS Design Maps Summary Report

User–Specified Input

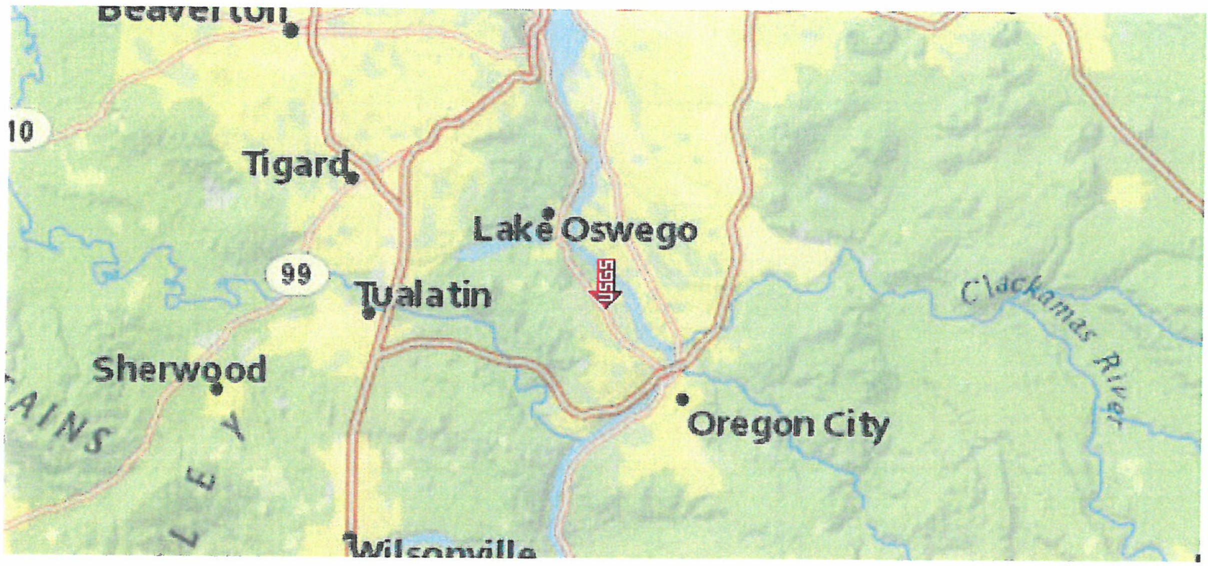
Report Title 16-4408, Old River Partition
Tue December 27, 2016 23:03:11 UTC

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 45.39494°N, 122.63998°W

Site Soil Classification Site Class D – "Stiff Soil"

Risk Category I/II/III



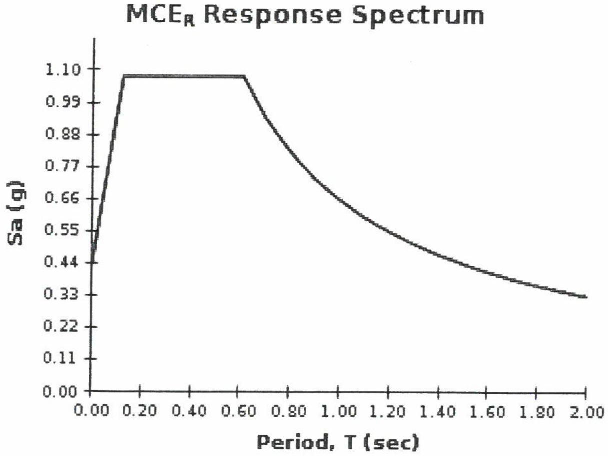
USGS–Provided Output

$S_S =$ 0.969 g　　$S_{MS} =$ 1.078 g　　$S_{DS} =$ 0.718 g

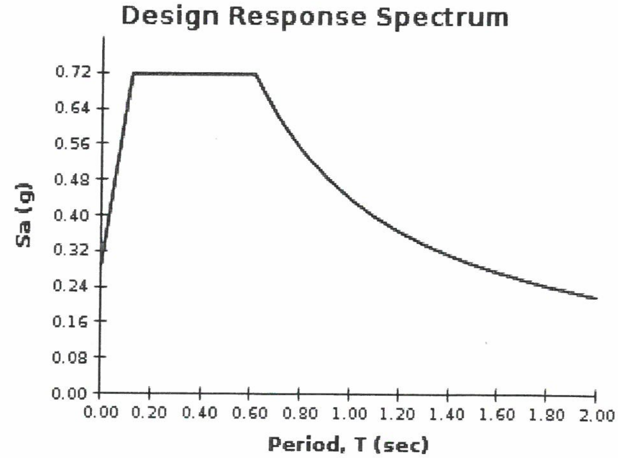
$S_1 =$ 0.415 g　　$S_{M1} =$ 0.657 g　　$S_{D1} =$ 0.438 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE_R Response Spectrum



Design Response Spectrum



For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (45.39494°N, 122.63998°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1]	$S_S = 0.969$ g
From Figure 22-2 [2]	$S_1 = 0.415$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ **or** $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4-1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = D and S_S = 0.969 g, F_a = 1.113

Table 11.4-2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.415 g, F_v = 1.585

Equation (11.4-1): $S_{MS} = F_aS_S = 1.113 \times 0.969 = 1.078$ g

Equation (11.4-2): $S_{M1} = F_vS_1 = 1.585 \times 0.415 = 0.657$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

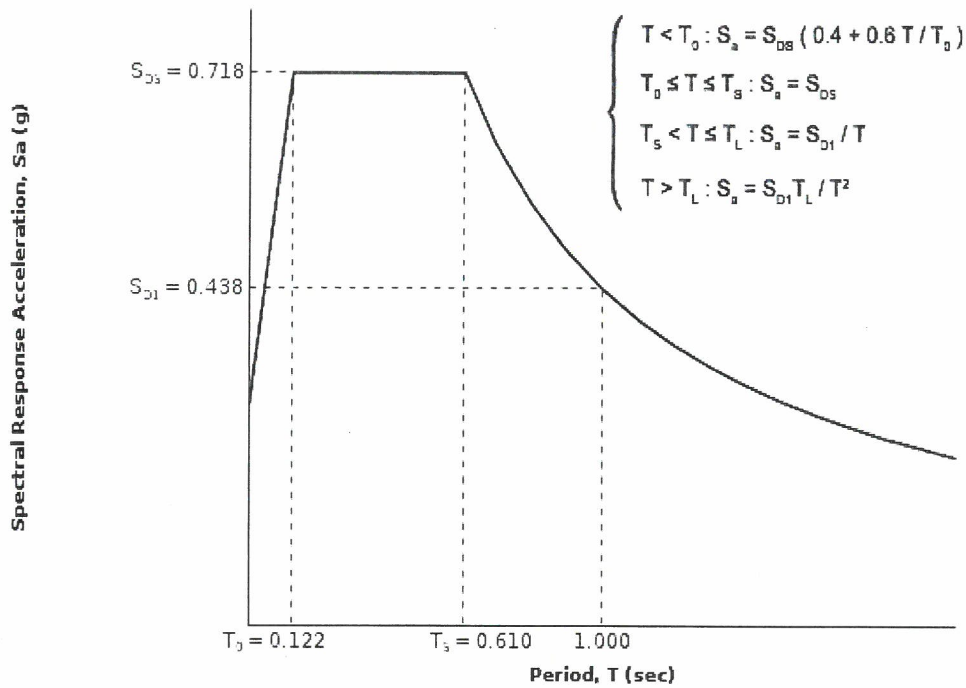
Equation (11.4-3): $S_{DS} = ⅔\ S_{MS} = ⅔ \times 1.078 = 0.718$ g

Equation (11.4-4): $S_{D1} = ⅔\ S_{M1} = ⅔ \times 0.657 = 0.438$ g

Section 11.4.5 — Design Response Spectrum

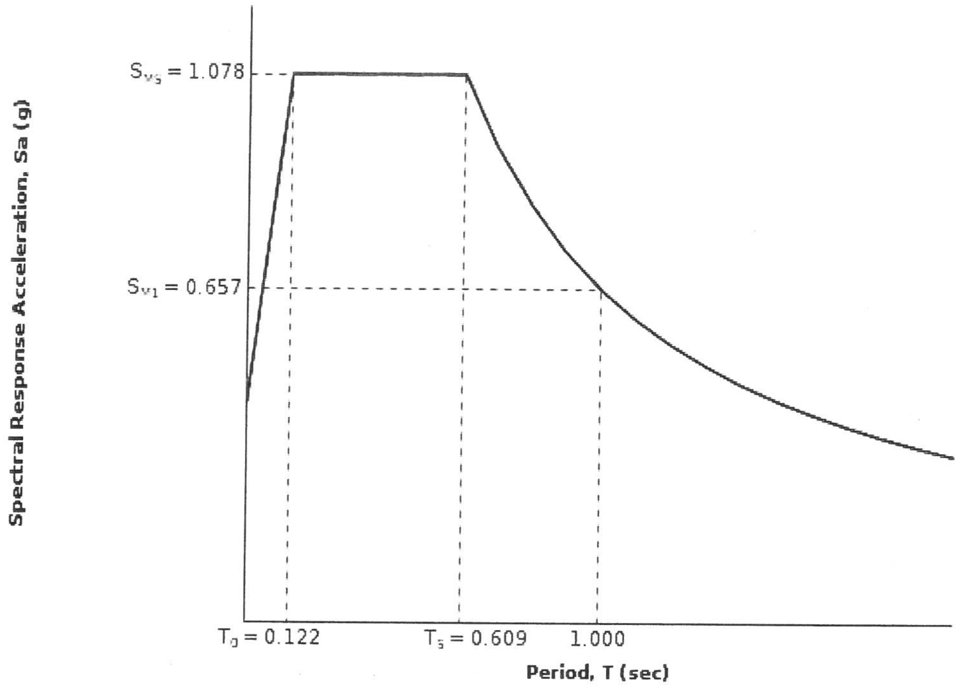
From Figure 22-12 [3] T_L = 16 seconds

Figure 11.4-1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.418

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.082 \times 0.418 = 0.452$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.418 g, F_{PGA} = 1.082

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 0.907

From Figure 22-18 [6] C_{R1} = 0.874

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.718 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.438 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

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Facing East

Facing West

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Facing South, Looking into Sloping Portion of Site

Sloping Portion of Site, 10 to 15 % Grade

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Facing Northeast, Break in Slope, 20 to 35 percent

Facing Southwest, Steep Drop Off to Creek Below

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Test Pit TP-1

Test Pit TP-1

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Test Pit TP-2

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Test Pit TP-4