GeoPacific Engineering, Inc.

Real-World Geotechnical Solutions
Investigation • Design • Construction Support

Preliminary Geotechnical Engineering Report

Fairfield Marriott

901 N Brutscher Street
Newberg, Oregon 97132
Yamhill County Tax Lot 1900, Tax Map 32W16

GeoPacific Engineering, Inc. Job No. 19-5391
January 3, 2019

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SUBJECT: PRELIMINARY GEOTECHNICAL ENGINEERING REPORT
FAIRFIELD MARRIOTT
901 N BRUTSCHER STREET
NEWBERG, OREGON 97132
YAMHILL COUNTY TAX LOT 1900, TAX MAP 32W16

1.0 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 Proposal No. P-7174, dated December 3, 2019, and your subsequent authorization of our proposal and *General Conditions for Geotechnical Services*.

Site Location:	901 N Brutscher Street Newberg, Oregon 97132 Yamhill County Tax Lot 1900, Tax Map 32W16 (see Figures 1 and 2)
Developer:	Hawkins Companies, LLC 850 Broad Street, Suite 300 Boise, ID 83709
Civil Engineer:	AKS Engineering 12965 SW Herman Rd, Ste 100 Tualatin, Oregon 97062
Jurisdictional Agency:	City of Newberg, Oregon
Geotechnical Engineer:	GeoPacific Engineering, Inc 14835 SW 72nd Avenue Portland, Oregon 97224 Phone: (503) 598-8445 Fax: (503) 941-9281

2.0 SITE AND PROJECT DESCRIPTION

As indicated on Figures 1 and 2, the subject site consists of Yamhill County Tax Lot 1900 on Tax Map 32W16, located across the street to the east from 901 N Brutscher Street in Newberg, Oregon. The property is rectangular in shape and totals approximately 1.6-acres in size. The site latitude and longitude are 45.306950, -122.940748, and the legal description is the SW ¼ of Section 16, T3S, R2W, Willamette Meridian. The site is bordered by Brutscher Street and an existing parking lot to the west, by the Newberg Veterinary Hospital to the north, by the Newberg Ford dealership to the east, and by the Argyle Winery distribution to the south. The site is undeveloped and is currently vegetated with grasses and sparse weeds. Historically the site was farmed. Topography at the site is relatively flat to gently sloping to the east with site elevations ranging from approximately 225 to 228 feet above mean sea level (amsl).

Site planning is currently preliminary. GeoPacific has not reviewed a grading plan or foundation plans. Based upon communication with the client and the structural engineer, and review of a conceptual site plan, GeoPacific understands that a four-story Fairfield Marriott hotel will be built in the central portion of the site. We anticipate that the building will consist of a wood-framed structure supported on a typical spread footing foundation including square column footings, and continuous perimeter footings. Based on communication with the structural engineer we expect maximum structural loading on column and continuous strip footings on the order of 10 to 80 kips; and 2 to 8 kips respectively; and a maximum applied bearing pressure on the order of 2,500 psf. We understand that development of the site will also include construction of flexible and rigid paved areas, and installation of associated new underground utilities. Based upon existing site grades we anticipate that grading will include cuts and fills of five feet or less.

3.0 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.

The *Generalized Geologic Map of the Willamette Lowland, Marshall W. Gannett and Rodney R. Caldwell, (U.S. Department of the Interior, U.S. Geological Survey, 1998)*, indicates that the site is underlain by Pleistocene-aged (approximately 2.6 million to 11,000 years ago) silt, sand, and gravel deposited primarily by late Pleistocene glacial outburst flooding commonly referred to as the Missoula Flood Events, but also including glaciofluvial sediments derived from weathering of the Cascade Range located to the east, and the Chehalem Mountains to the north (Qs).

The *Web Soil Survey (United States Department of Agriculture, Natural Resource Conservation Service (USDA NRCS 2019 Website)*, indicates that near-surface soils consist of the Woodburn silt load soil series. Woodburn series soils generally consist of very deep, moderately well drained soils that formed in silty, stratified, glaciolacustrine deposits. The Web Soil Survey soil map for the subject site is presented as an attachment to this report.

4.0 REGIONAL SEISMIC SETTING

At least three 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 Gales Creek-Newberg-Mt. Angel Structural Zone, and the Cascadia Subduction Zone.

4.1 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 18 miles northeast of the site. The Oatfield Fault occurs along the western side of the Portland Hills and is located approximately 16 miles northeast of the site. The East Bank Fault occurs along the eastern margin of the Willamette River, and is located approximately 21 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).

4.2 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 1.5 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.

4.3 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.

5.0 FIELD EXPLORATION AND SUBSURFACE CONDITIONS

Our subsurface explorations for this report were conducted on December 13, and December 16, 2019. A total of four exploratory test pits (TP-1 through TP-4) were excavated at the site using a Hitachi 40U rubber-tracked excavator subcontracted by GeoPacific to a maximum depth of approximately 10 feet bgs. In addition, one cone penetrometer test was conducted at the site using Oregon Geotechnical Explorations, Inc., truck-mounted electric cone penetrometer to a maximum depth of 60 feet bgs. Seismic shear wave velocity tests, and porewater pressure measurements were conducted during advancement of the cone.

Explorations were conducted under the full-time observation of a GeoPacific geologist. During the explorations pertinent information, including soil sample depths, stratigraphy, soil engineering characteristics, and groundwater occurrence was recorded. Soils were classified in accordance with the Unified Soil Classification System (USCS). Soil samples obtained from the explorations were placed in relatively air-tight plastic bags. At the completion of each test, the test pits were loosely backfilled with onsite soils. The approximate locations of the explorations are indicated on Figures 2 and 3. It should be noted that 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. Summary exploration logs are attached. The stratigraphic contacts shown on the individual test pit logs represent the approximate boundaries between soil types. The actual transitions may be more gradual. The soil and groundwater conditions depicted are only for the specific dates and locations reported, and therefore, are not necessarily representative of other locations and times. Soil and groundwater conditions encountered in the explorations are summarized below.

5.1 Soil Descriptions

Topsoil/Remnant Till Zone: At the locations of our test pits the topsoil horizon was typically observed to consist of grass covered, soft to medium stiff, very moist, moderately organic Lean CLAY (OL-CL), containing fine roots, and extending to depths ranging from 4 to 8 inches. It appears that due to historic plowing and tilling of the site during agriculture use, a remnant farm till zone is present across much of the site with disturbed soil conditions observed extending up to 18 inches bgs.

Undocumented Fill: At the location of test pit TP-3 we encountered undocumented fill soils which appeared to have been historically placed to level the site, or during development of adjacent parcels. The fill soils extended to a depth of approximately 36 inches at the test pit location, and based upon visual observation of the surrounding area, appeared to extend to the approximate limits indicated on Figure 2. The fill soils were observed to consist of relatively clean, light brown, soft, moist, Lean CLAY (CL), containing very sparse plastic debris fragments. It appeared that the soil type will be suitable for re-use as engineered fill provided it is thoroughly removed and replaced as described below in Section 6.2, *Engineered Fill*.

Lean CLAY: Underlying the topsoil within our subsurface explorations, soils were observed to consist of brown, medium stiff to very stiff, very moist, low to moderately plastic, Lean CLAY (CL), displaying pinhole structure. The soil type was observed to extend to depths ranging from approximately 6 to 7 feet bgs. Soils laboratory testing conducted on representative samples collected from test pit TP-1 indicated that the soil type classified as Lean CLAY (CL) according to the USCS soil classification system, and as A-7-6(25), and A-7-6(26) according to AASHTO standards. Sieve analysis indicated 97 percent by weight passing the U.S. No. 200 sieve, and moisture content of 21 to 25 percent. Atterberg Limit testing indicated a liquid limit of 47 to 49, and a plasticity index of 22 to 23. Pocket penetrometer measurements conducted within the upper four feet of the ground surface ranged from approximately 2.5 to 4.0 tons/ft^2. CPT tip resistances ranged from 11 to 40 tons/ft^2.

SILT: Underlying the Lean CLAY within our subsurface explorations, soils were observed to consist of brown, stiff, moist to wet, low-plasticity, SILT (ML), displaying pinhole structure. The soil type was observed to extend to the maximum depth of exploration within our test pits (10 feet) and was inferred to extend up to approximately 18 feet bgs within the CPT exploration. Soils laboratory testing conducted on a representative sample collected from test pit TP-1 indicated that the soil type classified as SILT (ML) according to the USCS soil classification system, and as A-6(16) according to AASHTO standards. Sieve analysis indicated 99 percent by weight passing the U.S. No. 200 sieve, and moisture content of 27 percent. Atterberg Limit testing indicated a liquid limit of 40, and a plasticity index of 14. CPT tip resistances ranged from 13 to 44 tons/ft^2.

Sandy SILT/Clayey SILT: CPT exploration data inferred that below the SILT soil type at an approximate depth of 18 feet bgs, soils become sandy and ranged from interlayered Sandy SILT to Clayey SILT to the maximum depth of exploration (approximately 60 feet bgs). CPT tip resistances ranged from 29 to 136 tons/ft^2, averaging in a range of 40 to 50 tons/ft^2.

5.2 Shrink-Swell Potential

Medium stiff to very stiff, fine-grained soils were encountered in the upper 10 feet of the site. Atterberg Limit testing indicated the soil types displayed plasticity indexes ranging from 14 to 23. Based upon the results of our soils laboratory testing and our local experience with the soil layers in the vicinity of the subject site, the plasticity of the soils is considered to be low to moderate, and the shrink-swell potential of the soil types is considered to be low. Special design measures are not considered necessary to minimize the risk of uncontrolled damage of foundations as a result of potential soil expansion at this site.

5.3 Groundwater and Soil Moisture

On December 13, and December 16, 2019, observed soil moisture conditions were generally very moist to wet. Groundwater seepage was observed within test pit TP-3 at an approximate depth of 9 feet bgs. Groundwater seepage was not observed in the other test pits. Porewater pressure measurements were conducted at depths of 30 and 50 feet bgs within the cone penetrometer exploration which indicated that groundwater is present at a depth of 3.43 feet bgs. Based on our observations of soil conditions within the test pits we believe that the porewater test measured pressures within confined soil layers and misinterpreted depth to groundwater at the site. Groundwater monitoring piezometers may be installed if the client wishes to monitor future seasonal fluctuations of the static groundwater table at the site. Based on our review of available well logs from the vicinity of the subject site we expect that static groundwater may be encountered at depths ranging from approximately 10 to 20 feet bgs, depending on ground surface elevation. Perched groundwater may be encountered in localized areas. Seeps and springs may exist in areas not explored and may become evident during site grading.

6.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Our site investigation indicates that the proposed construction appears to be 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 at the site are:

- The site contains a remnant agricultural till zone from past farming use that consists of soft to medium stiff, disturbed clayey soils extending to a depth of approximately 18 inches on average across the site.
- Up to 36 inches of undocumented fill soil was encountered within Test Pit TP-3 in the northeastern portion of the site which contained soft soil conditions. The fill material was relatively clean and is anticipated to be suitable for re-use as engineered fill.
- Static settlement calculations indicated that due to soft to medium stiff soil conditions encountered within the upper three feet of the ground surface, up to 2 inches of static settlement may be anticipated for assumed applied bearing pressures up to 2,500 psf. Recommendations are provided for mitigation of static settlement which include removal and replacement with compacted crushed aggregate underneath the footings.
- Soil liquefaction assessment conducted using the Robertson 2009 CPT-based method of analysis indicated the potential for up to approximately 4 inches of dynamic settlement during a peak Cascadia Subduction zone earthquake with a moment magnitude of 9.1, and a peak ground acceleration of 0.47g.

6.1 Site Preparation Recommendations

Areas proposed for construction and areas to receive fill should be cleared of vegetation, and any organic and inorganic debris. Inorganic debris and organic materials from clearing should be removed from the areas proposed for grading. Organic-rich soils and root zones should then be stripped from construction areas of the site or where engineered fill is to be placed. Depth of stripping of organic soils is estimated to be approximately 6 to 18 inches across the majority of the site. The heavy concentrated grass root mats associated with the topsoil layers were observed to have maximum depths of approximately 8 inches, however a disturbed agricultural till zone is present at the site due to past farming which extended to depths of approximately 18 inches bgs. Soft soil conditions should be expected within this disturbed zone, particularly during periods of wet weather. 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 areas proposed for placement of engineered fill. 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.

At the location of test pit TP-3 we encountered undocumented fill soils which appeared to have been historically placed to level the site, or during development of adjacent parcels. The fill soils extended to a depth of approximately 36 inches at the test pit location, and based upon visual observation of the surrounding area, appeared to extend to the approximate limits indicated on Figure 2. The fill soils were observed to consist of relatively clean, light brown, soft, moist, Lean CLAY (CL), containing very sparse plastic debris fragments. It appeared that the soil type will be suitable for re-use as engineered fill provided it is thoroughly removed and replaced as described below in Section 6.2, *Engineered Fill*. Based on our review of the conceptual site layout plan (Figure 3), it appears that the soil is located outside of the proposed hotel building envelope and will primarily affect construction of parking areas and drive lanes in the northeastern portion of the site.

If site development and grading are conducted during the dry summer months, we recommend that the agricultural till zone be recompacted as opposed to over-excavated and removed from the site. Following site stripping the existing ground surface may be scarified and recompacted prior to placement of structural fill or structures. The areas should be prepared by removing highly organic soil layers which contain abundant root concentration, or organic content in excess of approximately 4 to 5 percent by weight. The underlying soils then be ripped, and moisture conditioned to within two percent of optimum moisture content, and recompacted to project specifications for engineered fill as determined by the Standard Proctor (ASTM D698).

6.2 Engineered Fill

At this time site planning is preliminary and GeoPacific has not reviewed a grading plan. Based on existing site gradients and communication with the client we anticipate that engineered cuts and fills will be conducted on the order of 5 feet or less. Where incorporated into the project, 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. Site grading should be conducted in accordance with the requirements outlined in the 2018 International Building Code (IBC), and 2019 Oregon Structural Specialty Code (OSSC), Chapter 18 and Appendix J. Areas proposed for fill placement should be prepared as described in Section 6.1, *Site Preparation Recommendations*. Surface soils should then be scarified and recompacted prior to placement of structural fill. Site preparation, soil stripping, and grading activities should be observed and documented by a geotechnical engineer or his representative. Proper test frequency and earthwork documentation usually requires daily observation and testing during stripping, rough grading, and placement of engineered fill.

Onsite native soils appear to be suitable for use as engineered fill. Soils containing greater than 5 percent organic content should not be used as structural 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 12 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. Soils should be moisture conditioned to within two percent of optimum moisture. 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 may be impacted by shallow groundwater, soil moisture and wet weather conditions. Earthwork in wet weather would likely require extensive use of additional crushed aggregate, cement or lime treatment, or other special measures, at considerable additional cost compared to earthwork performed under dry-weather conditions.

6.3 Excavating Conditions and Utility Trench Backfill

We anticipate that onsite soils can generally be excavated using conventional heavy equipment. Bedrock was not encountered within our subsurface explorations which extended to a maximum depth of 60 feet bgs. 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 native soils classify as Type B Soil and temporary excavation side slope inclinations as steep as 1H:1V may be assumed for planning purposes. These cut slope inclinations are applicable to excavations above the water table only.

Shallow, perched groundwater may be encountered at the site 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.

Underground utility pipes should be installed in accordance with the procedures specified in ASTM D2321 and City of Newberg standards. We recommend that structural trench backfill be compacted to at least 95 percent of the maximum dry density obtained by the Standard Proctor (ASTM D698) 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 100-lineal-foot section of trench.

6.4 Erosion Control Considerations

During our field exploration program, we did not observe soil conditions which are 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.

6.5 Wet Weather Earthwork

Soils underlying the site are likely to be moisture sensitive and will 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 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.

6.6 Spread Foundations and Static Settlement Analysis

Based upon communication with the client and review of a conceptual site plan (Figure 3), GeoPacific understands that the proposed development at the site will consist of constructing a Marriott Hotel in the central portion of the site. The building will be a four-story, wood-framed structure, supported by a typical spread footing foundation including square column footings, and continuous perimeter footings. Based on communication with the structural engineer we expect maximum structural loading on column and continuous strip footings on the order of 10 to 80 kips, and 2 to 8 kips respectively, and maximum applied bearing pressures on the order of 2,500 psf. Square column footings may range in size from 4’x’4 to 6’x6’ and may be embedded 12 to 18 inches below existing ground surface.

Based upon soil conditions encountered within our subsurface explorations at this site, the anticipated allowable soil bearing pressure for in situ soil conditions is 1,500 lbs/ft^2. Heavier loads may result in static settlement of the structure beyond tolerable limits without additional ground improvement. As stated above, we understand that up to 2,500 psf allowable bearing pressure is needed at this site for design of the proposed four-story structure. We conducted a static settlement analysis for the soil profile encountered within cone penetrometer test CPT-1 based upon the structural loading information provided for the proposed structure using the Modified Schmertmann’s Method (1978) to calculate vertical displacement. Calculations were conducted using the Soil Structure Settlement Analysis v2.0.2 software. Calculations for long-term static settlement are based upon our understanding of proposed structural building loads, which will increase the vertical effective stress in subsurface soils and may potentially induce soil settlement. Due to natural variations in soil conditions across the site the calculated settlement values below should be considered to be estimates. Actual induced settlement during construction may vary greatly over short distances.

Our static settlement calculations indicated potential static settlement totals of up to 2 inches for existing conditions over a period of approximately 4 to 5 years. We typically understand anticipated settlements greater than 1 inch to be beyond tolerable limits of similar structures. We assessed the soil profile to determine if static settlement estimates could be reduced to less than one inch with over-excavation and re-placement of in situ soils with two feet of compacted crushed aggregate beneath the footings.

Based upon the results of our calculations, it appears that anticipated static settlement totals relative to a maximum applied bearing pressure of 2,500 psf can be reduced to 1 inch or less by constructing footings on crushed aggregate mats consisting of a minimum of 24-inches of 1.5”-0 crushed aggregate, compacted to at least 95 percent of the maximum dry density determined by ASTM D1557 (Modified Proctor) or equivalent. The crushed aggregate mats should extend at least 24 inches beyond the edges of the footings on all sides, and should be underlain by woven geotextile fabric consisting of Mirafi 500X or equivalent.

The recommended maximum allowable bearing pressure may be increased by 1/3 for short-term transient conditions such as wind and seismic loading. For applied bearing pressures in excess of 2,500 psf the geotechnical engineer should be consulted. If heavier loads than described above are proposed, it may be feasible to increase the thickness of the crushed aggregate mats, or to rammed aggregate piers (GeoPiers) may be considered. 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. Assuming the crushed aggregate pads are constructed as described, our static settlement calculations indicate that the maximum anticipated total and differential footing movements (generally from soil expansion and/or settlement) are in the range of 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.

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 12 inches below exterior grade. 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, additional crushed aggregate.

Our recommendations are for commercial construction incorporating conventional shallow spread and continuous footing foundations.

6.7 Concrete Slabs-on-Grade

Preparation of areas beneath concrete slab-on-grade floors should be performed as described in Section 6.1, *Site Preparation Recommendations* and Section 6.6, *Spread Foundations*. 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 dense, fine to coarse-grained soils anticipated to be present at foundation subgrade elevation following adequate site preparation as described above. This value assumes the concrete slab system is designed and constructed as recommended herein, with a minimum thickness of 8 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.

6.8 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 exposed ground in the crawlspace, and crawlspace ventilation (foundation vents). The client should be informed and educated that some slow flowing water in the crawlspaces is considered normal and not necessarily detrimental to the structures given these other design elements incorporated into construction. Appropriate design professionals should be consulted 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.

Perimeter footing drains may be eliminated at the discretion of the geotechnical engineer 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 deemed 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 4 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.

6.9 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 a passive earth pressure of 320 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 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.

Water collected from the wall 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.

6.10 Flexible Pavement Design: Private Parking and Drive Areas

As indicated on Figure 3, we understand that development at the site will include construction of private asphaltic concrete private parking and drive areas. For the flexible pavement section, we conservatively assume that the subgrade will exhibit a resilient modulus of at least 6,000, which correlates to a CBR value of 4. Based upon our understanding of the anticipated traffic which includes light-duty passenger vehicles, weekly trash pickups, and occasional fire trucks weighing up to 75,000 lbs, we calculated an anticipated 18-kip ESAL count of approximately 75,000 over 20 years. Table 1 presents our flexible pavement design input parameters. Table 2 presents our recommended minimum dry-weather pavement section for the proposed pavement section, supporting 20 years of vehicle traffic. Pavement design calculations are attached to this report.

Table 1: Flexible Pavement Section Design Input Parameters for Private Parking and Drive Areas

Input Parameter	Design Value
18-kip ESAL Initial Performance Period (20 Years)	75,000
Initial Serviceability	4.2
Terminal Serviceability	2.2
Reliability Level	85 Percent
Overall Standard Deviation	0.5
Roadbed Soil Resilient Modulus (PSI)	6,000
Structural Number	**2.38**

Table 2: Recommended Minimum Dry-Weather Pavement Section: Private Parking and Drive Areas

Material Layer	Section Thickness (in.)	Structural Coefficient	Compaction Standard
Asphaltic Concrete (AC)	3.5 in.	.42	91%/ 92% of Rice Density AASHTO T-209
Crushed Aggregate Base ¾”-0 (leveling course)	2 in.	.10	95% of Modified Proctor AASHTO T-180
Crushed Aggregate Base 1½”-0	8 in.	.10	95% of Modified Proctor AASHTO T-180
Subgrade	12 in.	6,000 PSI	95% of Standard Proctor AASHTO T-99 or equivalent
Total Calculated Structural Number		**2.47**	

6.11 Rigid Pavement Design: Private Drive Lanes

The private drive lanes the site may be constructed with Portland cement concrete (PCC) pavement in some areas. We assume that the proposed private drive lanes will be subjected to vehicle traffic primarily consisting of passenger vehicles, weekly trash trucks, occasional fire trucks weighing up to 75,000 lbs, and wheel loads up to HS-20 loading (up to three axles, maximum 32,000 lbs per axle). For the new private rigid pavement section, we conservatively assume that the subgrade will exhibit a resilient modulus of at least 6,000, which correlates to a CBR value of 4. Based upon the anticipated traffic, we calculated an anticipated 18-kip ESAL count of approximately 75,000 over 20 years.

Under these assumptions and based upon our calculations, our recommended pavement design for private rigid pavement areas consists of a steel reinforced PCC slab with a thickness of 6 inches, and a 4,000 psi minimum compressive strength concrete, placed over 8 inches of crushed aggregate compacted to a minimum of 95% relative to ASTM D1557. A single mat of No.4 reinforcing bars should be placed centrally, with a minimum spacing of 12-inches each way. The steel reinforcing should be placed to maintain at least 3 inches clearance from bottom, and 3 inches of clearance from the edges. Lap lengths should be a minimum of 40 bar diameters (db), or 20 inches. A minimum joint spacing of 10 feet should be maintained for the PCC concrete.

Tolerances of spacing, ties, and clearances, should be constructed in accordance with ACI 318, and the requirements of Chapter 19 of the 2015 IBC. Table 3 presents our rigid pavement design input parameters. Table 4 presents the recommended minimum section for the proposed rigid pavement. Pavement design calculations are attached to this report.

Table 3: Design Input Parameters: Rigid Pavement Areas-Private Drive Lanes

Input Parameter	Design Value
18-kip ESAL Initial Performance Period (20 Years)	75,000
Initial Serviceability	4.2
Terminal Serviceability	2.2
28-Day Flexural Strength (PSI)	650
28-Day Mean Elastic Modulus of Elasticity of Concrete (PSI)	3,500,000
Mean Effective K-Value (PSI)	33.03
Reliability Level	85 Percent
Overall Standard Deviation	0.39
Load Transfer Coefficient	3
Overall Drainage Coefficient	1
Roadbed Soil Resilient Modulus (PSI) Concrete Road 6 inches thick	6,000
Rigid Transverse Joint Spacing	10 Feet

Table 4: Rigid Pavement Section: Rigid Pavement Areas-Private Drive Lanes

Material Layer	Section Thickness	Standard
Portland Cement Concrete Pavement 4,000 psi (PCC)	6 inches	• Use class 4000-3/4 paving concrete • Concrete should be sampled and tested per the requirements of ACI 318 • 4,000 psi compressive strength at 28 days • Maximum air content 4 percent • Maximum slump 6 inches • Maximum lateral joint spacing = 10 feet • Reinforcing Steel: Single Mat No. 4 Longitudinal Bars, Spaced 12 inches each way, minimum 3-inch clearance on bottom and sides.
Crushed Aggregate Base (¾”-0 leveling underlain by 1.5”-0)	2 inches ¾”-0 6 inches 1.5”-0	• Use ¾”-0, or 1”-0 dense graded base aggregate meeting the requirements of ODOT 00641. Thickness may need to be increased to 12 inches or more for constructability in areas of soft or wet subgrade. Geotextile fabric consisting of Mirafi 500X to be utilized during wet weather. • 95% of Modified Proctor ASTM D1557
Competent Subgrade	12 inches	• Recompacted or cement treated • Visual inspection (Proofroll)

6.12 Roadway Subgrade Preparation

Roadway subgrade soils should be compacted and inspected by GeoPacific prior to the placement of crushed aggregate base for pavement. Typically, a proofroll with a fully loaded water or haul truck is conducted by travelling slowly across the grade and observing the subgrade for rutting, deflection, or movement. Any pockets of organic debris or loose fill encountered during ripping or tilling should be removed and replaced with engineered fill (see Section 6.1, *Site Preparation Recommendations*). In order to verify subgrade strength, we recommend proof-rolling directly on subgrade with a loaded dump truck during dry weather and on top of base course in wet weather. Soft areas that pump, rut, or weave should be stabilized prior to paving.

If pavement areas are to be constructed during wet weather, the subgrade and construction plan should be reviewed by the project geotechnical engineer at the time of construction so that condition specific recommendations can be provided. The moisture sensitive subgrade soils make the site a difficult wet weather construction project. General recommendations for wet weather pavement sections are provided below.

During placement of pavement section materials, density testing should be performed to verify compliance with project specifications. Generally, one subgrade, one base course, and one asphalt compaction test is performed for every 100 to 200 linear feet of paving.

6.13 Wet Weather Construction Pavement Section

This section presents our recommendations for wet weather pavement sections and construction for new pavement sections at the project. These wet weather pavement section recommendations are intended for use in situations where it is not feasible to compact the subgrade soils to project requirements, due to wet subgrade soil conditions, and/or construction during wet weather. Based on our site review, we recommend a wet weather section with a minimum subgrade deepening of 6 to 12 inches to accommodate a working subbase of additional 1½"-0 crushed rock. Geotextile fabric, Mirafi 500x or equivalent, should be placed on subgrade soils prior to placement of base rock.

In some instances, it may be preferable to use a subbase material in combination with over-excavation and increasing the thickness of the rock section. GeoPacific should be consulted for additional recommendations regarding use of additional subbase in wet weather pavement sections if it is desired to pursue this alternative. Cement treatment of the subgrade may also be considered instead of over-excavation. For planning purposes, we anticipate that treatment of the onsite soils would involve mixing cement powder to approximately 6 percent cement content and a mixing depth on the order of 12 to 18 inches.

With implementation of the above recommendations, it is our opinion that the resulting pavement section will provide equivalent or greater structural strength than the dry weather pavement section currently planned. However, it should be noted that construction in wet weather is risky and the performance of pavement subgrades depend on a number of factors including the weather conditions, the contractor's methods, and the amount of traffic the road is subjected to. There is a potential that soft spots may develop even with implementation of the wet weather provisions recommended in this letter. If soft spots in the subgrade are identified during roadway excavation, or develop prior to paving, the soft spots should be over-excavated and backfilled with additional crushed rock.

During subgrade excavation, care should be taken to avoid disturbing the subgrade soils. Removals should be performed using an excavator with a smooth-bladed bucket. Truck traffic should be limited until an adequate working surface has been established. We suggest that the crushed rock be spread using bulldozer equipment rather than dump trucks, to reduce the amount of traffic and potential disturbance of subgrade soils. Care should be taken to avoid over-compaction of the base course materials, which could create pumping, unstable subgrade soil conditions. Heavy and/or vibratory compaction efforts should be applied with caution. Following placement and compaction of the crushed rock to project specifications (95 percent of Modified Proctor), a finish proof-roll should be performed before paving.

The above recommendations are subject to field verification. GeoPacific should be on-site during construction to verify subgrade strength and to take density tests on the engineered fill, base rock and asphaltic pavement materials.

6.14 Cement Amending Procedures

This section provides recommendations for conducting cement amending should the method of subgrade stabilization be incorporated into project design for subgrade stabilization. The moisture sensitive subgrade soils make the site a difficult wet weather construction project. The client and contractor should be prepared that wet weather construction may be risky and costly.

We anticipate that cement treated soils would primarily consist of Lean CLAY. For planning purposes, the amount of cement used during treatment should be based on an assumed soil dry unit weight of 100 pounds per cubic foot for fine-grained soils. We anticipate that treatment of the onsite soils would involve mixing cement powder to approximately 5 to 6 percent cement content and a mixing depth on the order of 12 to 16 inches. Actual percentages of cement required to achieve design strength will ultimately be determined by the lab testing results prior to construction and the soil moisture content at the time of placement. GeoPacific should evaluate the moisture content of the roadway subgrade before cement amendment. The amount of cement used may need to be increased or adjusted depending on the soil moisture content, particularly if soils are in excess of 10 percent over optimum moisture content. Portland cement content should not exceed 8 percent without prior approval.

Cement amendment should be conducted with a maximum lift thickness of 16 inches. Cement amending operations should not be conducted during periods of heavy rainfall, or when the outside temperature is less than 40 degrees Fahrenheit. Following adequate placement and tilling of cement amended subgrade soils, a static, sheep's-foot compactor should immediately be utilized to thoroughly compact the cement amended fill to at least 95 percent of the maximum dry density determined by ASTM D558 (Standard Test Method for Moisture-Density Unit Weight Relations of Soil-Cement Mixtures). A vibratory compactor is not recommended because it may further disturb the existing subgrade soils. During placement of cement amended fill soils, density testing should be performed to verify compliance with project specifications. Generally, one compaction test is performed for each vertical foot of cement amended engineered fill placed, and for every 100 to 200 linear feet within the alignment. Field density testing should conform to ASTM D6938, D2922, and D3017. Soil-cement compression test specimens of cement amended soils may be obtained and tested in the soils laboratory in accordance with ASTM D558-04. A compressive strength in the range of 200 to 400 psi as determined by ASTM D 1633 Method A should ideally be achieved. If the soil moisture content is approximately 5 percent over optimum moisture content, as recommended, we anticipate that placement of 6 percent cement by weight of dry soil will be sufficient to achieve the required compressive strength. However, minimum cement percentage will be determined based upon the results of laboratory testing.

The contractor should avoid impacting the treated soils for a minimum period of 4 to 5 days to allow the cement to cure prior to subjecting the subgrade to construction traffic. After the initial cure period, a proof-roll should be observed prior to routing construction traffic over cement treated areas. Impacting the treated base with heavy equipment prior to final cure of the treated soils could reduce final cure strengths and soft areas may develop.

The primary risk associated with cement treatment of roadway subgrade soils is that there is a potential for soft areas to develop following treatment if there in inadequate cement content added to the soil, blending of cement, or compaction of treated soils. Also, soft areas may develop where soils which may have been disturbed underlying the area of treatment are not adequately removed or treated. It is possible that even after careful treatment with recommended percentages, soft areas may still be present which would require additional over-excavation.

7.0 SEISMIC DESIGN

The Oregon Department of Geology and Mineral Industries (DOGAMI), Oregon HazVu: 2020 Statewide GeoHazards Viewer indicates that the site is in an area where *very strong* ground shaking is anticipated during an earthquake. Structures should be designed to resist earthquake loading in accordance with the methodology described in the 2018 International Building Code (IBC) with applicable Oregon Structural Specialty Code (OSSC) revisions (current 2019). We recommend Site Class D be used for design as defined in ASCE 7-16, Chapter 20, and Table 20.3-1. Design values determined for the site using the ATC Hazards by Location 2019 Seismic Design Maps Summary Report are summarized in Table 5 and are based upon observed existing soil conditions.

Table 5: Recommended Earthquake Ground Motion Parameters (ASCE-7-16)

Parameter	Value
Location (Lat, Long), degrees	45.307, -122.938
Probabilistic Ground Motion Values, 2% Probability of Exceedance in 50 yrs	
Peak Ground Acceleration PGA_M	0.472 g
Short Period, S_s	0.848 g
1.0 Sec Period, S_1	0.409 g
Soil Factors for Site Class D:	
F_a	1.161
* F_v	1.891
$SD_s = 2/3 \times F_a \times S_s$	0.656 g
$*SD_1 = 2/3 \times F_v \times S_1$	0.515 g
Seismic Design Category	D

* F_v value reported in the above table is a straight-line interpolation of mapped spectral response acceleration at 1-second period, S_1 per Table 1613.2.3(2) with the assumption that Exception 2 of ASCE 7-16 Chapter 11.4.8 is met per the Structural Engineer. If Exception 2 is not met, and the long-period site coefficient (F_v) is required for design, GeoPacific Engineering can be consulted to provide a site-specific procedure as per ASCE 7-16, Chapter 21.

7.1 Soil Liquefaction

The Oregon Department of Geology and Mineral Industries (DOGAMI), Oregon HazVu: 2020 Statewide GeoHazards Viewer indicates that the site contains areas considered to be at *low* 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 typically occurs in loose sands, and granular soils located below the water table, and fine-grained soils with a plasticity index less than 10, and SPT N-Values lower than 15. The subsurface profile observed within our subsurface explorations, which extended to a maximum depth of 60 feet bgs, indicated that the site is underlain by medium stiff to stiff, low to moderately plastic, Lean CLAY, SILT and interlayered Clayey SILT, and Sandy SILT. On December 13, and December 16, 2019, observed soil moisture conditions were generally very moist to wet. Groundwater seepage was observed within test pit TP-3 at an approximate depth of 9 feet bgs.

The liquefaction potential at the subject site was analyzed for the soil profile encountered within cone penetrometer CPT-1 using CLiq version 3.0.2.4, by Geologismiki, and the Robertson clay-like behavior (2009) method of analysis. The depth of analysis was 60 feet bgs. The groundwater table during an earthquake was estimated to be 8 feet bgs during an earthquake. Using a peak horizontal ground acceleration of 0.47g, and an earthquake moment magnitude of 9.10 based upon data obtained from the U.Ss. Geological Survey (USGS) 2019 Earthquake Hazards Program, the factor of safety was less than 1 for some soil layers, indicating the potential for liquefaction during an earthquake. Based upon our analysis of the existing soil profile, potentially liquefiable layers are most prevalent underlying the subject site at depths ranging from 12 to 18 feet bgs, 30 to 40 feet bgs, and 50 to 60 feet bgs. Soils meeting the criteria for potentially liquefiable soil layers during an earthquake at this site include low plasticity SILT, and sandy SILT soils located below the static groundwater table. Our analysis indicates that total dynamic settlement due to soil liquefaction at the location of CPT-1 is anticipated to be approximately 3.9 inches. We anticipate that differential settlement would be approximately one-half of the total estimated settlement, measured between two adjacent building foundation components, or over a span of approximately 20 feet. Based on the relatively level topography at the site, and the lack of free slope faces in the vicinity of the subject site, it is our opinion that the risk of damage to the proposed structure due to lateral spreading is very low.

The design team and structural engineer should work together to determine the maximum allowable settlement that is considered to be tolerable to the structure during a strong seismic event. If determined necessary, soil liquefaction and lateral spreading may potentially be reduced to within tolerable limits with deep ground improvements. Methods such as installation of rammed aggregate piers (GeoPiers), stone columns, or deep soil mixing columns (DSM), may be feasible options. The geotechnical engineer should work closely with the design team to develop appropriate recommendations for the site.

8.0 UNCERTAINTIES AND LIMITATIONS

We have prepared this report for the owner and their consultants for use in design of this project only. This report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, 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 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 attempted to execute 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, expressed 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.

CERTIFIED OREGON Benjamin L. Cook E2245 ENGINEERING GEOLOGIST
EXP 1/1/21

Benjamin L. Cook, C.E.G.
Senior Engineering Geologist

REGISTERED PROFESSIONAL ENGINEER 14743PE OREGON JAN. 23, 1990 JAMES D. IMBRIE
EXPIRES: 06/30/2021

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

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CHECKLIST OF RECOMMENDED GEOTECHNICAL TESTING AND OBSERVATION

Item No.	Procedure	Timing	By Whom	Done
1	Preconstruction meeting	Prior to beginning site work	Contractor, Developer, Civil and Geotechnical Engineers	
2	Fill removal from site or sorting and stockpiling	Prior to mass stripping	Soil Technician/ Geotechnical Engineer	
3	Stripping, aeration, and root-picking operations	During stripping	Soil Technician	
4	Compaction testing of engineered fill (95% of Standard Proctor)	During filling, tested every 2 vertical feet	Soil Technician	
5	Foundation Subgrade Compaction (95% of Modified Proctor)	During Foundation Preparation, Prior to Placement of Reinforcing Steel	Soil Technician/ Geotechnical Engineer	
6	Compaction testing of trench backfill (95% of Modified Proctor)	During backfilling, tested every 4 vertical feet for every 200 linear feet	Soil Technician	
7	Street Subgrade Inspection (95% of Standard Proctor)	Prior to placing base course	Soil Technician	
8	Base course compaction (95% of Modified Proctor)	Prior to paving, tested every 200 linear feet	Soil Technician	
9	Asphalt Compaction (92% Rice Value)	During paving, tested every 100 linear feet	Soil Technician	
10	Final Geotechnical Engineer's Report	Completion of project	Geotechnical Engineer	

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FIGURES

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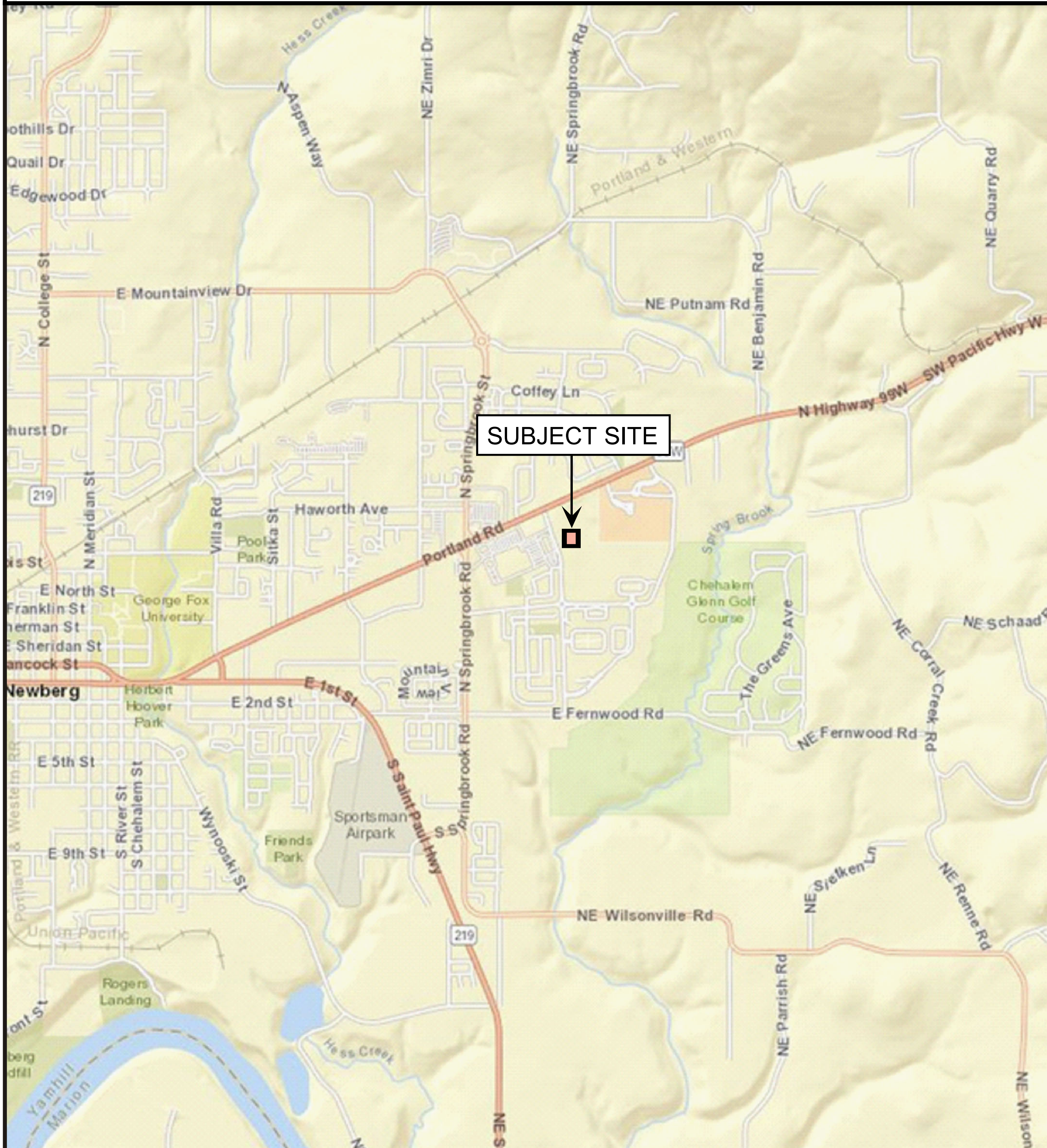
SITE VICINITY MAP



Base Map: Street View, 2019
Date: 12/27/2019
Drawn by: BLC

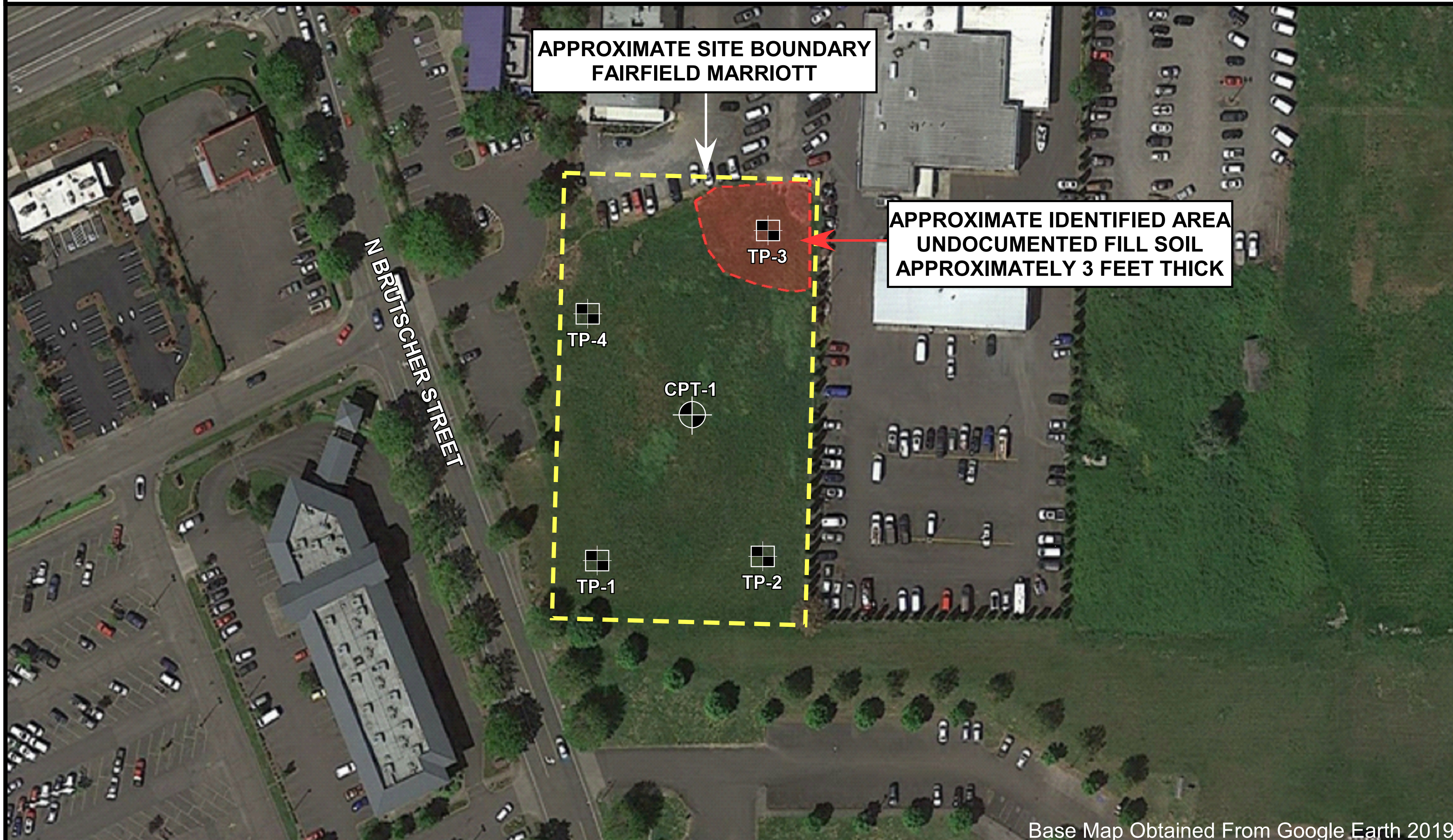
North

Project: Fairfield Marriott Tax Lot 01900, R3216 Newberg, Oregon 97132	Project No. 19-5391	FIGURE 1

GeoPacific Engineering, Inc.

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

SITE AERIAL AND EXPLORATION LOCATIONS



Legend:

- Test Pit Exploration Designation and Approximate Location
- Cone Penetrometer Test Designation and Approximate Location

APPROXIMATE SCALE (FEET)

60 0 60 120

Drawn By: BLC
Date:12/27/2019

North

Project: Fairfield Marriott Tax Lot 01900, R3216, Newberg, Oregon 97132	Project No. 19-5391	FIGURE 2

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

SITE PLAN AND EXPLORATION LOCATIONS

Legend:

- Test Pit Exploration Designation and Approximate Location
- Cone Penetrometer Test Designation and Approximate Location

APPROXIMATE SCALE (AS NOTED)

Drawn By: BLC

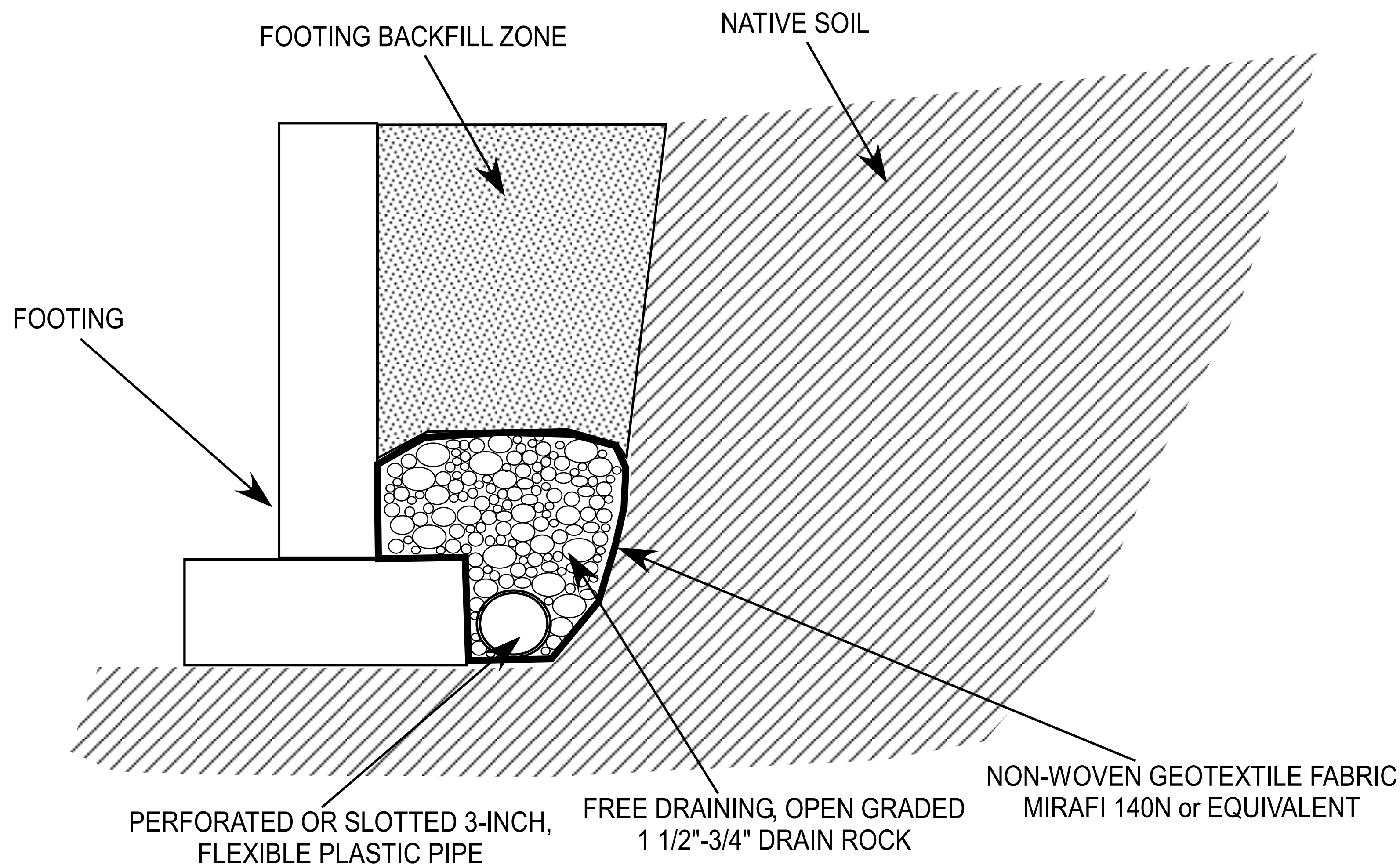
Date:12/30/2019

North

Project: Fairfield Marriott Tax Lot 01900, R3216, Newberg, Oregon 97132	Project No. 19-5391	FIGURE 3

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

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: 12/30/2019
Drawn by: BLC

Project: Fairfield Marriott Tax Lot 01900, R3216, Newberg, Oregon 97132	Project No. 19-5391	FIGURE 4

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EXPLORATION LOGS

GeoPacific Engineering, Inc.
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Portland, Oregon 97224
Tel: (503) 598-8445 Fax: (503) 941-9281

TEST PIT LOG

Project: Fairfield Marriott
Newberg, Oregon

Project No.19-5391

Test Pit No. **TP-1**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Torvane Shear (tons/ft²)	Sample Type	% Passing No. 200 Sieve	Moisture Content (%)	Water Bearing Zone	Material Description
0–0.5							TOPSOIL. Organic Lean CLAY (OL-CL), brown, damp to moist, fine roots, extends to approximately 6 inches bgs.
1	2.5						Lean CLAY (CL), light brown, medium stiff to very stiff, moist, pinhole structure, low to moderate plasticity.
2	3.0						
3	4.0		100 to 1,000 g	97.3	21.6		AASHTO Classification= A-7-6(26); LL=47; PI=23
4	4.0						
5							Lean CLAY (CL), brown, very stiff, very moist, pinhole structure, low to moderate plasticity.
6			100 to 1,000 g	97.2	24.9		AASHTO Classification= A-7-6(25); LL=49; PI=22
7							SILT (ML), brown, stiff, moist, pinhole structure, low plasticity.
8							
9			100 to 1,000 g	99.4	27.0		AASHTO Classification= A-6(16); LL=40; PI=14
10							Test pit terminated at 10 feet bgs. No groundwater observed
11							
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: 12/13/2019
Logged By: B. Cook
Surface Elevation: 228 Feet

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

TEST PIT LOG

Project: Fairfield Marriott
Newberg, Oregon

Project No.19-5391

Test Pit No. **TP-2**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Torvane Shear (tons/ft^2)	Sample Type	% Passing No. 200 Sieve	Moisture Content (%)	Water Bearing Zone	Material Description
0–1.5							TOPSOIL. Organic Lean CLAY (OL-CL), brown, damp to moist, fine roots, extends to approximately 18 inches bgs. Old farm till zone.
1	1.5						
1.5–3							Lean CLAY (CL), light brown, medium stiff to stiff, moist, pinhole structure, low to moderate plasticity.
2	2.5						
3–7							Lean CLAY (CL), brown, stiff to very stiff, very moist, pinhole structure, low to moderate plasticity.
3	4.0						
4	4.0						
5							
6							
7–10							SILT (ML), brown, stiff, moist, pinhole structure, low plasticity.
8							
9							
10							Test pit terminated at 10 feet bgs. No groundwater observed
11							
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: 12/13/2019
Logged By: B. Cook
Surface Elevation: 226 Feet

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

TEST PIT LOG

Project: Fairfield Marriott
Newberg, Oregon

Project No.19-5391

Test Pit No. **TP-3**

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Torvane Shear (tons/ft^2)	Sample Type	% Passing No. 200 Sieve	Moisture Content (%)	Water Bearing Zone	Material Description
0–1.5							TOPSOIL. Organic Lean CLAY (OL-CL), dark brown, damp to moist, fine roots, extends to approximately 18 inches bgs. Old farm till zone.
1	0.5						
1.5–3							FILL. Lean CLAY (CL), light brown, soft, moist, contains minor plastic debris fragments.
2	0.5						
3–6							Lean CLAY (CL), brown, stiff to very stiff, very moist, pinhole structure, low to moderate plasticity.
3	3.0						
4	3.5						
6–10							SILT (ML), brown, stiff becoming medium stiff at -9 feet, very moist to wet, low plasticity.
8–9						Seepage	
9–10						Water Bearing Zone	
10							Test pit terminated at 10 feet bgs. Groundwater seepage observed at 9 feet bgs.

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: 12/13/2019
Logged By: B. Cook
Surface Elevation: 225 Feet

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

TEST PIT LOG

Project: Fairfield Marriott Newberg, Oregon	Project No.19-5391	Test Pit No. TP-4

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Torvane Shear (tons/ft^2)	Sample Type	% Passing No. 200 Sieve	Moisture Content (%)	Water Bearing Zone	Material Description
0–1.5							TOPSOIL. Organic Lean CLAY (OL-CL), dark brown, damp to moist, fine roots, extends to approximately 18 inches bgs. Old farm till zone. Plastic perforated drain pipe encountered.
1	2.5						
1.5–6							Lean CLAY (CL), brown, medium stiff to very stiff, very moist, pinhole structure, low to moderate plasticity.
2	2.5						
3	4.0						
4	4.0						
6–10							SILT (ML), brown, stiff to very stiff, moist, pinhole structure, low plasticity.
10							Test pit terminated at 10 feet bgs. No groundwater observed

Depth scale: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17

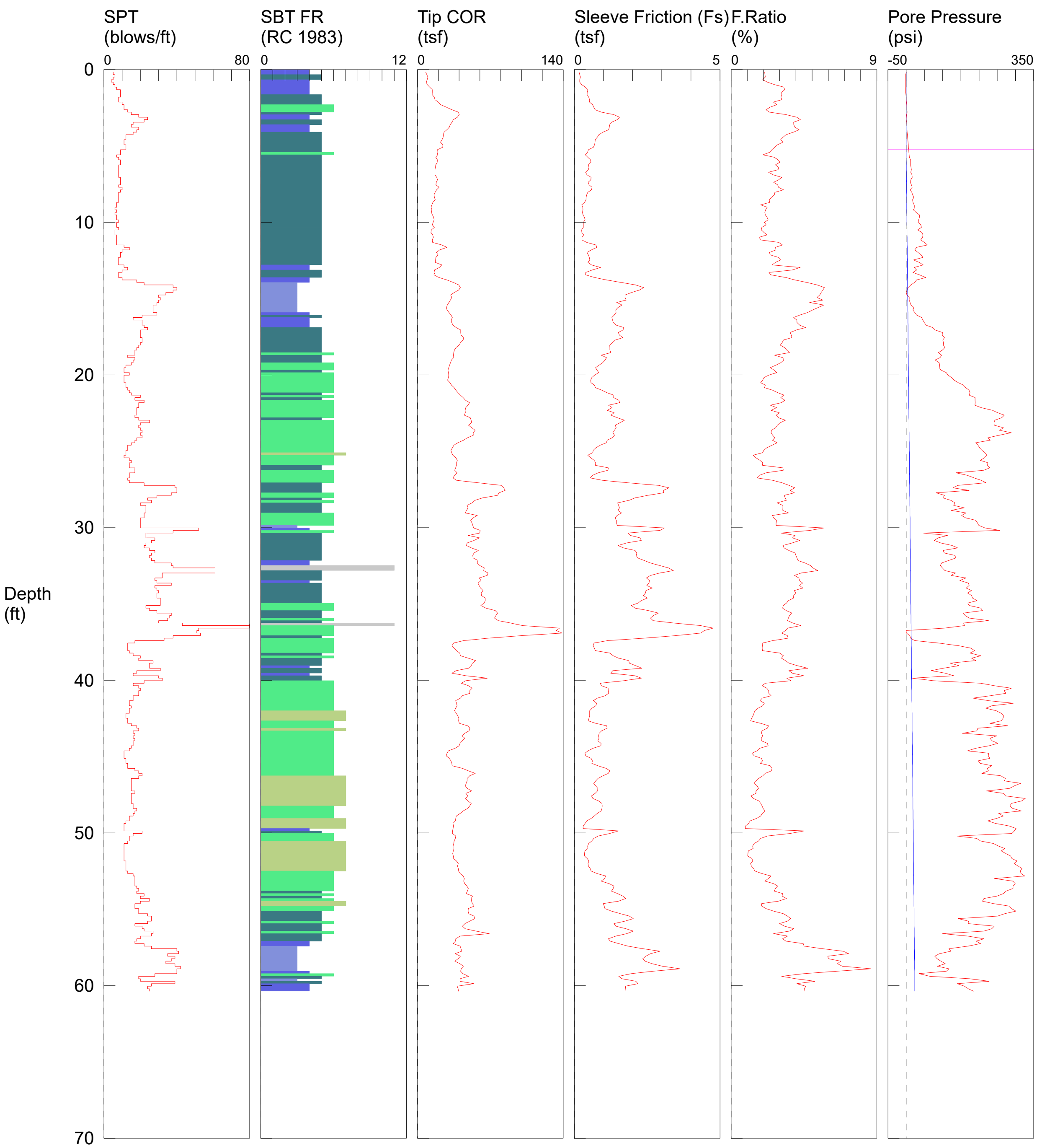
LEGEND

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: 12/13/2019
Logged By: B. Cook
Surface Elevation: 227 Feet

GeoPacific / CPT-1 / N Brutscher St Newberg

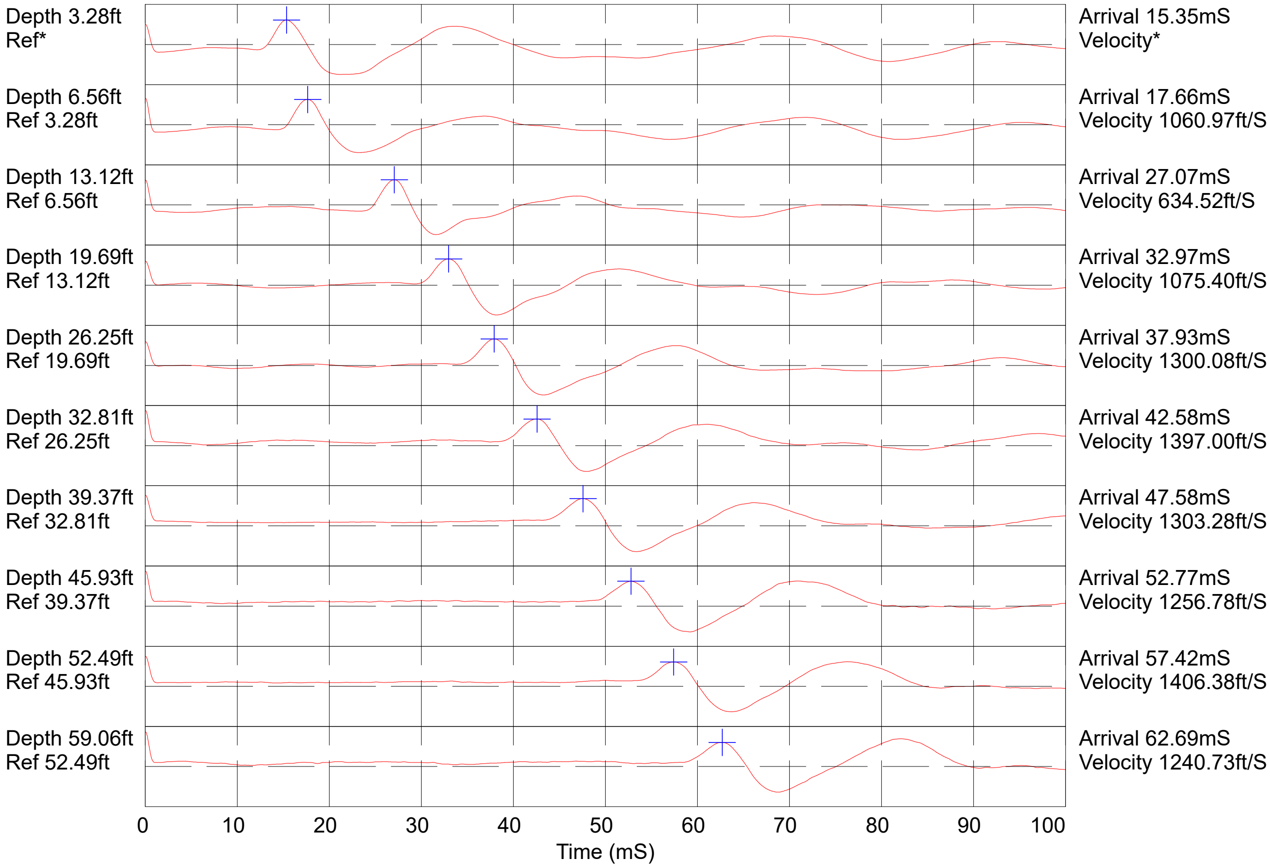
OPERATOR: OGE BAK
CONE ID: DPG1211
HOLE NUMBER: CPT-1
TEST DATE: 12/16/2019 9:15:53 AM
TOTAL DEPTH: 60.367 ft



1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

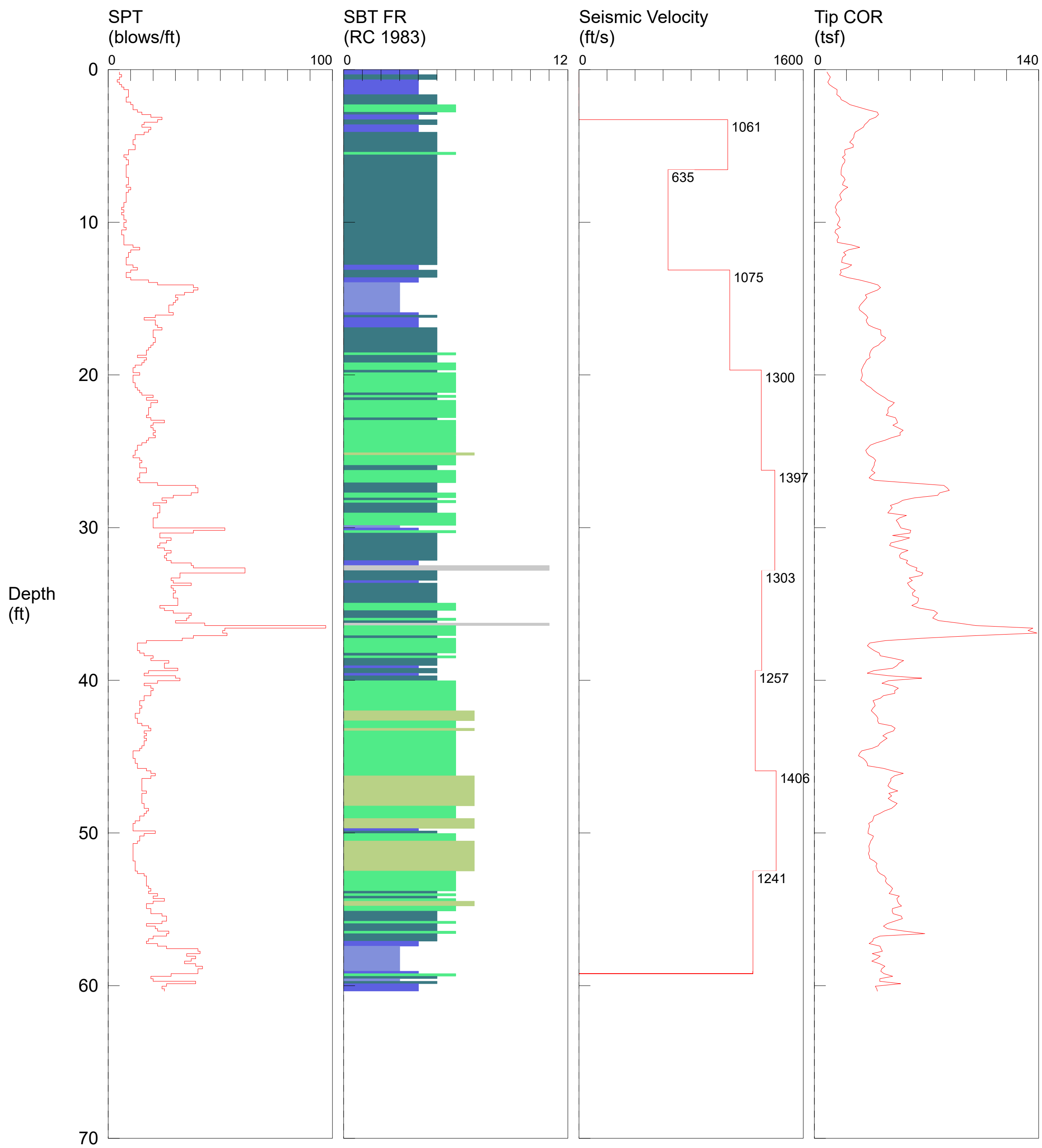
COMMENT: GeoPacific / CPT-1 / 901 N Brutscher St Newberg



Hammer to Rod String Distance (ft): 4.27
* = Not Determined

GeoPacific / CPT-1 / N Brutscher St Newberg

OPERATOR: OGE BAK
CONE ID: DPG1211
HOLE NUMBER: CPT-1
TEST DATE: 12/16/2019 9:15:53 AM
TOTAL DEPTH: 60.367 ft

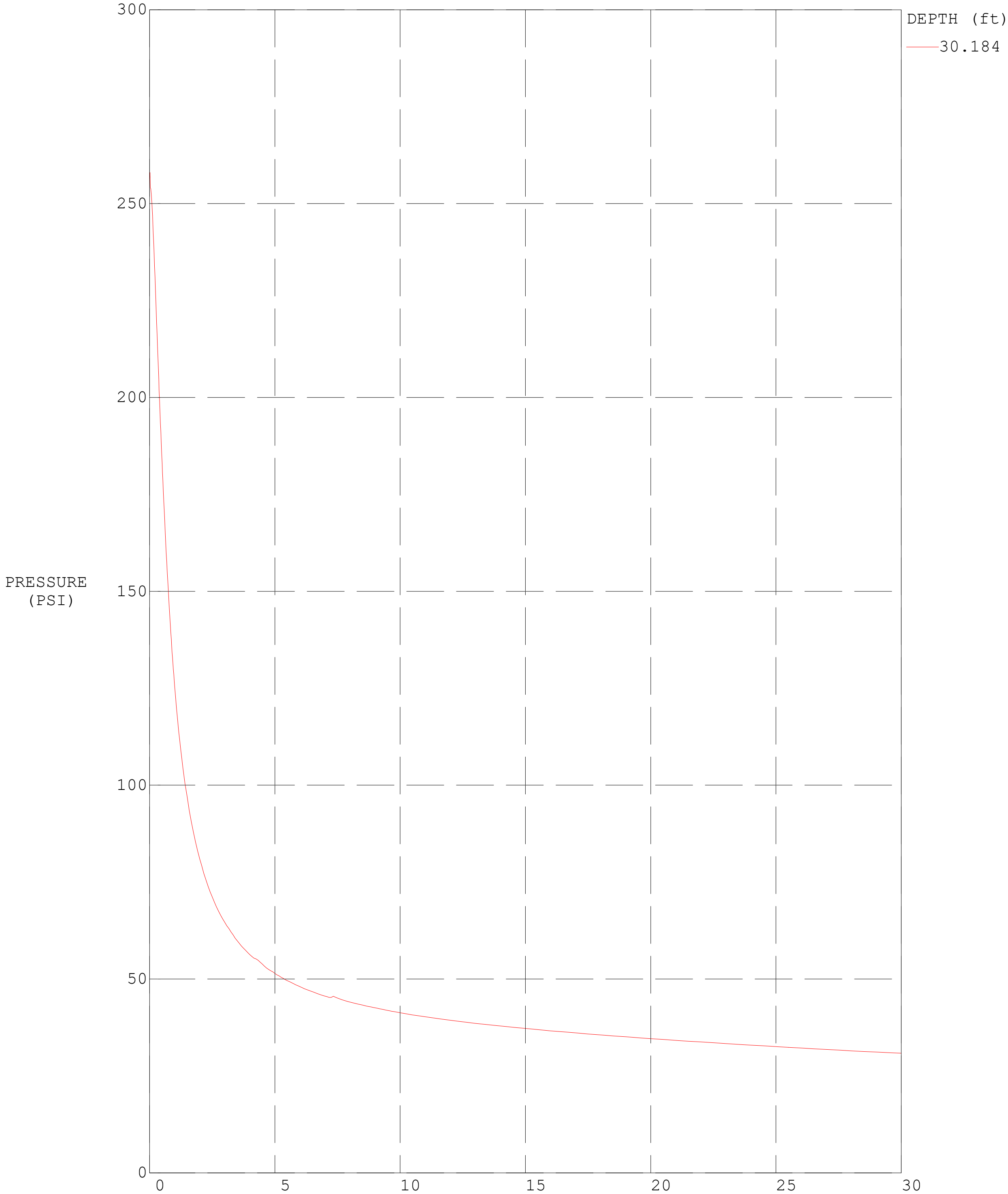


1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: GeoPacific / CPT-1 / 901 N Brutscher St Newberg

TEST DATE: 12/16/2019 9:15:53 AM

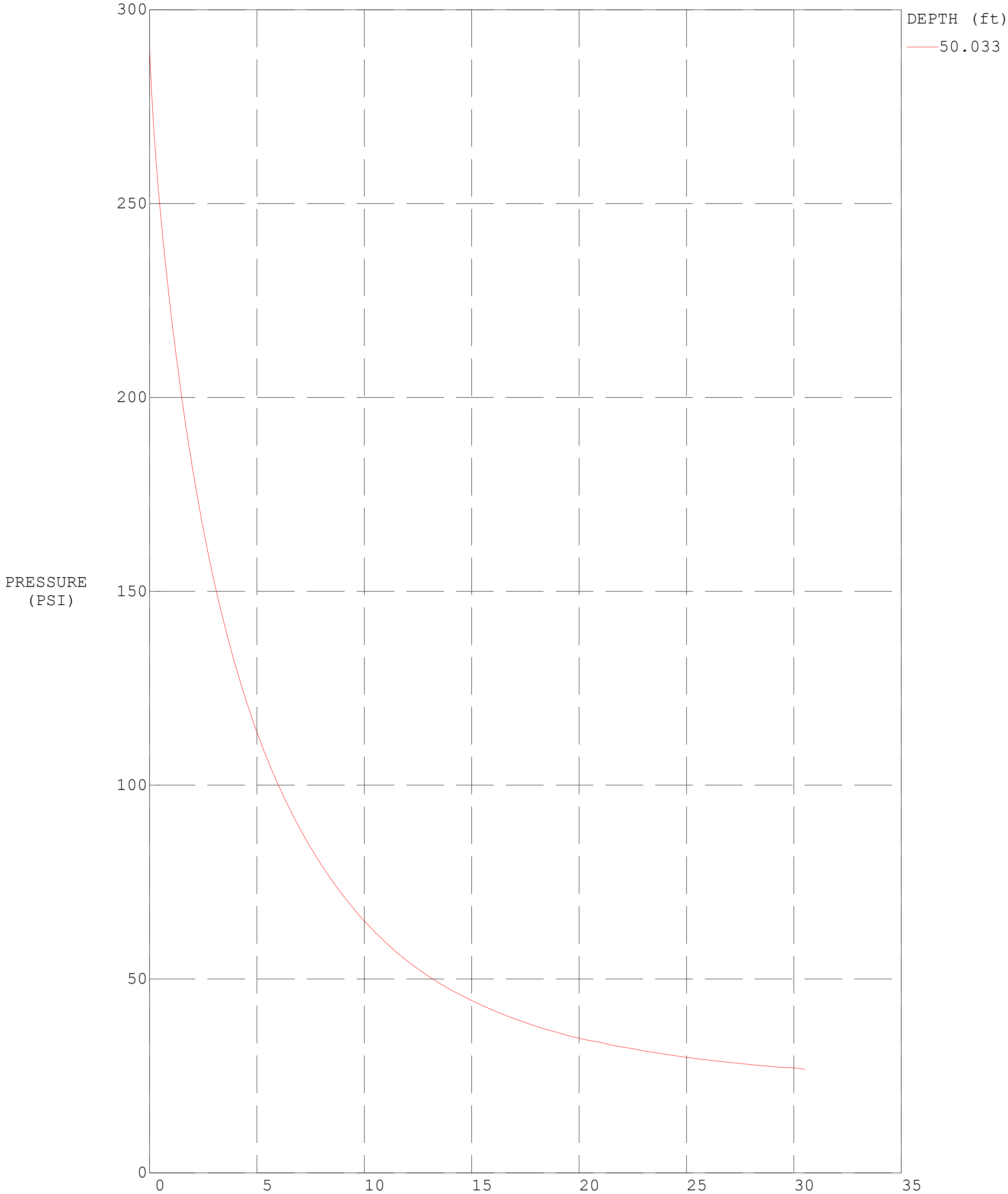


MAXIMUM PRESSURE = 258.014 (PSI)

HYDROSTATIC PRESSURE = 11.596 (PSI), WATER TABLE: 3.43 ft

COMMENT: GeoPacific / CPT-1 / 901 N Brutscher St Newberg

TEST DATE: 12/16/2019 9:15:53 AM



MAXIMUM PRESSURE = 291.065 (PSI)E: (MINUTES)

HYDROSTATIC PRESSURE = 20.198 (PSI), WATER TABLE: 3.43 ft

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LABORATORY TEST RESULTS

14835 SW 72nd Avenue
Portland, Oregon 97224

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

UNIFIED SOIL CLASSIFICATION SYSTEM

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

COARSE-GRAINED SOILS (more than 50% of material is larger than No. 200 sieve size.)		
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	Clean Gravels (Less than 5% fines)	
	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
	GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
	Gravels with fines (More than 12% fines)	
	GM	Silty gravels, gravel-sand-silt mixtures
	GC	Clayey gravels, gravel-sand-clay mixtures
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	Clean Sands (Less than 5% fines)	
	SW	Well-graded sands, gravelly sands, little or no fines
	SP	Poorly graded sands, gravelly sands, little or no fines
	Sands with fines (More than 12% fines)	
	SM	Silty sands, sand-silt mixtures
	SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (50% or more of material is smaller than No. 200 sieve size.)		
SILTS AND CLAYS Liquid limit less than 50%	ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts with slight plasticity
	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
	OL	Organic silts and organic silty clays of low plasticity
SILTS AND CLAYS Liquid limit 50% or greater	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
	CH	Inorganic clays of high plasticity, fat clays
	OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS	PT	Peat and other highly organic soils

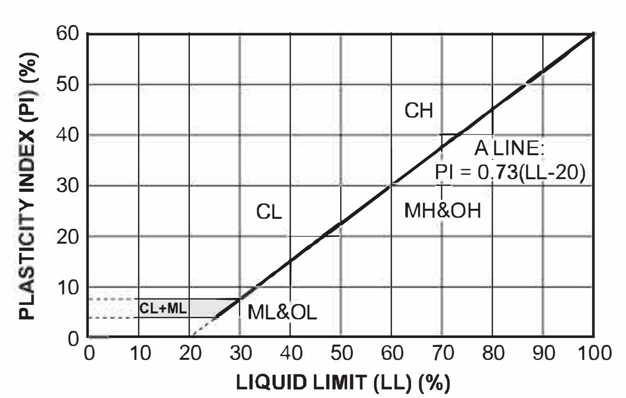
LABORATORY CLASSIFICATION CRITERIA

GW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
GP	Not meeting all gradation requirements for GW	
GM	Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
GC	Atterberg limits above "A" line with P.I. greater than 7	
SW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
SP	Not meeting all gradation requirements for GW	
SM	Atterberg limits below "A" line or P.I. less than 4	Limits plotting in shaded zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols.
SC	Atterberg limits above "A" line with P.I. greater than 7	

Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:

Less than 5 percent .... GW, GP, SW, SP
More than 12 percent .... GM, GC, SM, SC
5 to 12 percent .... Borderline cases requiring dual symbols

PLASTICITY CHART



SOIL DESCRIPTION AND CLASSIFICATION GUIDELINES

Particle-Size Classification

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

Consistency for Cohesive Soil

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

Relative Density for Granular Soil

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

Moisture Designations

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

AASHTO SOIL CLASSIFICATION SYSTEM

TABLE 1. Classification of Soils and Soil-Aggregate Mixtures

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

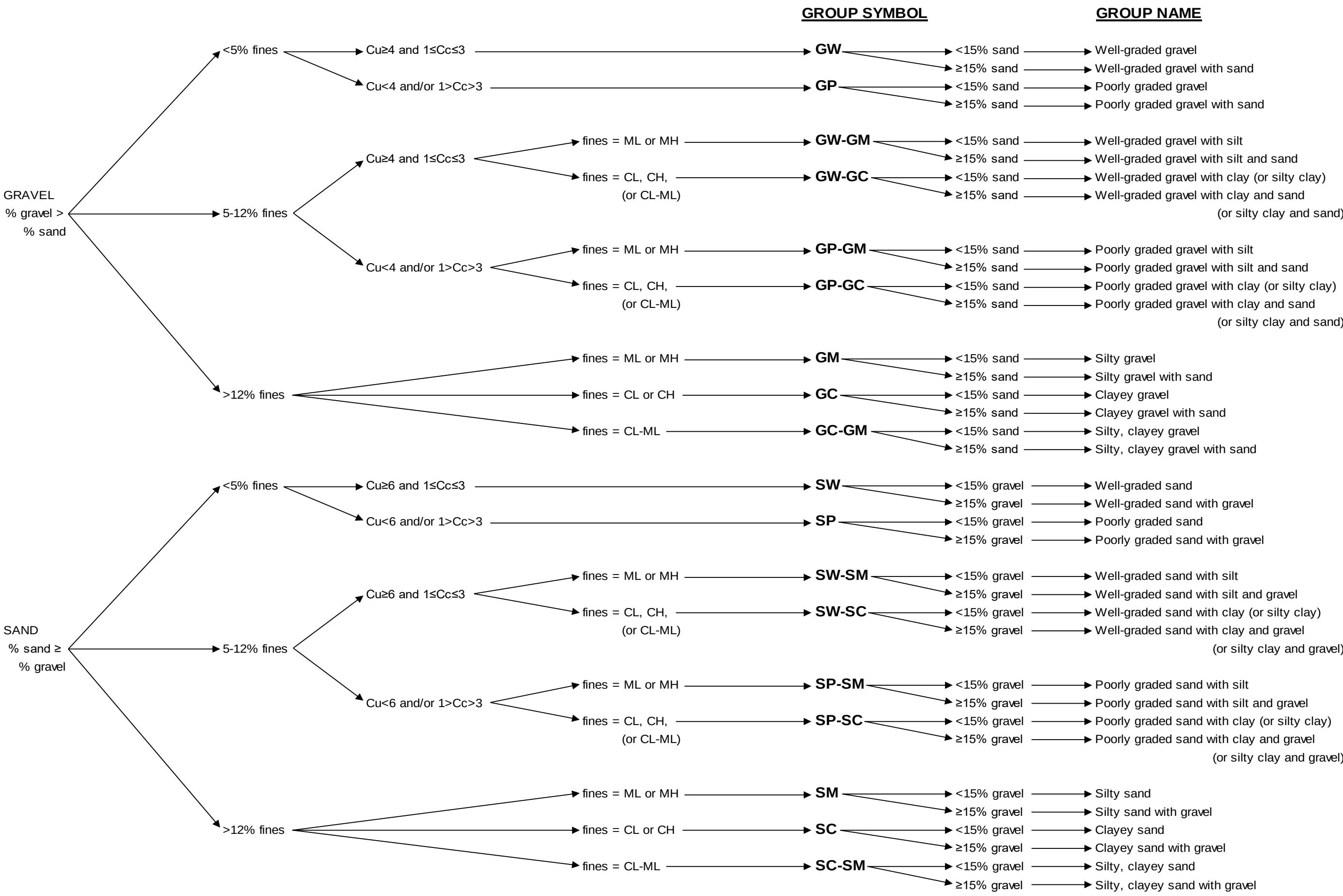
Note: The placing of A-3 before A-2 is necessary in the "left to right elimination process" and does not indicate superiority of A-3 over A-2.

TABLE 2. Classification of Soils and Soil-Aggregate Mixtures

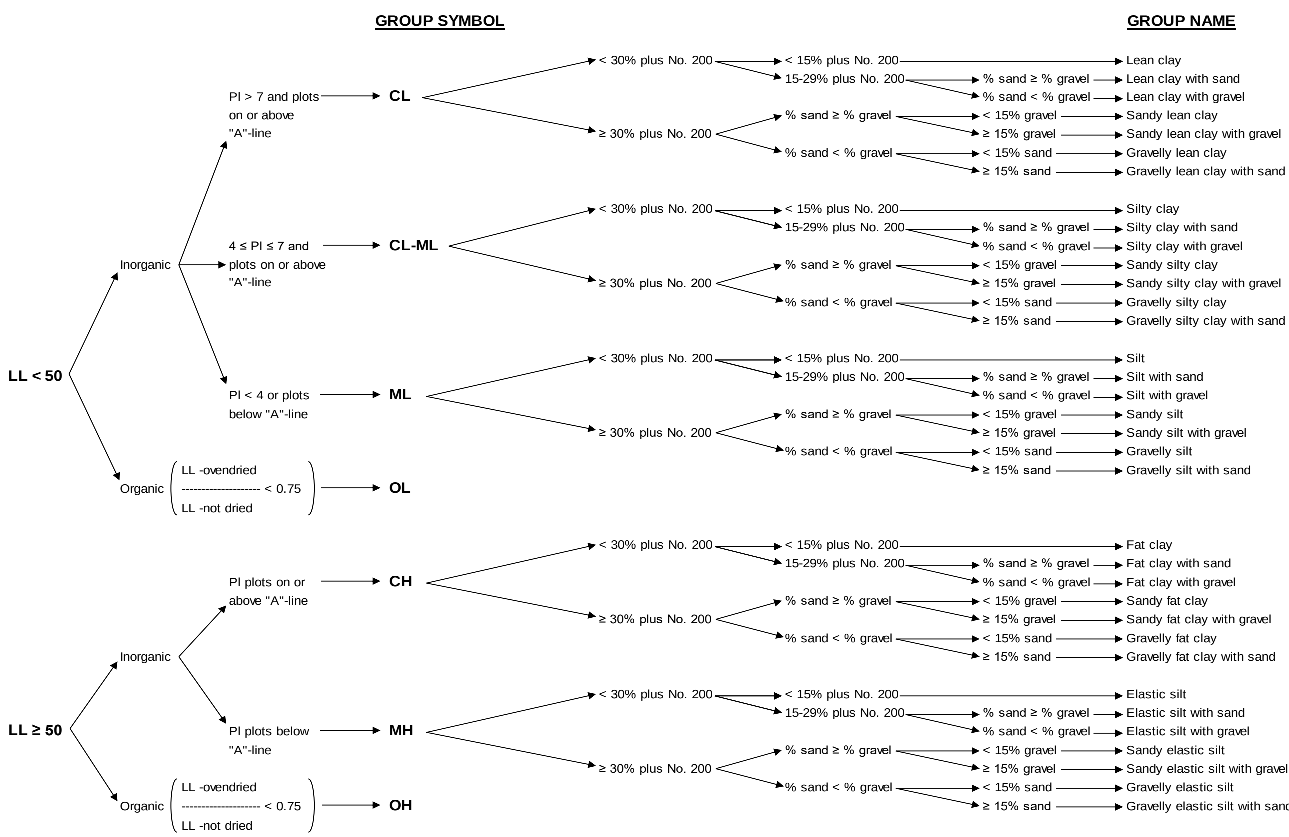
General Classification	Granular Materials (35 Percent or Less Passing 0.075 mm)							Silt-Clay Materials (More than 35 Percent Passing 0.075 mm)			
	A-1			A-2							A-7
Group Classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5, A-7-6
Sieve analysis, percent passing:											
2.00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

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

AASHTO = American Association of State Highway and Transportation Officials

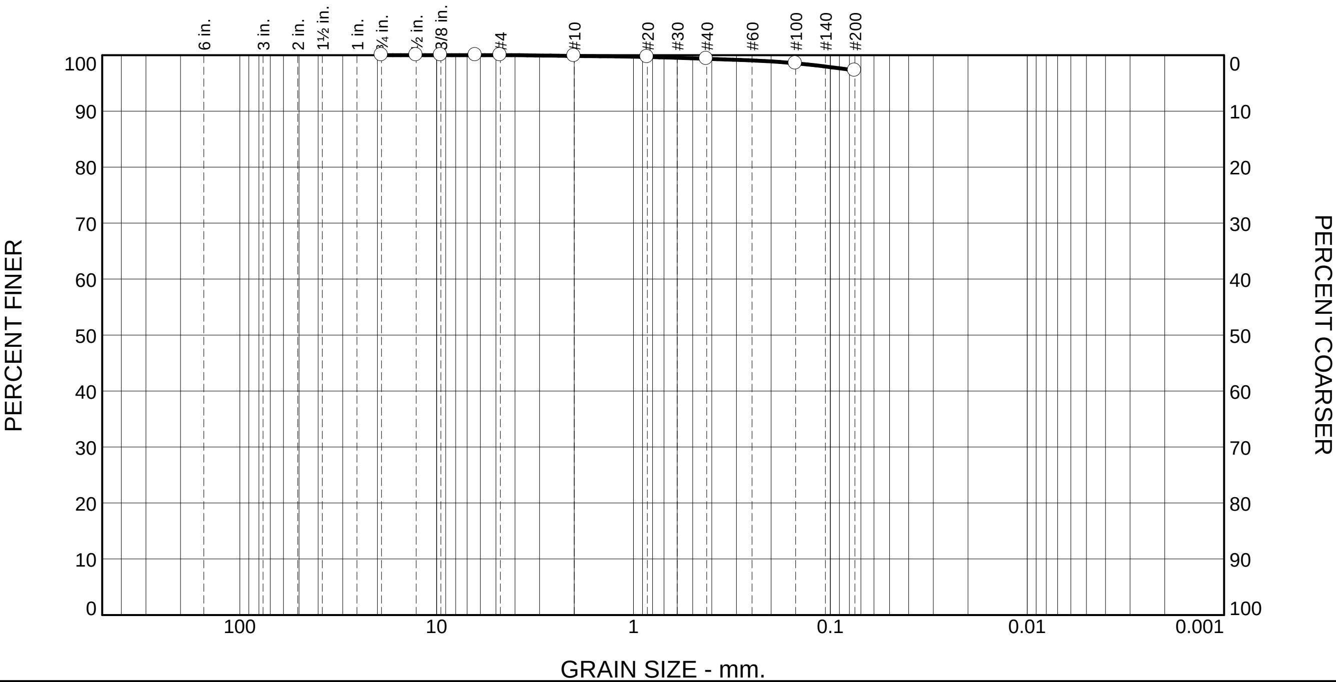


Flow Chart for Classifying Coarse-Grained Soils (More Than 50% Retained on No. 200 Sieve)



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

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.1	0.6	2.0	97.3	

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	99.9		
#20	99.7		
#40	99.3		
#100	98.5		
#200	97.3		

* (no specification provided)

Material Description

Lean Clay

Atterberg Limits (ASTM D 4318)

PL= 24.8 **LL=** 47.8 **PI=** 23.0

Classification

USCS (D 2487)= CL **AASHTO (M 145)=** A-7-6(26)

Coefficients

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

Remarks

Moisture 21.6%

Date Received: **Date Tested:** 12/16/2019

Tested By: SJC

Checked By:

Title:

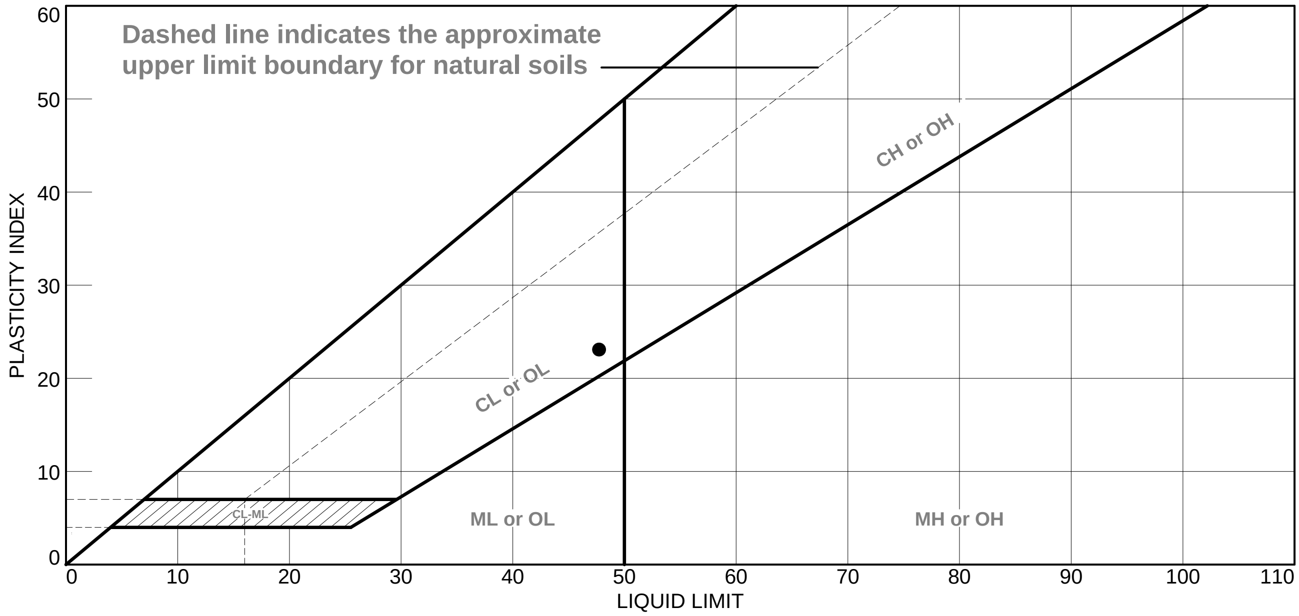
Location: TP-1
Sample Number: S19-307 Redo **Depth:** 3' **Date Sampled:** 12/13/2019

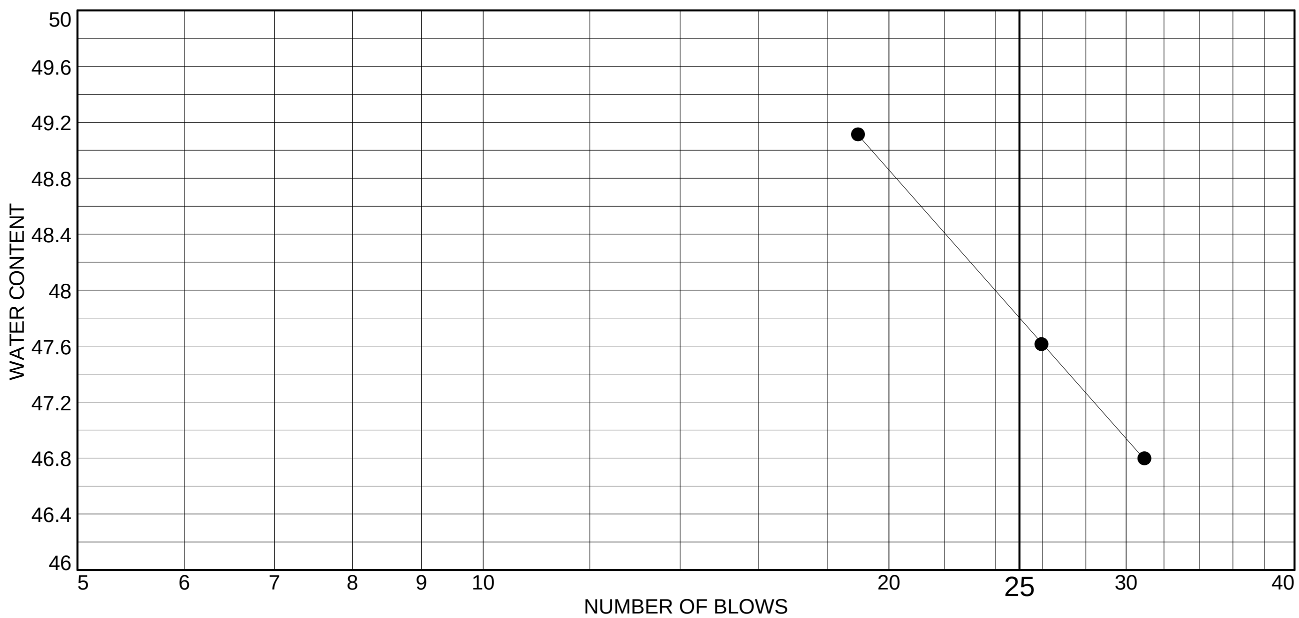
GEOPACIFIC ENGINEERING, INC.

Client: Hawkins Companies, LLC
Project: Fairfield Marriott Newberg

Project No: 19-5391 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Lean Clay	47.8	24.8	23.0	99.3	97.3	CL

Project No. 19-5391 **Client:** Hawkins Companies, LLC

Project: Fairfield Marriott Newberg

Location: TP-1

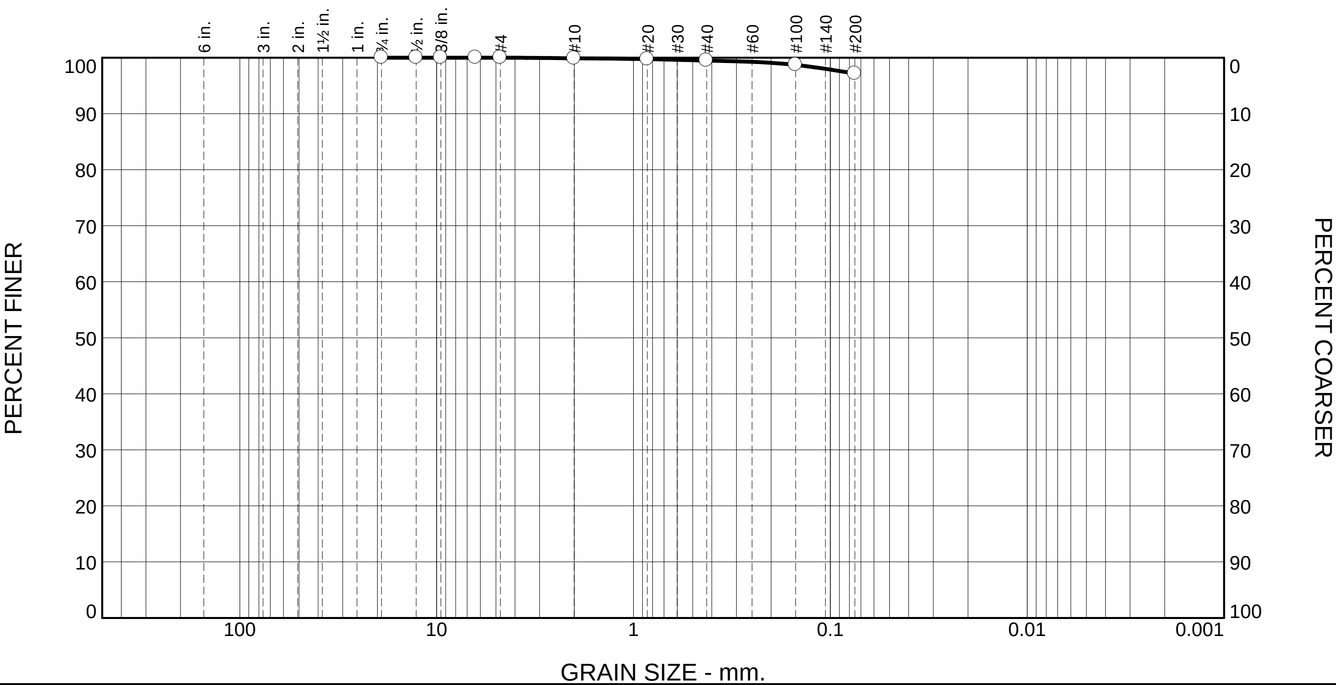
Sample Number: S19-307 Redo **Depth:** 3'

Remarks:

GEOPACIFIC ENGINEERING, INC.

Figure

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.1	0.4	2.3	97.2	

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	99.7		
#40	99.5		
#100	98.7		
#200	97.2		

* (no specification provided)

Material Description

Lean Clay

Atterberg Limits (ASTM D 4318)

PL= 27.0 **LL=** 49.1 **PI=** 22.1

Classification

USCS (D 2487)= CL **AASHTO (M 145)=** A-7-6(25)

Coefficients

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

Remarks

Moisture 24.9%

Date Received: **Date Tested:** 12/16/2019

Tested By: SJC

Checked By:

Title:

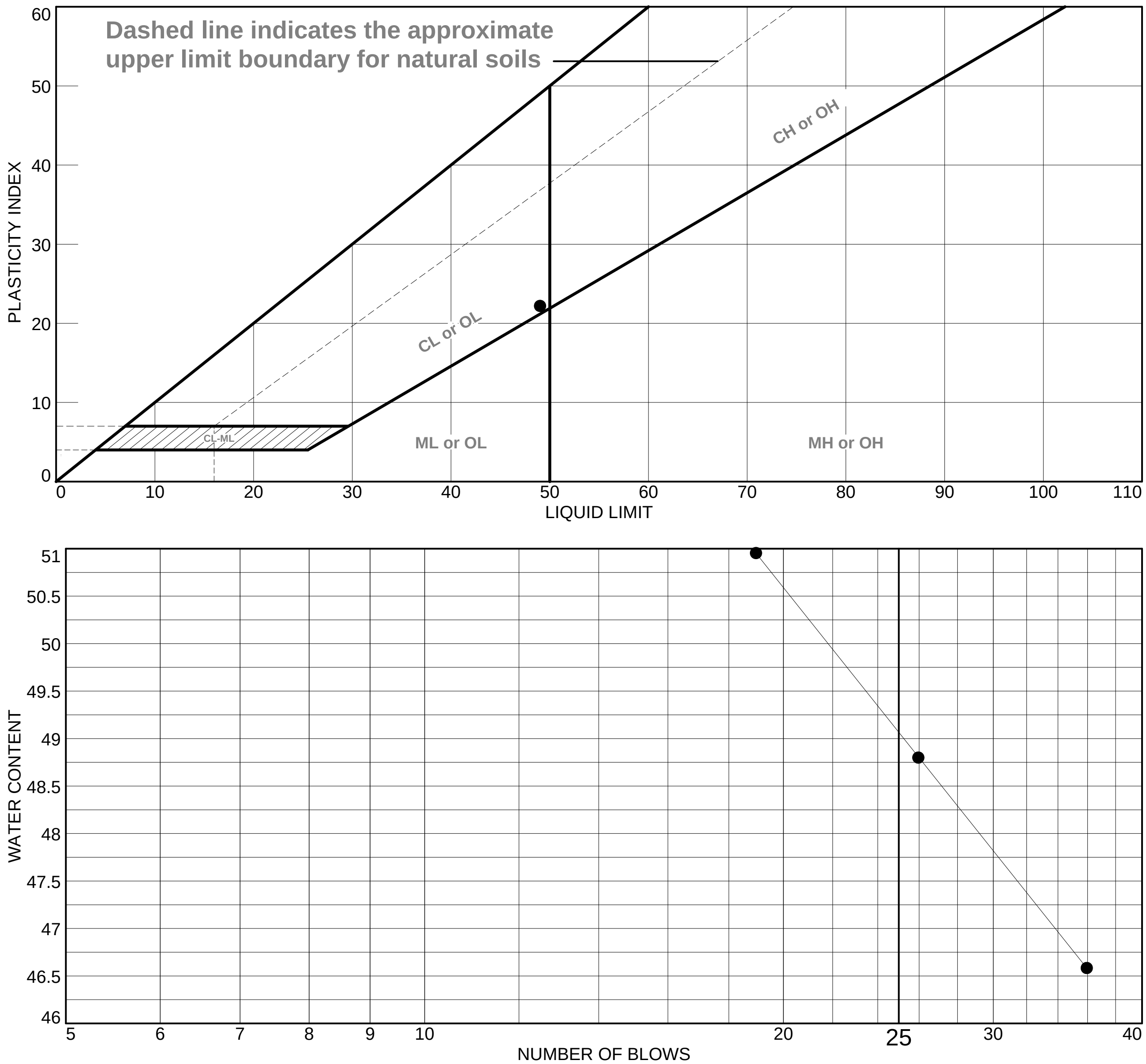
Location: TP-1
Sample Number: S19-308 **Depth:** 6' **Date Sampled:** 12/13/2019

GEOPACIFIC ENGINEERING, INC.

Client: Hawkins Companies, LLC
Project: Fairfield Marriott Newberg

Project No: 19-5391 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Lean Clay	49.1	27.0	22.1	99.5	97.2	CL

Project No. 19-5391 **Client:** Hawkins Companies, LLC

Project: Fairfield Marriott Newberg

Location: TP-1

Sample Number: S19-308 **Depth:** 6'

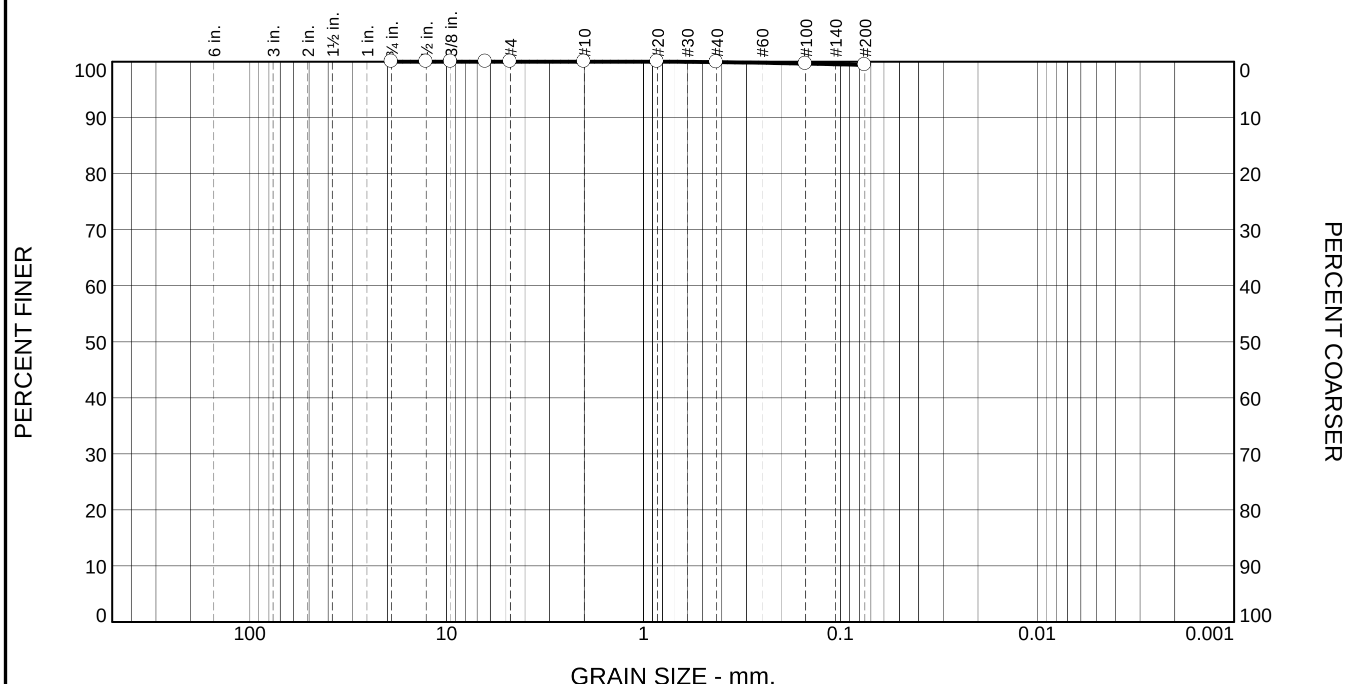
Remarks:

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	0.0	0.1	0.5	99.4	

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	100.0		
#40	99.9		
#100	99.7		
#200	99.4		

* (no specification provided)

Material Description

Silt

Atterberg Limits (ASTM D 4318)

PL= 26.3 **LL=** 40.3 **PI=** 14.0

Classification

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

Coefficients

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

Remarks

Moisture 27.0%

Date Received: **Date Tested:** 12/16/2019

Tested By: SJC

Checked By:

Title:

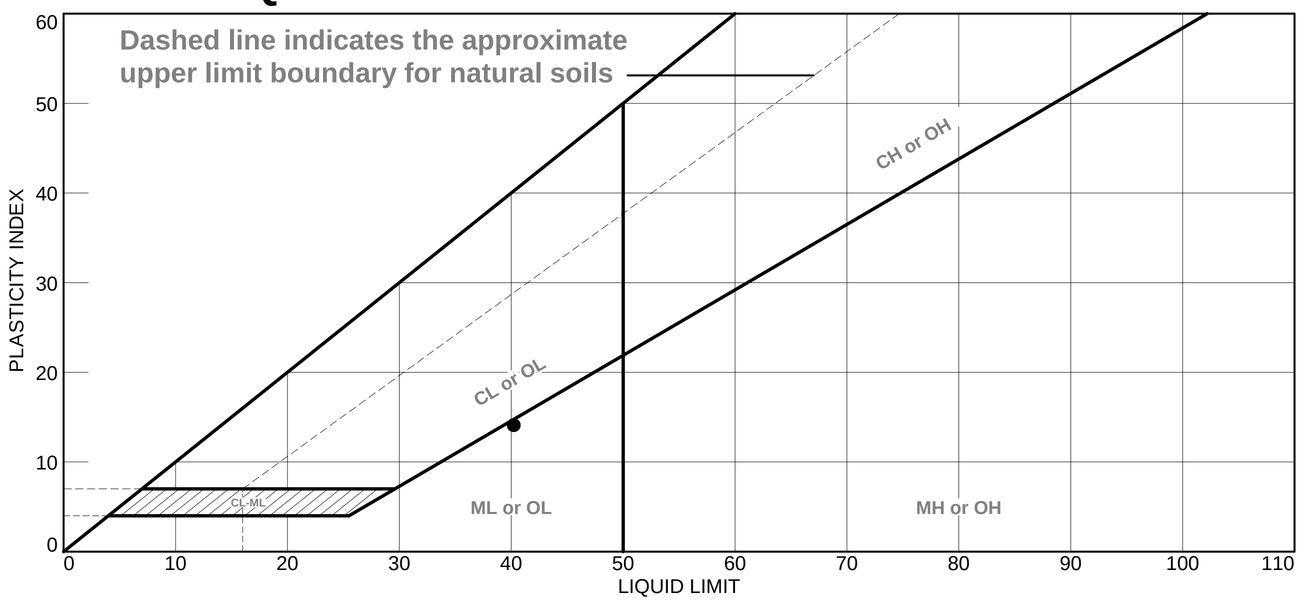
Location: TP-1
Sample Number: S19-309 **Depth:** 9' **Date Sampled:** 12/13/2019

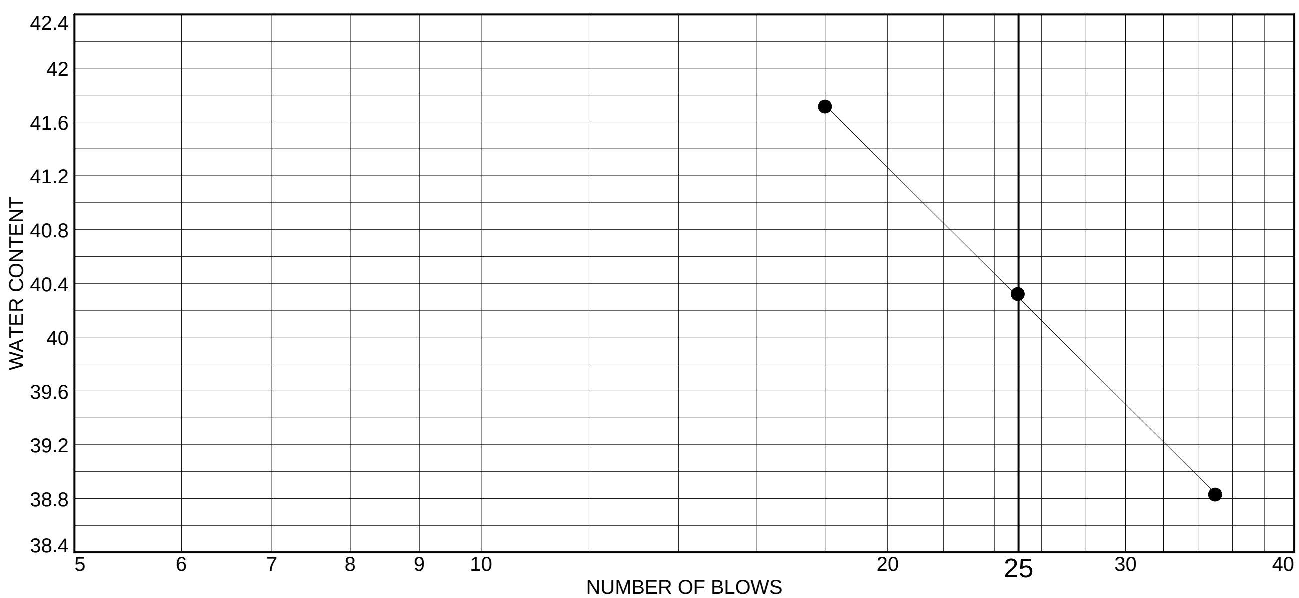
GEOPACIFIC ENGINEERING, INC.

Client: Hawkins Companies, LLC
Project: Fairfield Marriott Newberg

Project No: 19-5391 **Figure**

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Silt	40.3	26.3	14.0	99.9	99.4	ML

Project No. 19-5391 **Client:** Hawkins Companies, LLC

Project: Fairfield Marriott Newberg

Location: TP-1

Sample Number: S19-309 **Depth:** 9'

Remarks:

GEOPACIFIC ENGINEERING, INC.

Figure

Tested By: SJC

STATIC SETTLEMENT ANALYSIS

Settlement Analysis

Organization:	**GeoPacific Engineering, Inc.**
Project Name:	**19-5391, Fairfield Marriott, Ne**
Job #:	
Design by:	**BLC**
Date:	**12/23/2019**

Foundation Geometry, GWT & Loading

Units:	**English**
Footing Shape:	**Square**
Method:	**Schmertmann et al**

Variable	**Value**	**Variable**	**Value**
Footing Width	6.00 ft	Ground Water Depth	10.00 ft
Footing Thickness	3.00 ft	Soil Unit Weight	120.0 lb/ft^3
Footing Length	6.00 ft	Max. Depth	60.00 ft
Embedment Depth	1.00 ft	Time	20.00 years
Axial Load	80.00 k		

Time Rate Inputs

Thickness of Clay	10.00 ft	Drainage Condition	Single Drainage
Coef. of Consolidation	0.100 ft^2/day		

Geotechnical Properties

#	Material Type	USCS	Layer Thick, ft	Consistency	Soil Modulus Es lb/ft^2
1	User Defined		3.00 0 - 3		60000.000
2	User Defined		10.00 3 - 13		150000.000
3	User Defined		5.00 13 - 18		300000.000
4	User Defined		12.00 18 - 30		350000.000
5	User Defined		10.00 30 - 40		450000.000
6	User Defined		17.00 40 - 57		200000.000
7	User Defined		3.00 57 - 60		300000.000

Results

Applied Pressure, q:	2672.2 lb/ft^2	Drainage Height:	10.00 ft
Total Settlement, S:	1.81 in	Time for 99% Consol.:	4.88 years

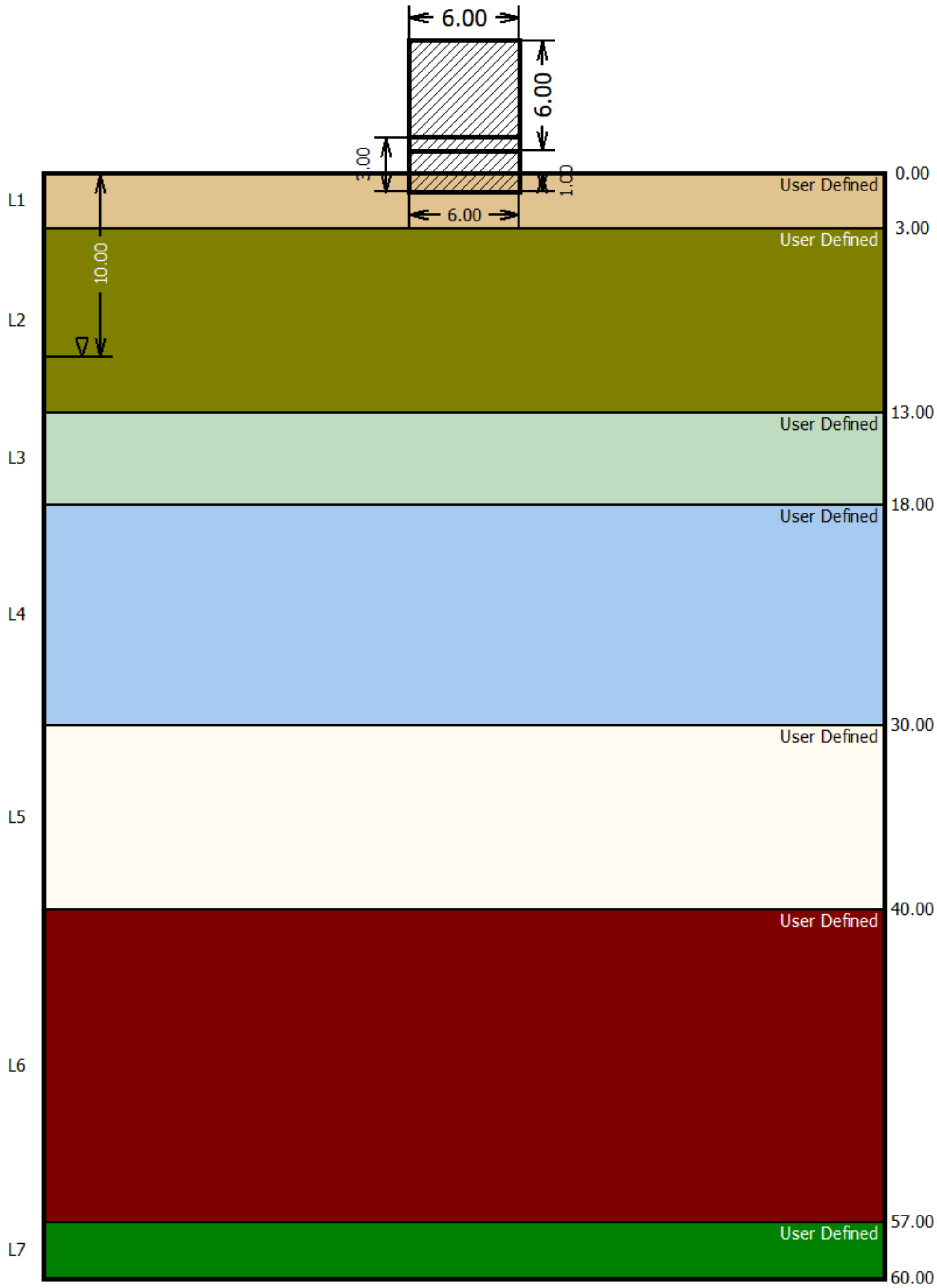


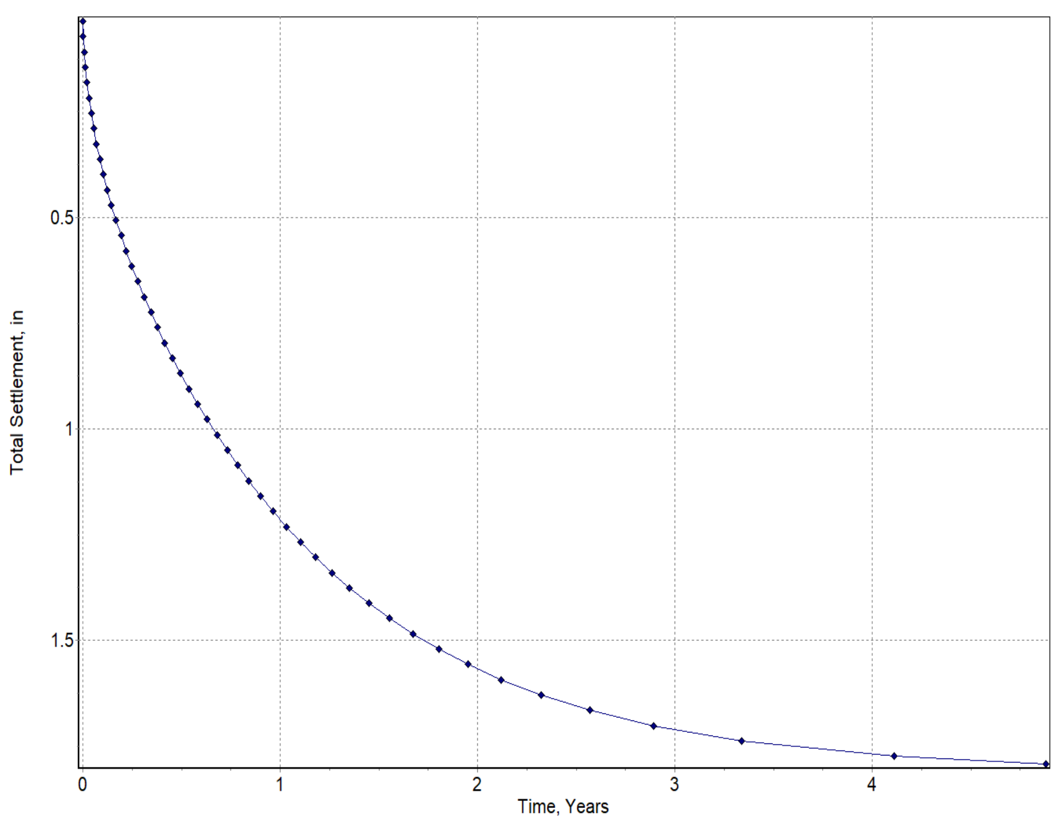
Fig. 1: Plan and Cross Section

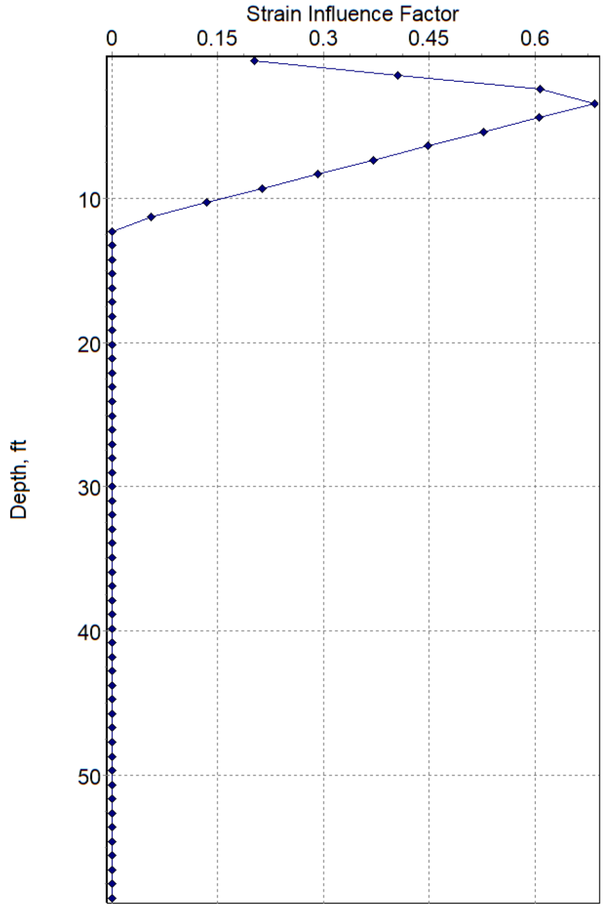
Table of Test Results

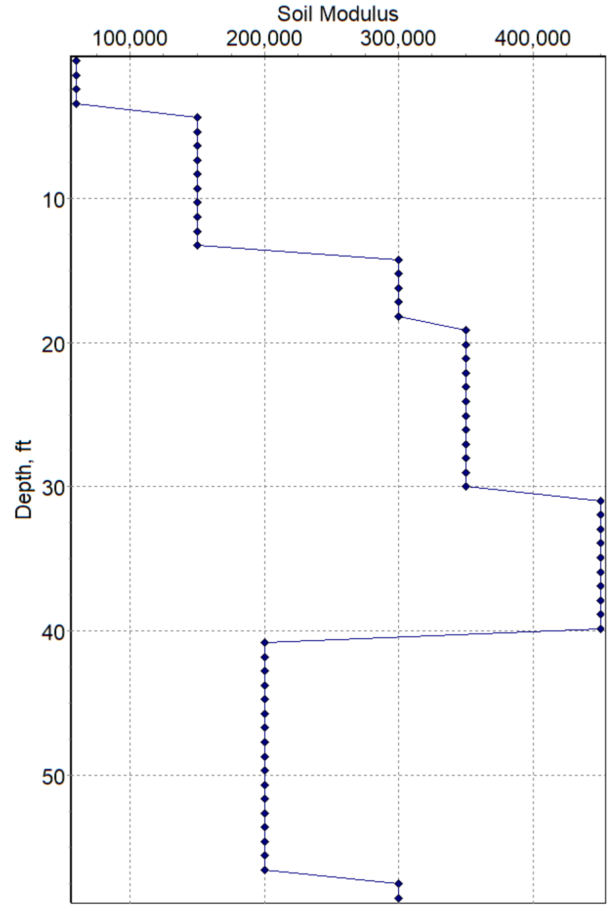
Node #	Depth (ft)	I epsilon	Strain (%)	Indiv. Sett. (in)	Tot. Sett. (in)
1	0.49	0.20	1.046	0.123	0.123
2	1.48	0.40	2.099	0.248	0.371
3	2.46	0.61	3.153	0.372	0.743
4	3.44	0.68	3.552	0.419	1.162
5	4.43	0.61	1.258	0.148	1.311
6	5.41	0.53	1.094	0.129	1.440
7	6.39	0.45	0.931	0.110	1.550
8	7.38	0.37	0.768	0.091	1.640
9	8.36	0.29	0.605	0.071	1.712
10	9.34	0.21	0.441	0.052	1.764
11	10.33	0.13	0.278	0.033	1.797
12	11.31	0.06	0.115	0.014	1.810
13	12.29	0.00	0.000	0.000	1.810
14	13.28	0.00	0.000	0.000	1.810
15	14.26	0.00	0.000	0.000	1.810
16	15.24	0.00	0.000	0.000	1.810
17	16.23	0.00	0.000	0.000	1.810
18	17.21	0.00	0.000	0.000	1.810
19	18.19	0.00	0.000	0.000	1.810
20	19.18	0.00	0.000	0.000	1.810
21	20.16	0.00	0.000	0.000	1.810
22	21.14	0.00	0.000	0.000	1.810
23	22.13	0.00	0.000	0.000	1.810
24	23.11	0.00	0.000	0.000	1.810
25	24.09	0.00	0.000	0.000	1.810
26	25.08	0.00	0.000	0.000	1.810
27	26.06	0.00	0.000	0.000	1.810
28	27.04	0.00	0.000	0.000	1.810
29	28.03	0.00	0.000	0.000	1.810
30	29.01	0.00	0.000	0.000	1.810
31	29.99	0.00	0.000	0.000	1.810
32	30.98	0.00	0.000	0.000	1.810
33	31.96	0.00	0.000	0.000	1.810
34	32.94	0.00	0.000	0.000	1.810
35	33.93	0.00	0.000	0.000	1.810
36	34.91	0.00	0.000	0.000	1.810
37	35.89	0.00	0.000	0.000	1.810
38	36.88	0.00	0.000	0.000	1.810
39	37.86	0.00	0.000	0.000	1.810
40	38.84	0.00	0.000	0.000	1.810
41	39.83	0.00	0.000	0.000	1.810
42	40.81	0.00	0.000	0.000	1.810
43	41.79	0.00	0.000	0.000	1.810
44	42.78	0.00	0.000	0.000	1.810
45	43.76	0.00	0.000	0.000	1.810
46	44.74	0.00	0.000	0.000	1.810
47	45.73	0.00	0.000	0.000	1.810
48	46.71	0.00	0.000	0.000	1.810
49	47.69	0.00	0.000	0.000	1.810
50	48.68	0.00	0.000	0.000	1.810
51	49.66	0.00	0.000	0.000	1.810
52	50.64	0.00	0.000	0.000	1.810
53	51.63	0.00	0.000	0.000	1.810
54	52.61	0.00	0.000	0.000	1.810
55	53.59	0.00	0.000	0.000	1.810
56	54.58	0.00	0.000	0.000	1.810
57	55.56	0.00	0.000	0.000	1.810
58	56.54	0.00	0.000	0.000	1.810
59	57.53	0.00	0.000	0.000	1.810
60	58.51	0.00	0.000	0.000	1.810

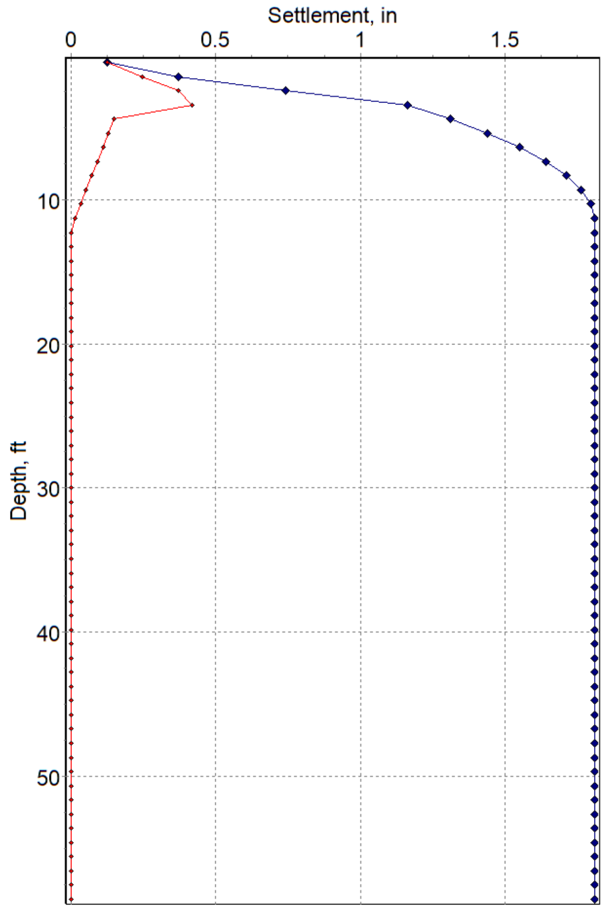
Table of Time Rate Results

Node #	Tot. Sett. (in)	Time Factor (Tv)	Time (years)
1	0.04	0.00030	0.00
2	0.07	0.00013	0.00
3	0.11	0.00283	0.01
4	0.14	0.00502	0.01
5	0.18	0.00785	0.02
6	0.22	0.01130	0.03
7	0.25	0.01540	0.04
8	0.29	0.02010	0.06
9	0.33	0.02540	0.07
10	0.36	0.03140	0.09
11	0.40	0.03800	0.10
12	0.43	0.04520	0.12
13	0.47	0.05310	0.15
14	0.51	0.06150	0.17
15	0.54	0.07070	0.19
16	0.58	0.08030	0.22
17	0.62	0.09070	0.25
18	0.65	0.10200	0.28
19	0.69	0.11300	0.31
20	0.72	0.12600	0.35
21	0.76	0.13800	0.38
22	0.80	0.15200	0.42
23	0.83	0.16600	0.45
24	0.87	0.18100	0.50
25	0.91	0.19700	0.54
26	0.94	0.21200	0.58
27	0.98	0.23000	0.63
28	1.01	0.24800	0.68
29	1.05	0.26700	0.73
30	1.09	0.28600	0.78
31	1.12	0.30700	0.84
32	1.16	0.32900	0.90
33	1.19	0.35200	0.96
34	1.23	0.37700	1.03
35	1.27	0.40300	1.10
36	1.30	0.43100	1.18
37	1.34	0.46100	1.26
38	1.38	0.49300	1.35
39	1.41	0.52900	1.45
40	1.45	0.56700	1.55
41	1.48	0.61000	1.67
42	1.52	0.65800	1.80
43	1.56	0.71200	1.95
44	1.59	0.77400	2.12
45	1.63	0.84800	2.32
46	1.67	0.93800	2.57
47	1.70	1.05500	2.89
48	1.74	1.21900	3.34
49	1.77	1.50000	4.11
50	1.79	1.78100	4.88









References:

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2. Geotechnical Engineering: Principles & Practices, 2nd edition, Coduto, Yeung & Kitch, 2010
3. Theories of Consolidation, R. L. Schiffman, 2001
4. Geotechnical Engineering & Soil Testing, Al-Khafaji & Andersland, 1995
5. Soil Mechanics, A.R. Jumikis, 1984
6. Settlement Analysis v2.0.2, SoilStructure Software, 2019

Settlement Analysis

Organization:	**GeoPacific Engineering, Inc.**
Project Name:	**19-5391, Fairfield Marriott, Ne**
Job #:	
Design by:	**BLC**
Date:	**12/23/2019**

Foundation Geometry, GWT & Loading

Units:	**English**
Footing Shape:	**Square**
Method:	**Schmertmann et al**

Variable	**Value**	**Variable**	**Value**
Footing Width	6.00 ft	Ground Water Depth	10.00 ft
Footing Thickness	3.00 ft	Soil Unit Weight	120.0 lb/ft^3
Footing Length	6.00 ft	Max. Depth	60.00 ft
Embedment Depth	1.00 ft	Time	20.00 years
Axial Load	80.00 k		

Time Rate Inputs

Thickness of Clay	10.00 ft	Drainage Condition	Single Drainage
Coef. of Consolidation	0.100 ft^2/day		

Geotechnical Properties

#	Material Type	USCS	Layer Thick, ft	Consistency	Soil Modulus Es lb/ft^2
1	Granular Soil		3.00 0 - 3		500000.000
2	User Defined		10.00 3 - 13		150000.000
3	User Defined		5.00 13 - 18		300000.000
4	User Defined		12.00 18 - 30		350000.000
5	User Defined		10.00 30 - 40		450000.000
6	User Defined		17.00 40 - 57		200000.000
7	User Defined		3.00 57 - 60		300000.000

Results

Applied Pressure, q:	2672.2 lb/ft^2	Drainage Height:	10.00 ft
Total Settlement, S:	0.79 in	Time for 99% Consol.:	4.88 years

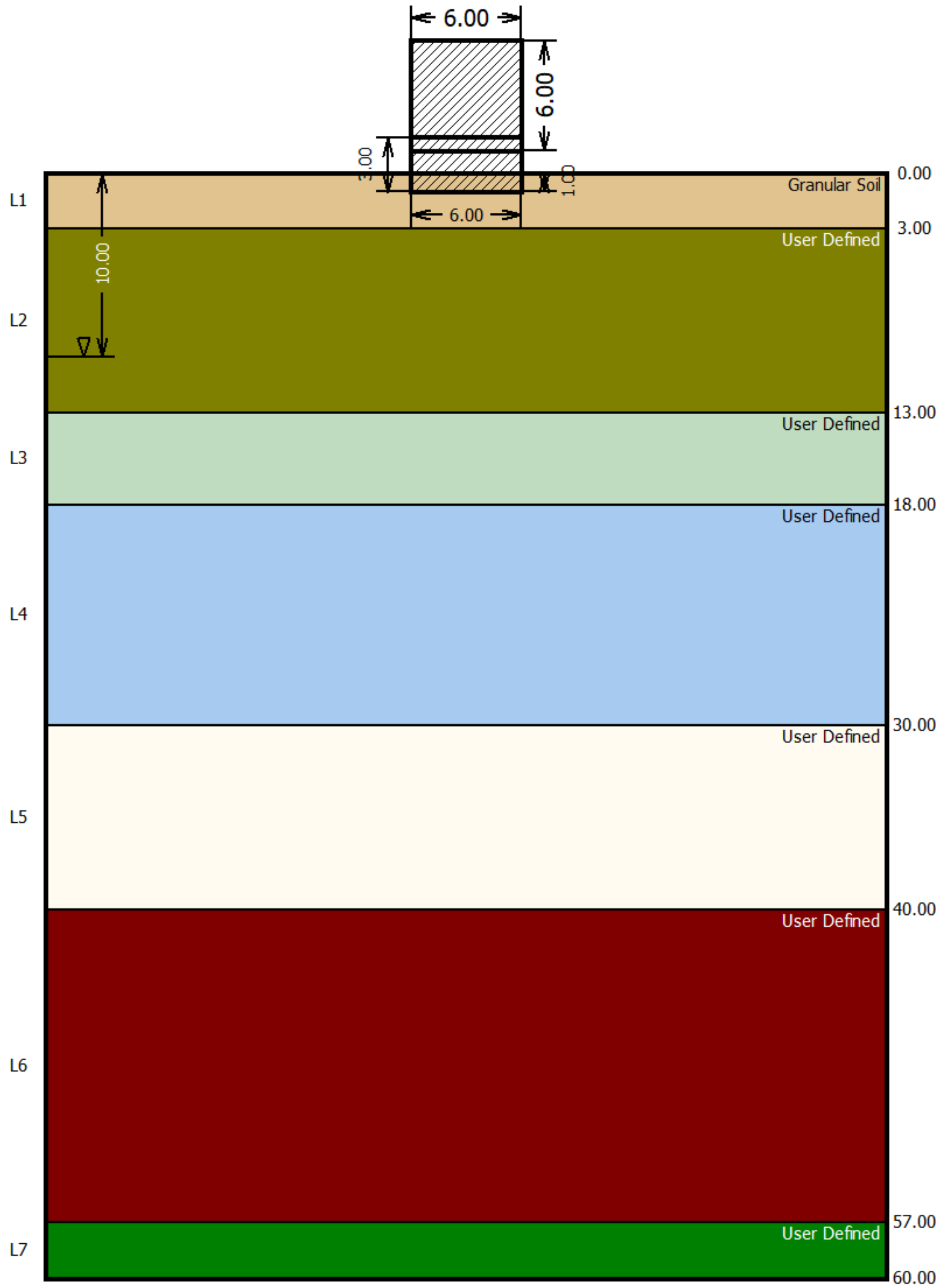


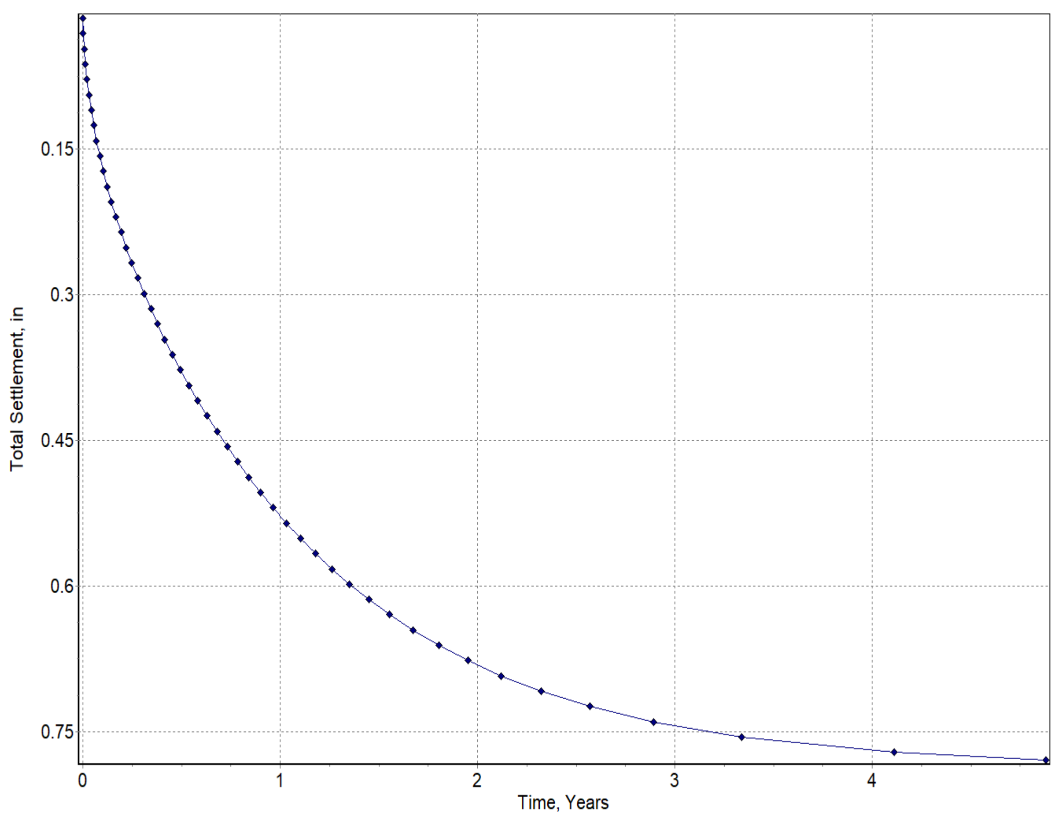
Fig. 1: Plan and Cross Section

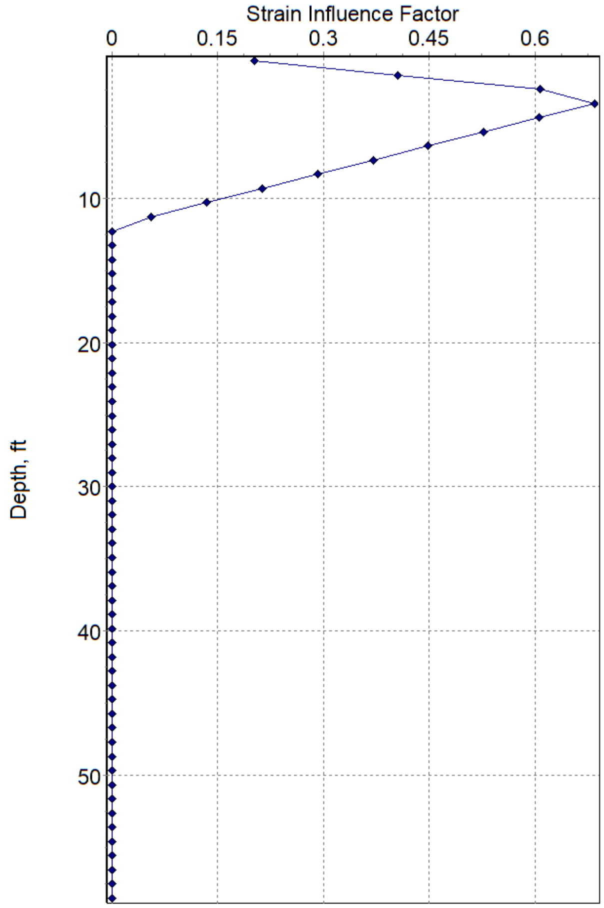
Table of Test Results

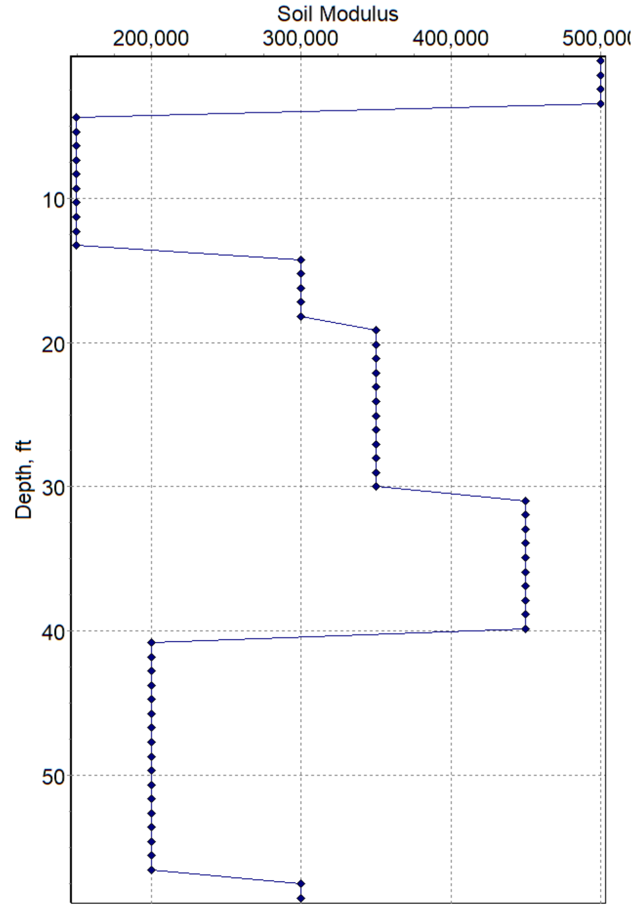
Node #	Depth (ft)	I epsilon	Strain (%)	Indiv. Sett. (in)	Tot. Sett. (in)
1	0.49	0.20	0.125	0.015	0.015
2	1.48	0.40	0.252	0.030	0.045
3	2.46	0.61	0.378	0.045	0.089
4	3.44	0.68	0.426	0.050	0.139
5	4.43	0.61	1.258	0.148	0.288
6	5.41	0.53	1.094	0.129	0.417
7	6.39	0.45	0.931	0.110	0.527
8	7.38	0.37	0.768	0.091	0.617
9	8.36	0.29	0.605	0.071	0.689
10	9.34	0.21	0.441	0.052	0.741
11	10.33	0.13	0.278	0.033	0.774
12	11.31	0.06	0.115	0.014	0.787
13	12.29	0.00	0.000	0.000	0.787
14	13.28	0.00	0.000	0.000	0.787
15	14.26	0.00	0.000	0.000	0.787
16	15.24	0.00	0.000	0.000	0.787
17	16.23	0.00	0.000	0.000	0.787
18	17.21	0.00	0.000	0.000	0.787
19	18.19	0.00	0.000	0.000	0.787
20	19.18	0.00	0.000	0.000	0.787
21	20.16	0.00	0.000	0.000	0.787
22	21.14	0.00	0.000	0.000	0.787
23	22.13	0.00	0.000	0.000	0.787
24	23.11	0.00	0.000	0.000	0.787
25	24.09	0.00	0.000	0.000	0.787
26	25.08	0.00	0.000	0.000	0.787
27	26.06	0.00	0.000	0.000	0.787
28	27.04	0.00	0.000	0.000	0.787
29	28.03	0.00	0.000	0.000	0.787
30	29.01	0.00	0.000	0.000	0.787
31	29.99	0.00	0.000	0.000	0.787
32	30.98	0.00	0.000	0.000	0.787
33	31.96	0.00	0.000	0.000	0.787
34	32.94	0.00	0.000	0.000	0.787
35	33.93	0.00	0.000	0.000	0.787
36	34.91	0.00	0.000	0.000	0.787
37	35.89	0.00	0.000	0.000	0.787
38	36.88	0.00	0.000	0.000	0.787
39	37.86	0.00	0.000	0.000	0.787
40	38.84	0.00	0.000	0.000	0.787
41	39.83	0.00	0.000	0.000	0.787
42	40.81	0.00	0.000	0.000	0.787
43	41.79	0.00	0.000	0.000	0.787
44	42.78	0.00	0.000	0.000	0.787
45	43.76	0.00	0.000	0.000	0.787
46	44.74	0.00	0.000	0.000	0.787
47	45.73	0.00	0.000	0.000	0.787
48	46.71	0.00	0.000	0.000	0.787
49	47.69	0.00	0.000	0.000	0.787
50	48.68	0.00	0.000	0.000	0.787
51	49.66	0.00	0.000	0.000	0.787
52	50.64	0.00	0.000	0.000	0.787
53	51.63	0.00	0.000	0.000	0.787
54	52.61	0.00	0.000	0.000	0.787
55	53.59	0.00	0.000	0.000	0.787
56	54.58	0.00	0.000	0.000	0.787
57	55.56	0.00	0.000	0.000	0.787
58	56.54	0.00	0.000	0.000	0.787
59	57.53	0.00	0.000	0.000	0.787
60	58.51	0.00	0.000	0.000	0.787

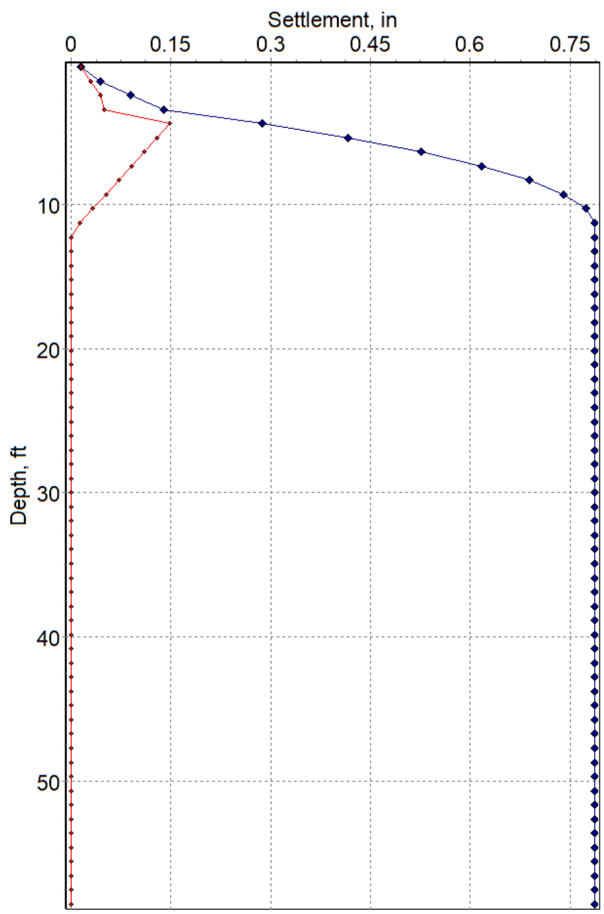
Table of Time Rate Results

Node #	Tot. Sett. (in)	Time Factor (Tv)	Time (years)	Node #	Tot. Sett. (in)	Time Factor (Tv)	Time (years)
1	0.02	0.00030	0.00	26	0.41	0.21200	0.58
2	0.03	0.00013	0.00	27	0.43	0.23000	0.63
3	0.05	0.00283	0.01	28	0.44	0.24800	0.68
4	0.06	0.00502	0.01	29	0.46	0.26700	0.73
5	0.08	0.00785	0.02	30	0.47	0.28600	0.78
6	0.09	0.01130	0.03	31	0.49	0.30700	0.84
7	0.11	0.01540	0.04	32	0.50	0.32900	0.90
8	0.13	0.02010	0.06	33	0.52	0.35200	0.96
9	0.14	0.02540	0.07	34	0.54	0.37700	1.03
10	0.16	0.03140	0.09	35	0.55	0.40300	1.10
11	0.17	0.03800	0.10	36	0.57	0.43100	1.18
12	0.19	0.04520	0.12	37	0.58	0.46100	1.26
13	0.20	0.05310	0.15	38	0.60	0.49300	1.35
14	0.22	0.06150	0.17	39	0.61	0.52900	1.45
15	0.24	0.07070	0.19	40	0.63	0.56700	1.55
16	0.25	0.08030	0.22	41	0.65	0.61000	1.67
17	0.27	0.09070	0.25	42	0.66	0.65800	1.80
18	0.28	0.10200	0.28	43	0.68	0.71200	1.95
19	0.30	0.11300	0.31	44	0.69	0.77400	2.12
20	0.31	0.12600	0.35	45	0.71	0.84800	2.32
21	0.33	0.13800	0.38	46	0.72	0.93800	2.57
22	0.35	0.15200	0.42	47	0.74	1.05500	2.89
23	0.36	0.16600	0.45	48	0.76	1.21900	3.34
24	0.38	0.18100	0.50	49	0.77	1.50000	4.11
25	0.39	0.19700	0.54	50	0.78	1.78100	4.88









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3. Theories of Consolidation, R. L. Schiffman, 2001
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            DARWin(tm) - Pavement Design

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             Computer Software Product

        -------------------------------------

         Flexible Structural Design Module

            GeoPacific Engineering, Inc.
                14835 SW 72nd Avenue
                 Portland, OR 97224

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Project Description
   19-5391, Fairfield Marriott, Newberg, Oregon, Flexible Pavement Design,
   Private Parking and Drive Areas, 20-Year Design Life

Flexible Structural Design Module Data
   18-kip ESALs Over Initial Performance Period: 75,000
                      Initial Serviceability: 4.2
                     Terminal Serviceability: 2.2
                       Reliability Level (%): 85
                  Overall Standard Deviation: .5
         Roadbed Soil Resilient Modulus (PSI): 6,000
                          Stage Construction: 1

                 Calculated Structural Number: 2.38

Specified Layer Design
                                       Layer: 1
                        Material Description: Asphaltic Concrete (A/C)
                   Structural Coefficient (Ai): .42
                     Drainage Coefficient (Mi): 1
                     Layer Thickness (Di) (in): 3.50
                           Calculated Layer SN: 1.47

                                       Layer: 2
                        Material Description: 3/4"-0 Crushed Aggregate
                   Structural Coefficient (Ai): .1
                     Drainage Coefficient (Mi): 1
                     Layer Thickness (Di) (in): 2.00
                           Calculated Layer SN: .20

                                       Layer: 3
                        Material Description: 1.5"-0 Crushed Aggregate
                   Structural Coefficient (Ai): .1
                     Drainage Coefficient (Mi): 1
                     Layer Thickness (Di) (in): 8.00
                           Calculated Layer SN: .80

                        Total Thickness (in): 13.50
                         Total Calculated SN: 2.47
```

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DARWin(tm) - Pavement Design

A Proprietary AASHTOWARE(tm)
Computer Software Product

Rigid Structural Design Module

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, OR 97224

Project Description
19-5391, Fairfield Marriott, Newberg, Oregon, Rigid Pavement Design, Private Parking/Drive Lanes

Rigid Structural Design Module Data

Pavement type:	JPCP
18-kip ESALs for initial performance period:	75,000
Initial Serviceability:	4.2
Terminal Serviceability:	2.2
28-day mean PCC Modulus of Rupture (psi):	650
28-day mean Elastic Modulus of Slab (psi):	3,500,000
Mean Effective k-value (pci):	33.03
Reliability Level (%):	85
Overall Standard Deviation:	.39
Load Transfer Coefficient:	3
Overall Drainage Coefficient:	1
Stage Construction:	3
Calculated Design Thickness (in):	4.90

Rigid Structural Design Joint Spacing

Joint Spacing (ft):	10.00

Additional Pavement Layers

Layer Number:	2
Material Type:	3/4"-0 Crushed Aggr
Description:	Leveling Course
Thickness (in):	2.00
Layer Number:	3
Material Type:	1.5"-0 Crushed Aggr
Description:	Base Course
Thickness (in):	6.00

Effective Modulus of Subgrade Reaction (k)

Base Type:	Crushed Aggregate
Base Thickness (in):	8
Depth to Bedrock (ft):	100
Projected Slab Thickness (in):	6
Loss of Support:	2
Period:	1
Roadbed Soil Resilient Modulus (PSI):	6,000
Base Elastic Modulus (PSI):	6,000
Effective Modulus of subgrade reaction (PCI):	33.03

SOIL LIQUEFACTION ANALYSIS

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Portland, Oregon 97224

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

TABLE OF CONTENTS

CPT-1 results

GeoPacific Engineering, Inc.
14835 SW 72nd Avenue
Portland, Oregon 97224
http://www.geopacificeng.com/

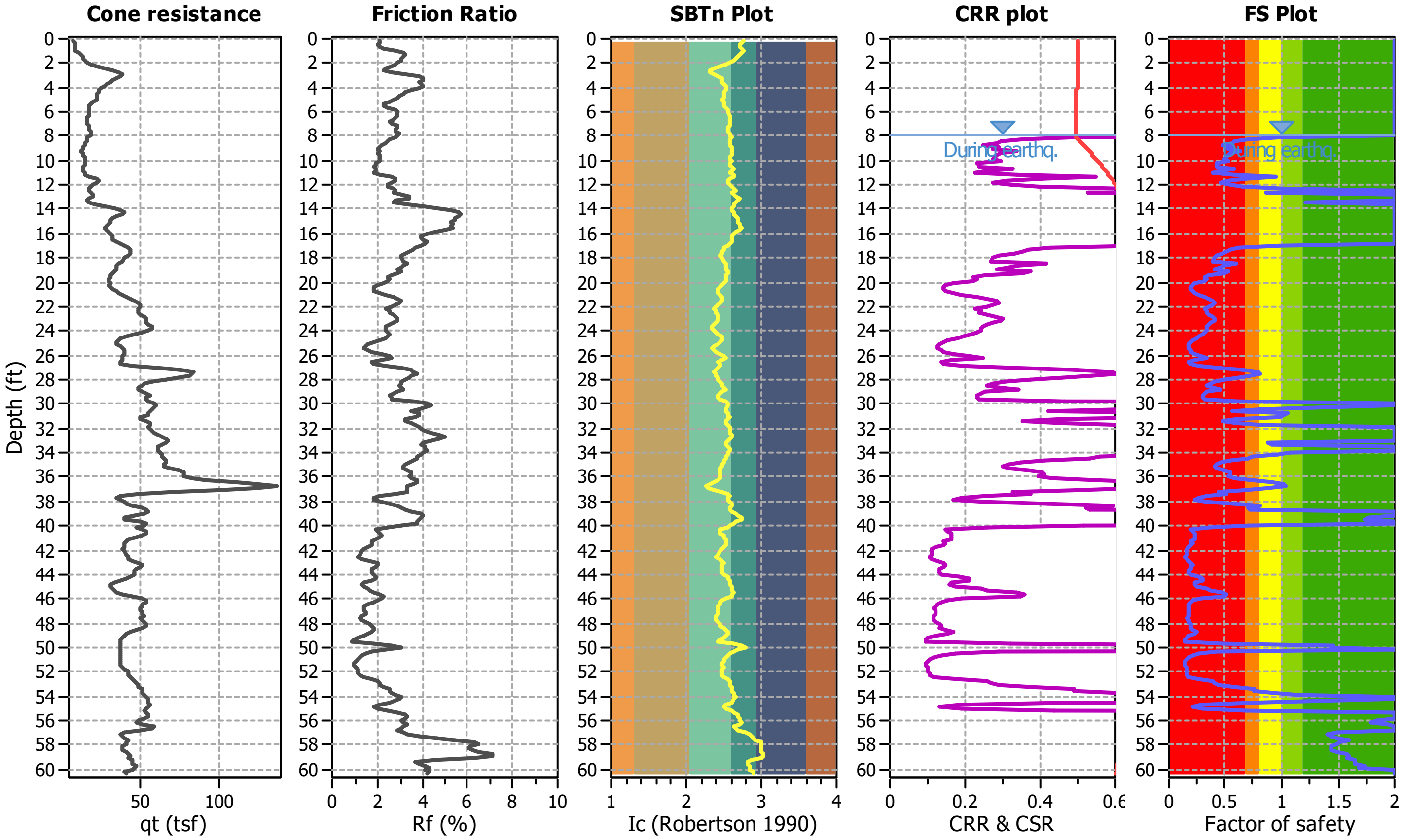
LIQUEFACTION ANALYSIS REPORT

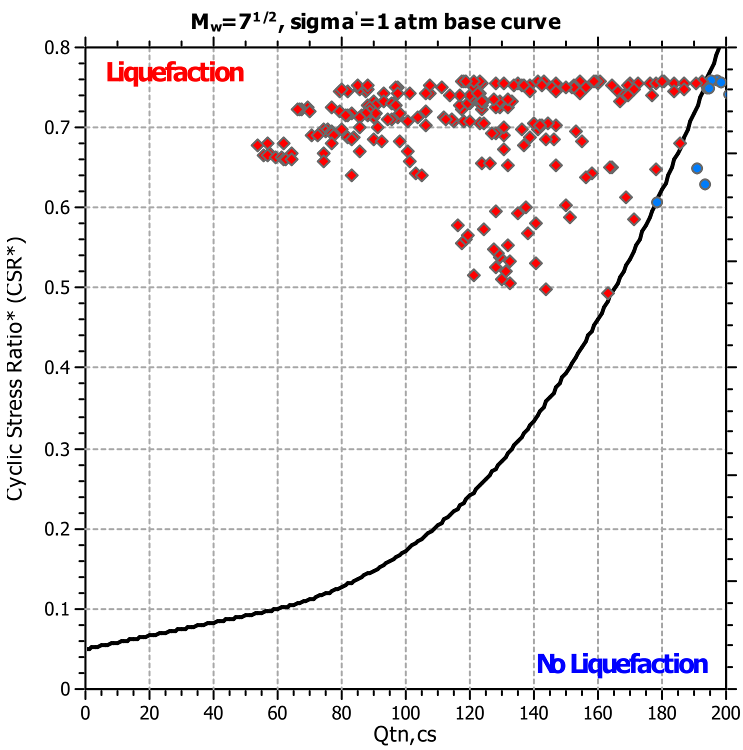
Project title : 19-5391, Fairfield Marriott **Location : 45.306859, -122.940625, Newberg, Oregon**

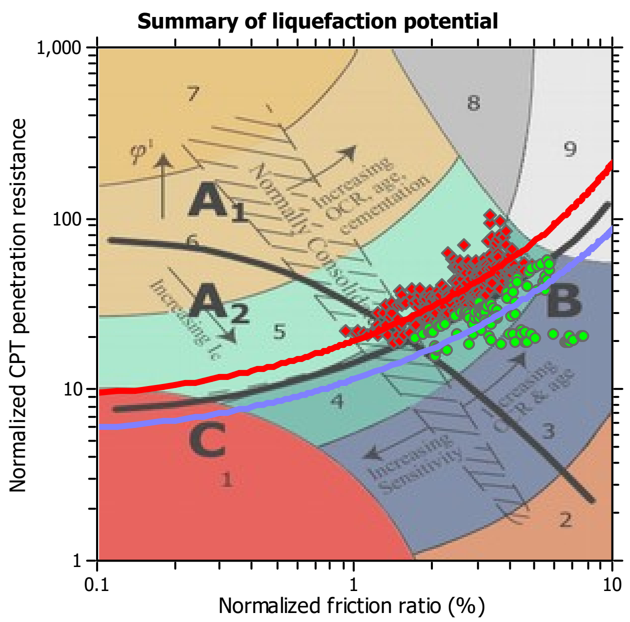
CPT file : CPT-1

Input parameters and analysis data

Analysis method:	Robertson (2009)	G.W.T. (in-situ):	10.00 ft	Use fill:	No	Clay like behavior applied:	All soils
Fines correction method:	Robertson (2009)	G.W.T. (earthq.):	8.00 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	9.10	Ic cut-off value:	2.60	Trans. detect. applied:	No	MSF method:	Method based
Peak ground acceleration:	0.47	Unit weight calculation:	Based on SBT	K_σ applied:	No		

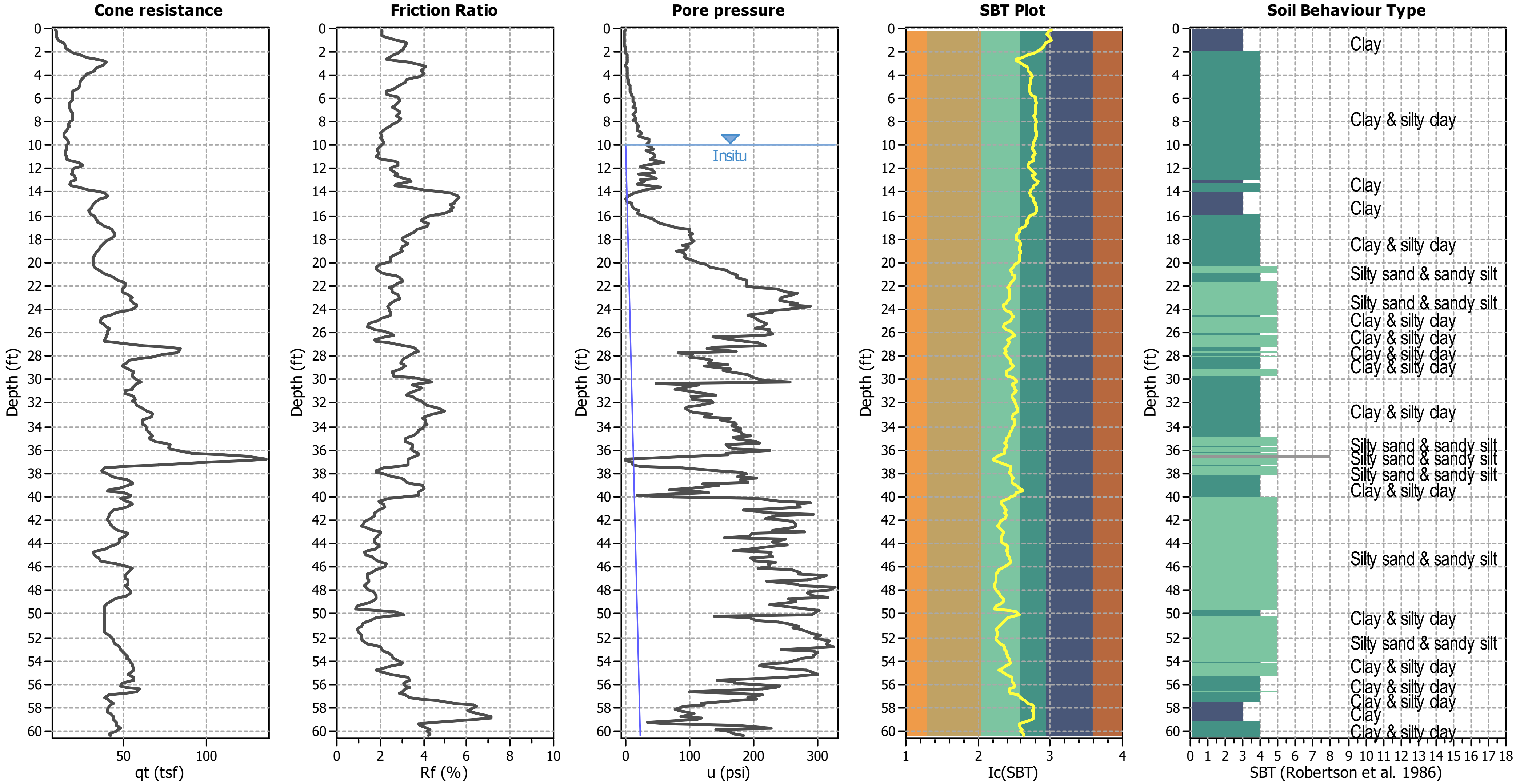






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots



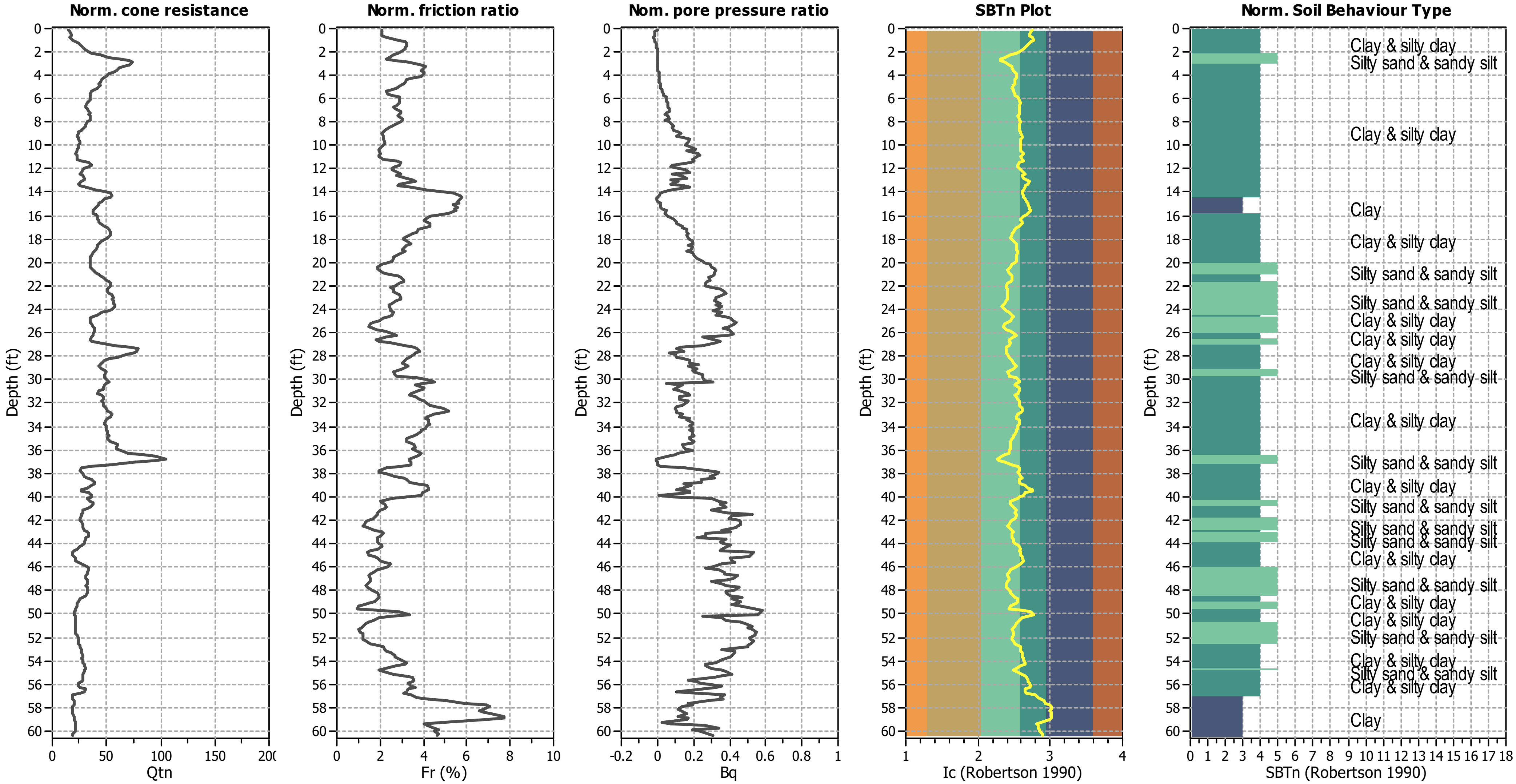
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	9.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.47	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBT legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

CPT basic interpretation plots (normalized)



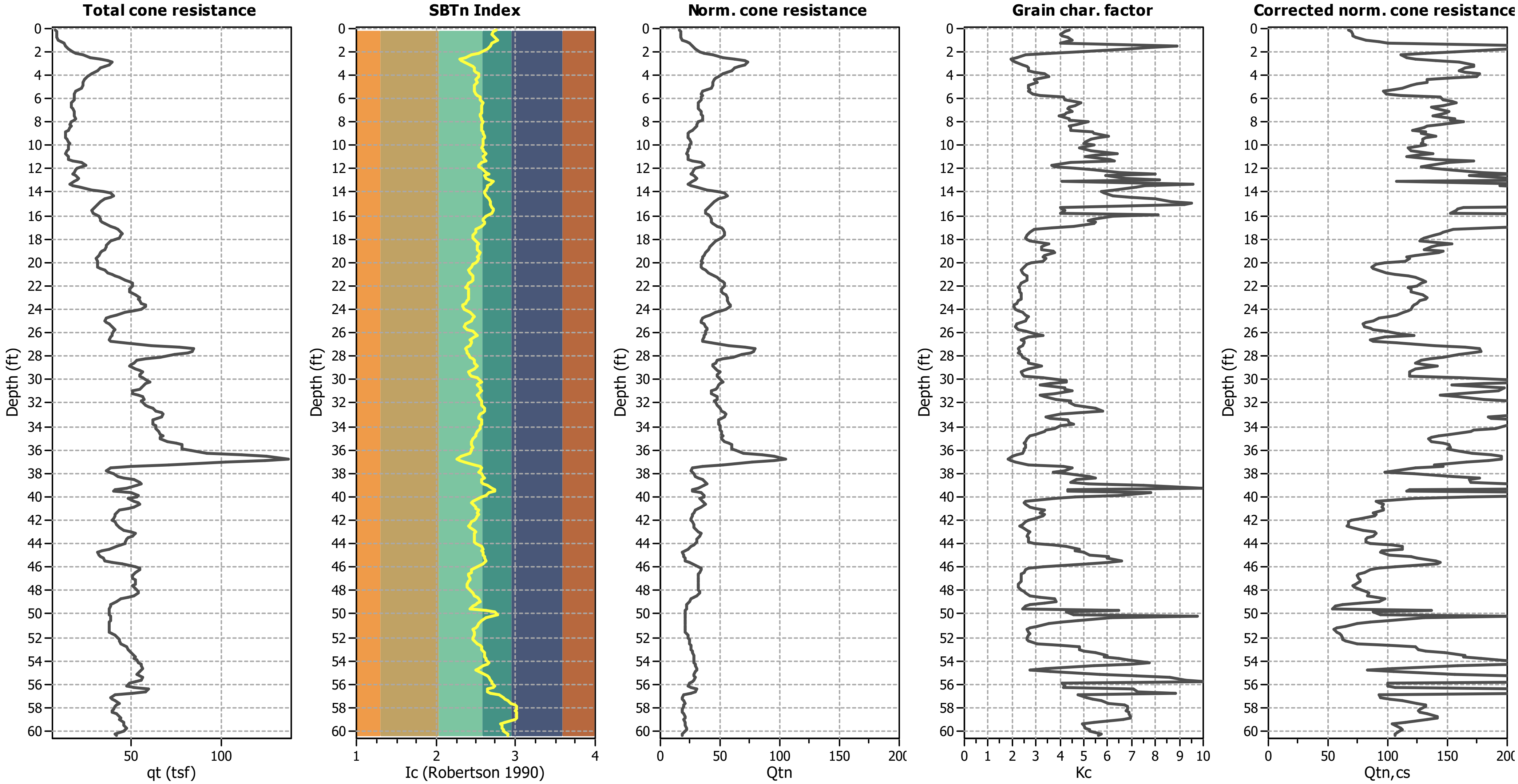
Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_{σ} applied:	No
Earthquake magnitude M_w:	9.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.47	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

SBTn legend

- 1. Sensitive fine grained
- 2. Organic material
- 3. Clay to silty clay
- 4. Clayey silt to silty
- 5. Silty sand to sandy silt
- 6. Clean sand to silty sand
- 7. Gravely sand to sand
- 8. Very stiff sand to
- 9. Very stiff fine grained

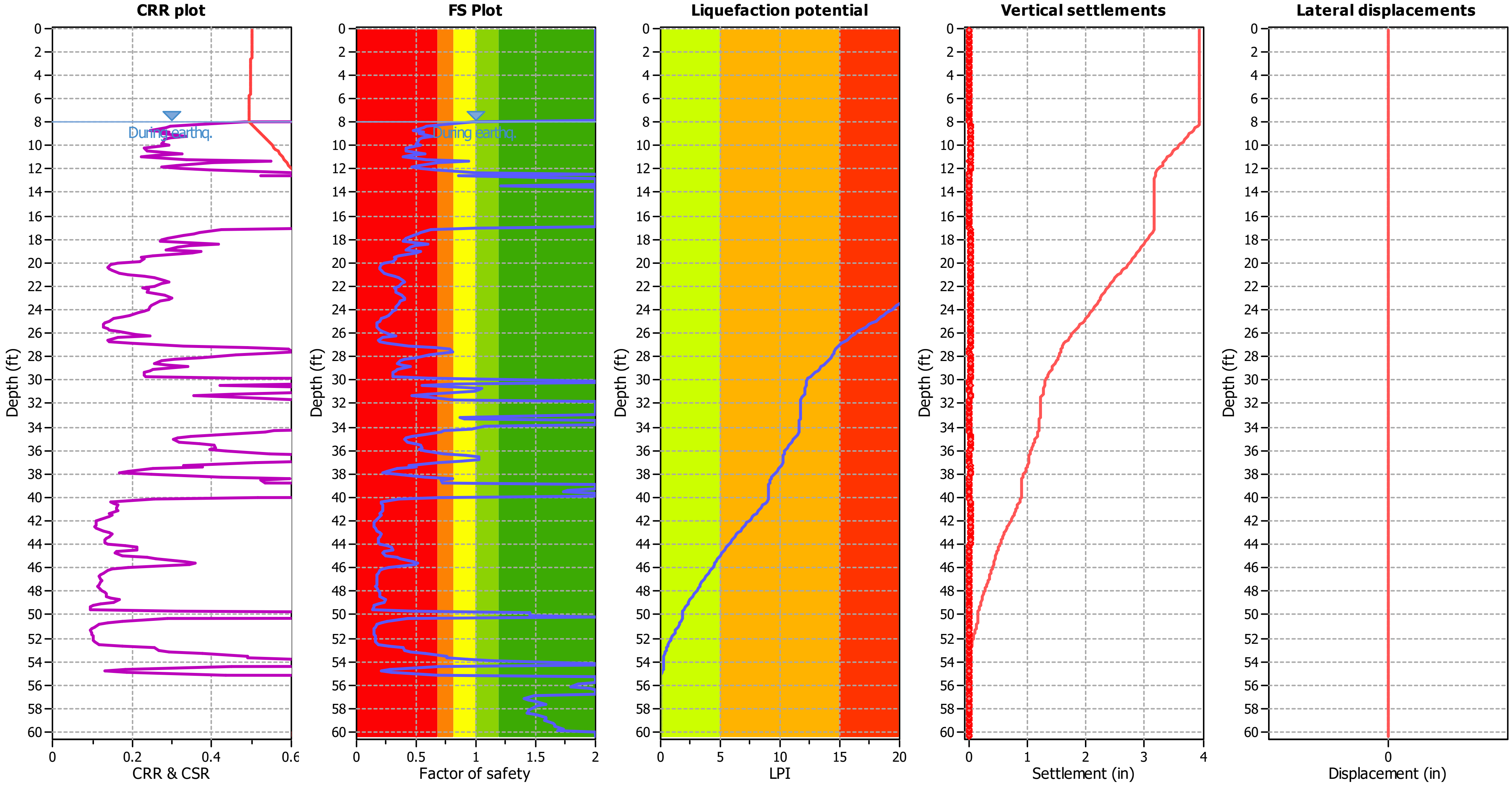
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	9.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.47	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	9.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.47	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

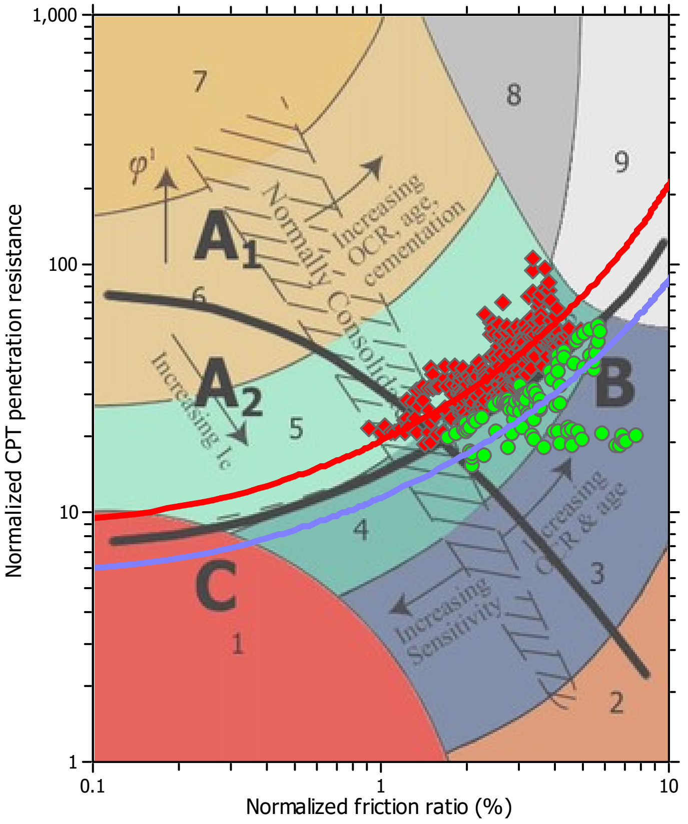
F.S. color scheme

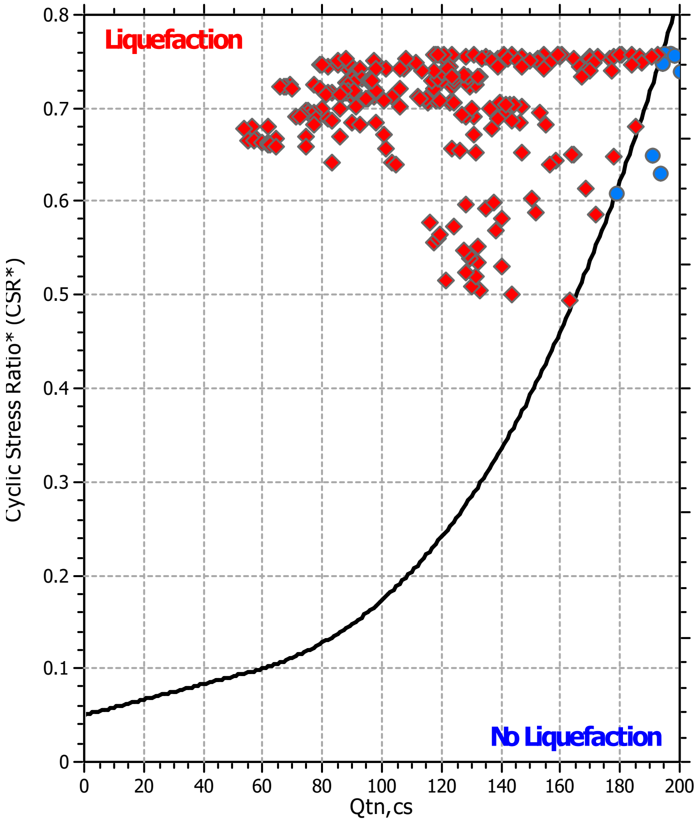
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

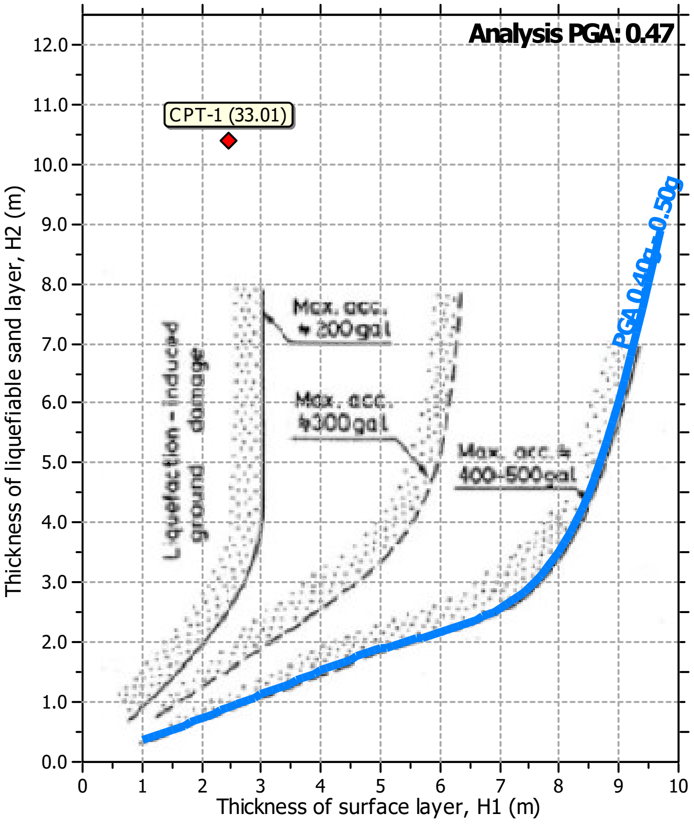
LPI color scheme

- Very high risk
- High risk
- Low risk

Liquefaction analysis summary plots



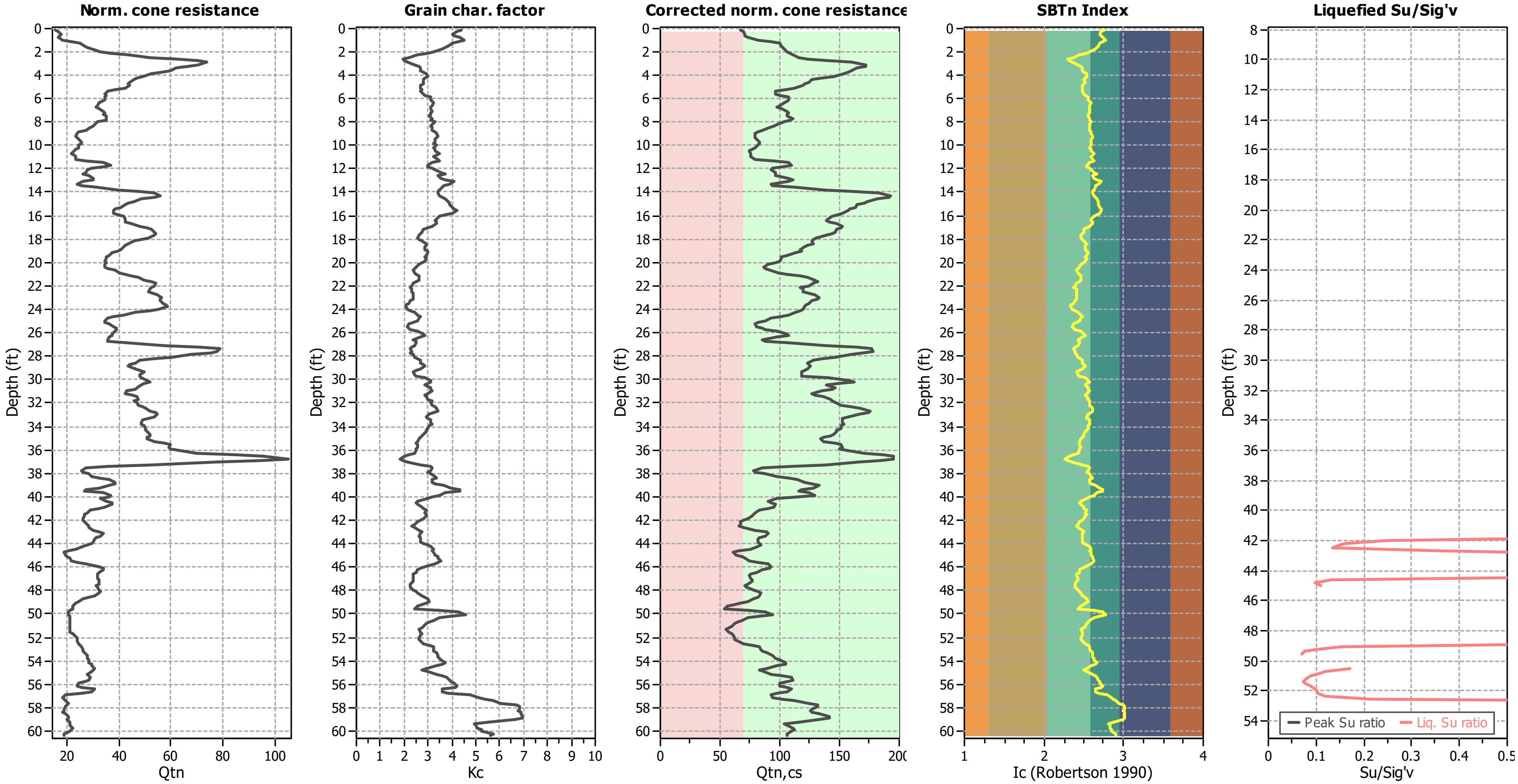




Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	9.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.47	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

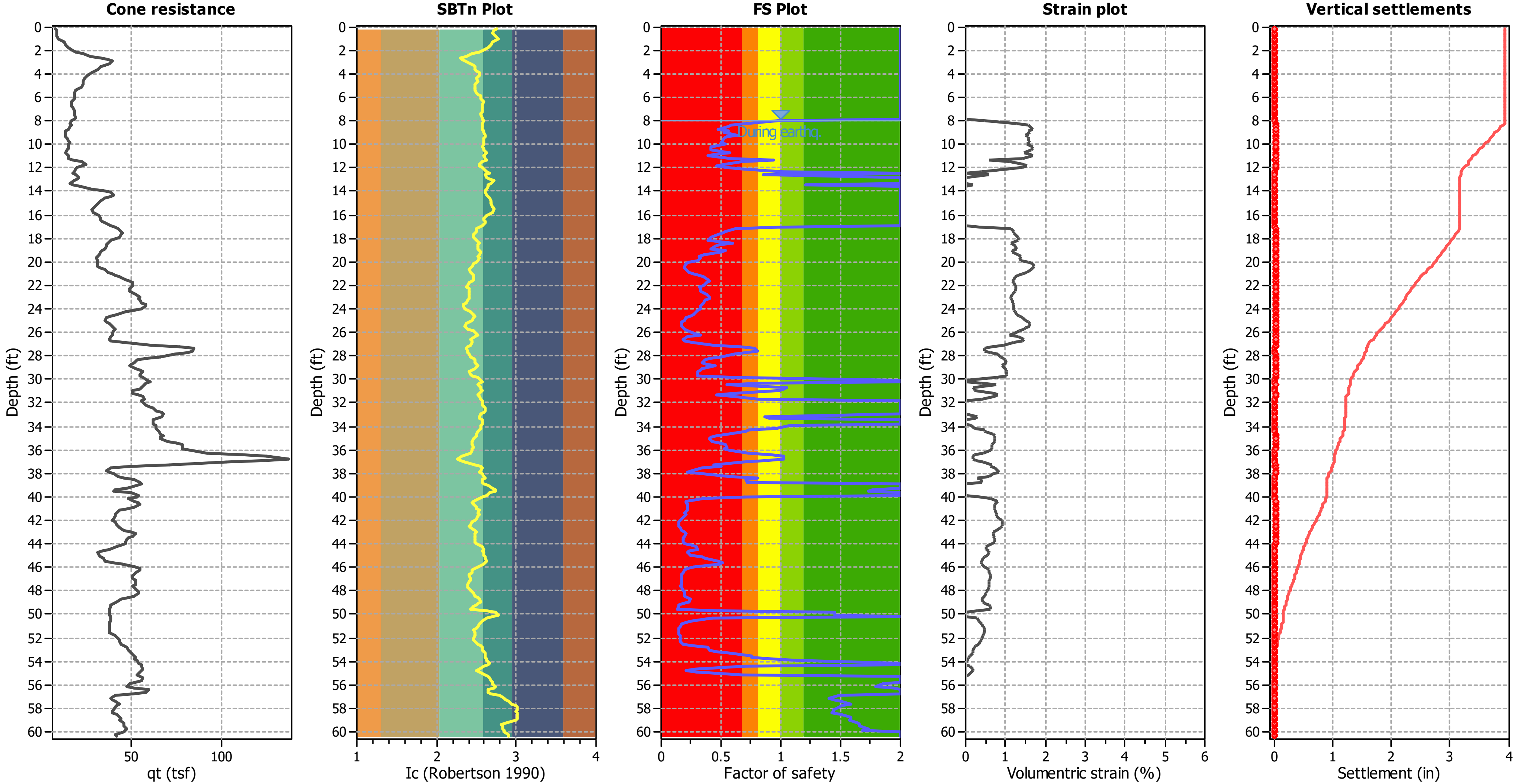
Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	Robertson (2009)	Depth to water table (erthq.):	8.00 ft	Fill weight:	N/A
Fines correction method:	Robertson (2009)	Average results interval:	3	Transition detect. applied:	No
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	No
Earthquake magnitude M_w:	9.10	Unit weight calculation:	Based on SBT	Clay like behavior applied:	All soils
Peak ground acceleration:	0.47	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	10.00 ft	Fill height:	N/A	Limit depth:	60.00 ft

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

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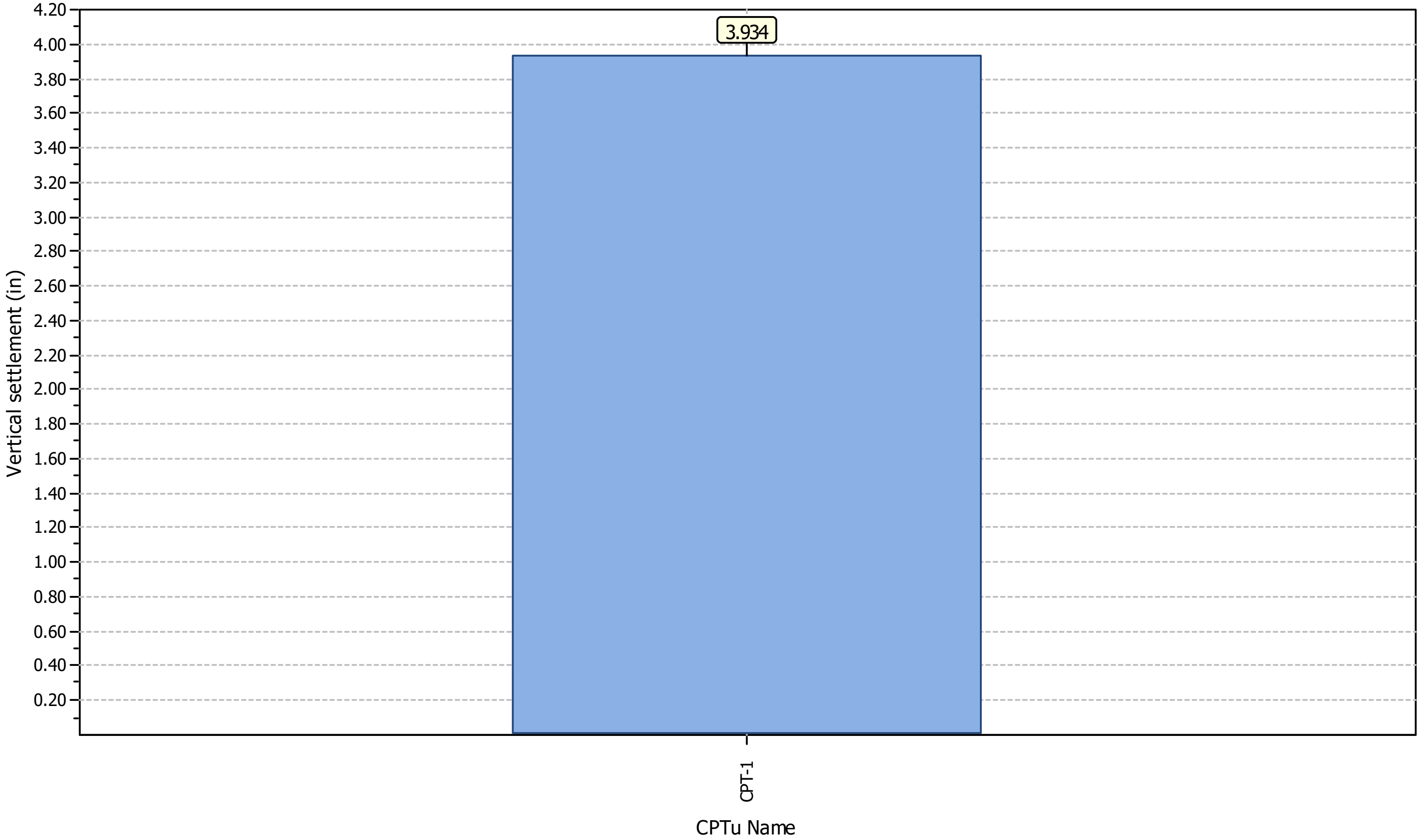
Project title : 19-5391, Fairfield Marriott

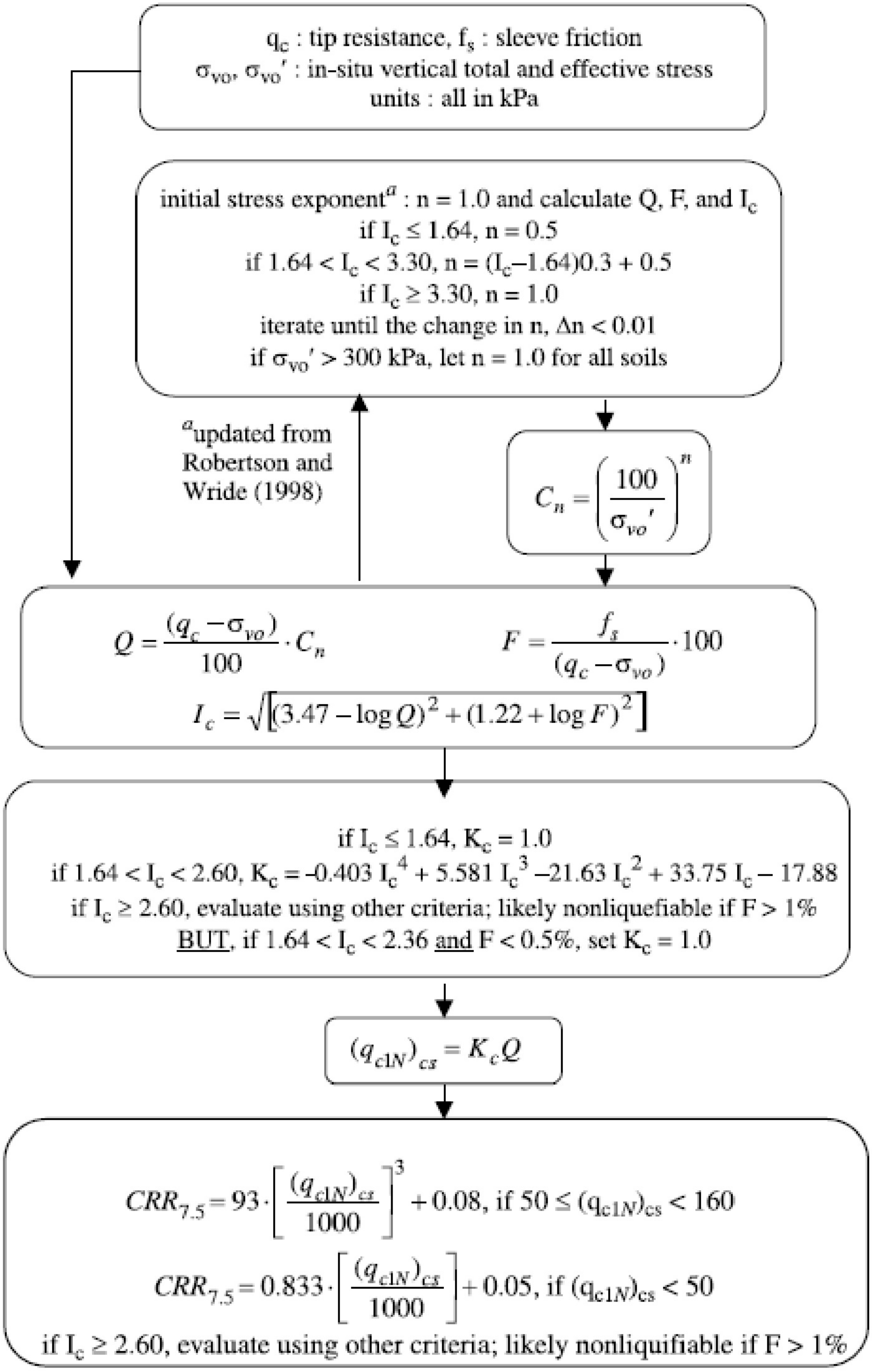
Location : 45.306859, -122.940625, Newberg, Oregon

Overall vertical settlements report



Procedure for the evaluation of soil liquefaction resistance, NCEER (1998)

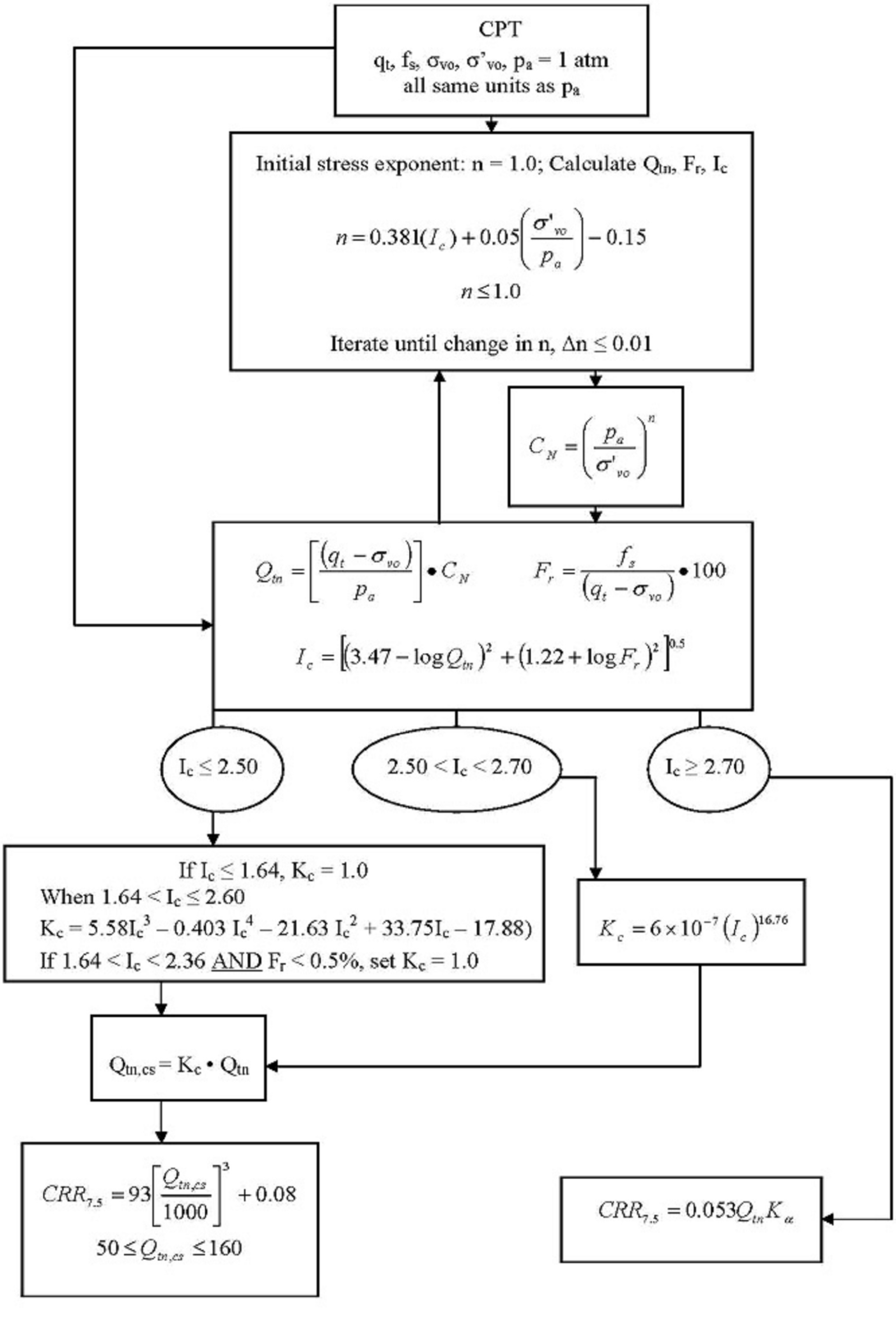
Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:



[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

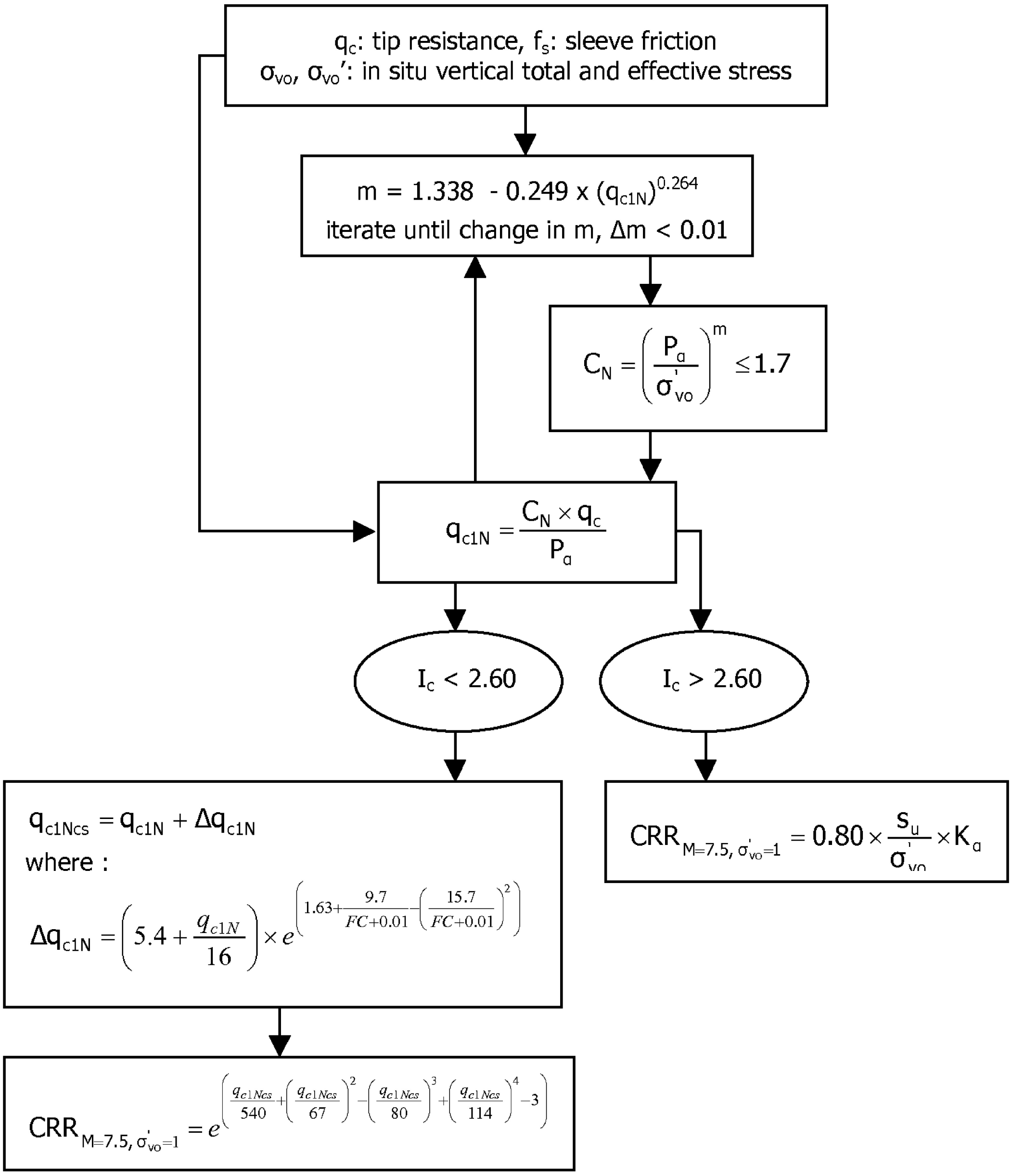
Procedure for the evaluation of soil liquefaction resistance (all soils), Robertson (2010)

Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. This procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:



[1] P.K. Robertson, 2009. "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009

Procedure for the evaluation of soil liquefaction resistance, Idriss & Boulanger (2008)



Procedure for the evaluation of soil liquefaction resistance (sandy soils), Moss et al. (2006)

CPT

q_t, f_s, I_c

$I_c < I_c$ cut-off

Initial estimate using raw tip measurements, friction ratio. Calculate $q_{t,1}$. Repeat until an acceptable convergence tolerance is achieved.

$$c = f_1 \cdot \left(\frac{R_f}{f_3}\right)^{f_2}$$

$$C_q = \left(\frac{P_a}{\sigma'_v}\right)^{c}$$

$$q_{t,1} = C_q \cdot q_t$$

$$CRR = \exp\left[\frac{q_{t,1}^{1.045} + q_{t,1}\cdot(0.110\cdot R_f) + (0.001\cdot R_f) + c\cdot(1 + 0.850\cdot R_f) - 0.848\cdot\ln(M_w) - 0.002\cdot\ln(\sigma'_v) - 20.923 + 1.632\cdot\Phi^{-1}(P_L)}{7.177}\right]$$

Procedure for the evaluation of soil liquefaction resistance, Boulanger & Idriss(2014)

$$FS_{liq} = \frac{CRR_{M=7.5,\sigma'_v=1atm}}{CSR_{M=7.5,\sigma'_v=1atm}}$$

$$CSR_{M=7.5,\sigma'_v=1atm} = 0.65\frac{\sigma_v}{\sigma'_v}\frac{a_{\max}}{g} r_d \frac{1}{MSF}\frac{1}{K_\sigma}$$

$$r_d = \exp\left[\alpha(z)+\beta(z)\cdot M\right]$$

$$\alpha(z) = -1.012 - 1.126\sin\left(\frac{z}{11.73}+5.133\right)$$

$$\beta(z) = 0.106 + 0.118\sin\left(\frac{z}{11.28}+5.142\right)$$

$$K_\sigma = 1 - C_\sigma \ln\left(\frac{\sigma'_v}{P_a}\right) \le 1.1$$

$$C_\sigma = \frac{1}{37.3 - 8.27\left(q_{c1Ncs}\right)^{0.264}} \le 0.3$$

$$MSF = 1 + \left(MSF_{max} - 1\right)\left(8.64\exp\left(\frac{-M}{4}\right) - 1.325\right)$$

$$MSF_{max} = 1.09 + \left(\frac{q_{c1Ncs}}{180}\right)^3 \le 2.2$$

σ'_v *and* σ_v *at start of earthquake shaking*

$$CRR_{M=7.5,\sigma'_v=1atm} = \exp\left(\frac{q_{c1Ncs}}{113} + \left(\frac{q_{c1Ncs}}{1000}\right)^2 - \left(\frac{q_{c1Ncs}}{140}\right)^3 + \left(\frac{q_{c1Ncs}}{137}\right)^4 - 2.80\right)$$

$$q_{c1Ncs} = q_{c1N} + \Delta q_{c1N}$$

$$\Delta q_{c1N} = \left(11.9 + \frac{q_{c1N}}{14.6}\right)\exp\left(1.63 - \frac{9.7}{FC+2} - \left(\frac{15.7}{FC+2}\right)^2\right)$$

$$q_{c1N} = C_{N\sigma}\frac{q_c}{P_a}$$

$$C_{N\sigma} = \left(\frac{P_a}{\sigma'_v}\right)^m \le 1.7$$

$$m = 1.338 - 0.249\left(q_{c1Ncs}\right)^{0.264} \quad with \quad 0.264 \le m \le 0.782$$

$$FC = 80\left(I_C + C_{FC}\right) - 137 \quad with \quad 0\% \le FC \le 100\%$$

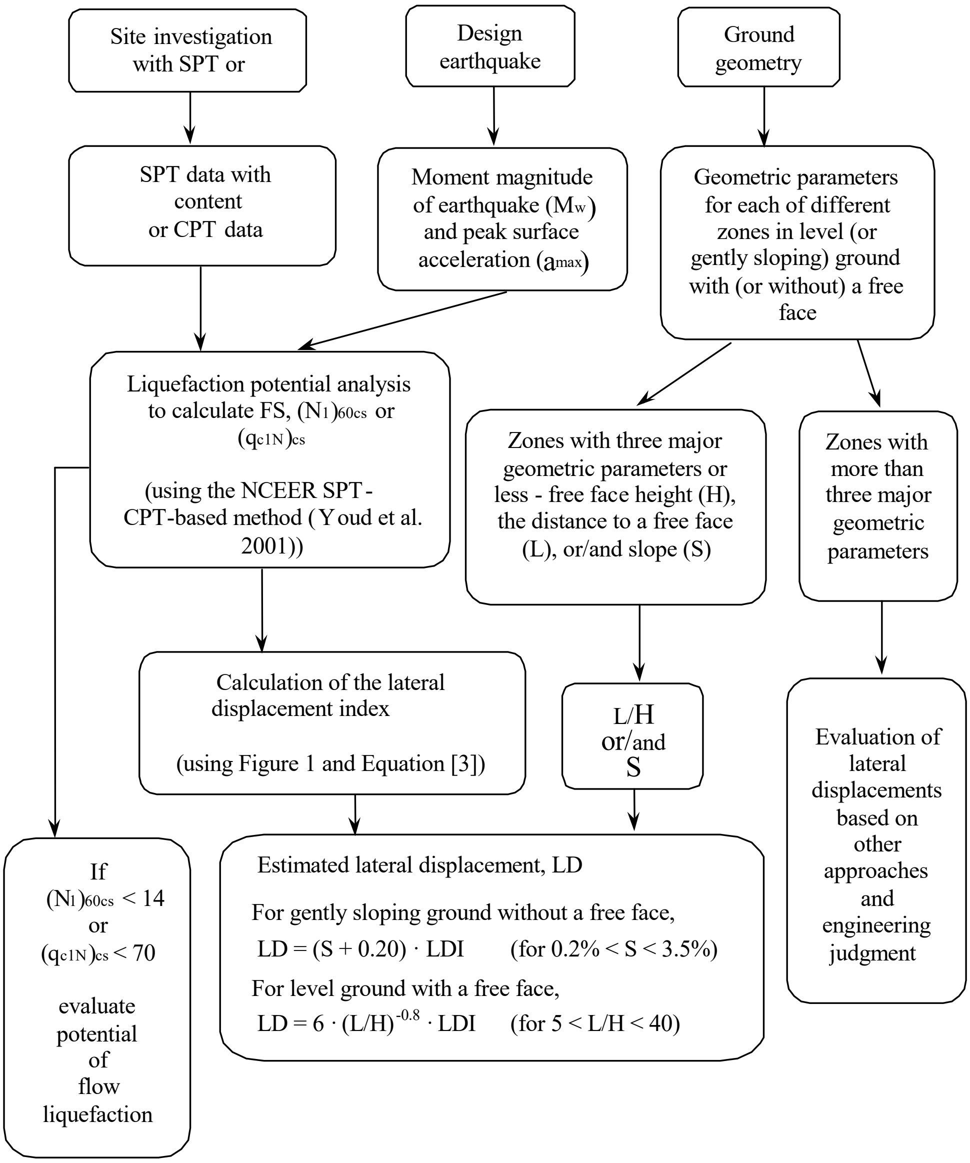
$$I_c = \left[\left(3.47 - \log(Q)\right)^2 + \left(1.22 + \log(F)\right)^2\right]^{0.5}$$

$$Q = \left(\frac{q_c - \sigma_{vc}}{P_a}\right)\left(\frac{P_a}{\sigma'_{vc}}\right)^n \; with\; 0.5 \le n \le 1.0 \; per\; Robertson \,\&\, Wride\,(1998)$$

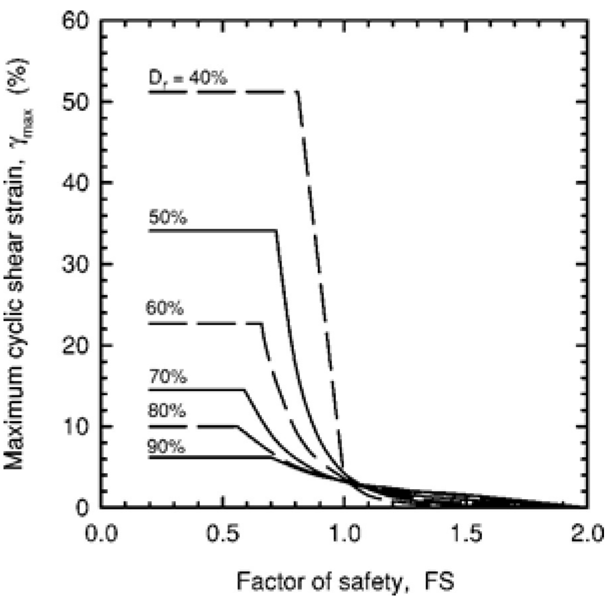
$$F = \left(\frac{f_s}{q_c - \sigma_{vc}}\right)\cdot 100\%$$

σ'_v *at time of CPT sounding*

Procedure for the evaluation of liquefaction-induced lateral spreading displacements



[1] Flow chart illustrating major steps in estimating liquefaction-induced lateral spreading displacements using the proposed approach



[1] **Figure 1**

$$LDI = \int_0^{Z_{max}} \gamma_{max} \, dz$$

[1] **Equation [3]**

[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

Procedure for the estimation of seismic induced settlements in dry sands

Average shear stress, τ_{av}

$$\tau_{av} = CSR^{*} \cdot \sigma_{v0}' = 0.65 \cdot \frac{a_{max}}{g} \cdot \sigma_{v0} \cdot \tau_a$$

Estimate small shear strain modulus, G_0

$$G_0 = 0.0188 \cdot \left[10^{(0.55 I_c - 1.68)}\right] \cdot (q_t - \sigma_v)$$

Estimate shear strain amplitude, γ

(based on Pradel (1998))

$$\gamma = \left[\frac{1 + \alpha \cdot e^{bR}}{1 + \alpha}\right] \cdot R \cdot 100\ (\%)$$

$$R = \frac{\tau_{av}}{G_0} \text{ (Note } \tau_{av} \text{ and } G_0 \text{ same units)}$$

$$\alpha = 0.0389 \cdot \left(\frac{\sigma_v}{p_a}\right) + 0.124$$

$$b = 6400 \cdot \left(\frac{\sigma_v}{p_a}\right)$$

Estimate volumetric strain in 15 cycles

$$\varepsilon_{vol(15)} = \gamma \cdot \left[\frac{(N_1)_{60cs}}{20}\right]^{-1.20}$$

$$(N_1)_{60cs} = \frac{Q_{tn,cs}}{8.5 \cdot \left[1 - \frac{I_c}{4.6}\right]}$$

Volumetric strain in design earthquake

$$\Sigma_{vol} = \varepsilon_{vol(15)} \cdot \left[\frac{N_c}{15}\right]^{0.45}$$

$$N_c = (M - 4)^{2.17}$$

Seismic settlement, s

$$s = 2 \cdot \int_0^{GWT} \Sigma_{vol} \cdot dz$$

Robertson, P.K. and Lisheng, S., 2010, "Estimation of seismic compression in dry soils using the CPT" FIFTH INTERNATIONAL CONFERENCE ON RECENT ADVANCES IN GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS, Symposium in honor of professor I. M. Idriss, San Diego, CA

Liquefaction Potential Index (LPI) calculation procedure

Calculation of the Liquefaction Potential Index (LPI) is used to interpret the liquefaction assessment calculations in terms of severity over depth. The calculation procedure is based on the methology developed by Iwasaki (1982) and is adopted by AFPS.

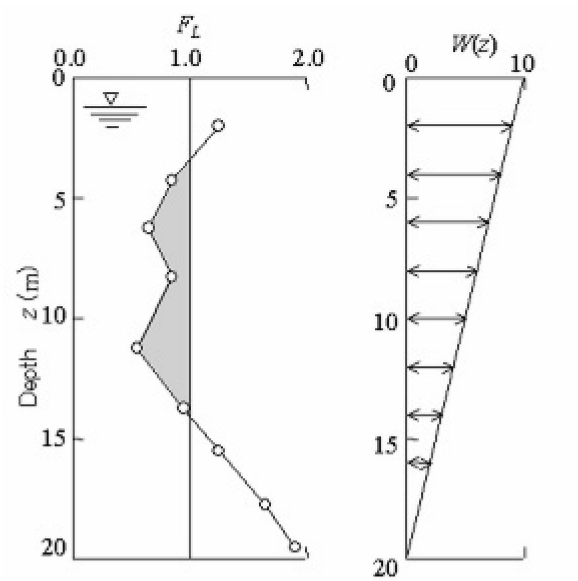
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

$$\textbf{LPI} = \int_0^{20} (10 - 0{,}5_z) \times F_L \times d_z$$

where:
F_L = 1 - F.S. when F.S. less than 1
F_L = 0 when F.S. greater than 1
z depth of measurment in meters

Values of LPI range between zero (0) when no test point is characterized as liquefiable and 100 when all points are characterized as susceptible to liquefaction. Iwasaki proposed four (4) discrete categories based on the numeric value of LPI:

- LPI = 0 : Liquefaction risk is very low
- 0 < LPI <= 5 : Liquefaction risk is low
- 5 < LPI <= 15 : Liquefaction risk is high
- LPI > 15 : Liquefaction risk is very high



Graphical presentation of the LPI calculation procedure

Shear-Induced Building Settlement (Ds) calculation procedure

The shear-induced building settlement (Ds) due to liquefaction below the building can be estimated using the relationship developed by Bray and Macedo (2017):

$$Ln(Ds) = c1 + c2 * LBS + 0.58 * Ln\left(Tanh\left(\frac{HL}{6}\right)\right) +$$
$$4.59 * Ln(Q) - 0.42 * Ln(Q)^2 - 0.02 * B +$$
$$0.84 * Ln(CAVdp) + 0.41 * Ln(Sa1) + \varepsilon$$

where Ds is in the units of mm, c1= -8.35 and c2= 0.072 for LBS ≤ 16, and c1= -7.48 and c2= 0.014 otherwise. Q is the building contact pressure in units of kPa, HL is the cumulative thickness of the liquefiable layers in the units of m, B is the building width in the units of m, CAVdp is a standardized version of the cumulative absolute velocity in the units of g-s, Sa1 is 5%-damped pseudo-acceleration response spectral value at a period of 1 s in the units of g, and ε is a normal random variable with zero mean and 0.50 standard deviation in Ln units. The liquefaction-induced building settlement index (LBS) is:

$$LBS = \sum W * \frac{\varepsilon_{shear}}{z} dz$$

where z (m) is the depth measured from the ground surface > 0, W is a foundation-weighting factor wherein W = 0.0 for z less than Df, which is the embedment depth of the foundation, and W = 1.0 otherwise. The shear strain parameter (ε_shear) is the liquefaction-induced free-field shear strain (in %) estimated using Zhang et al. (2004). It is calculated based on the estimated Dr of the liquefied soil layer and the calculated safety factor against liquefaction triggering (FSL).

References

- Lunne, T., Robertson, P.K., and Powell, J.J.M 1997. Cone penetration testing in geotechnical practice, E & FN Spon Routledge, 352 p, ISBN 0-7514-0393-8.

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SITE RESEARCH

Hazards by Location

Search Information

Coordinates: 45.307545, -122.938352

Elevation: 225 ft

Timestamp: 2019-12-23T19:13:52.346Z

Hazard Type: Seismic

Reference Document: NEHRP-2015

Risk Category: II

Site Class: D

Basic Parameters

Name	Value	Description
S_S	0.848	MCE_R ground motion (period=0.2s)
S_1	0.409	MCE_R ground motion (period=1.0s)
S_{MS}	0.984	Site-modified spectral acceleration value
S_{M1}	* 0.773	Site-modified spectral acceleration value
S_{DS}	0.656	Numeric seismic design value at 0.2s SA
S_{D1}	* 0.515	Numeric seismic design value at 1.0s SA

* See Section 11.4.7

Additional Information

Name	Value	Description
SDC	* D	Seismic design category
F_a	1.161	Site amplification factor at 0.2s
F_v	* 1.891	Site amplification factor at 1.0s
CR_S	0.883	Coefficient of risk (0.2s)
CR_1	0.867	Coefficient of risk (1.0s)
PGA	0.39	MCE_G peak ground acceleration
F_{PGA}	1.21	Site amplification factor at PGA
PGA_M	0.472	Site modified peak ground acceleration
T_L	16	Long-period transition period (s)

SsRT	0.848	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.96	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.409	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.471	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.647	Factored deterministic acceleration value (1.0s)
PGAd	0.53	Factored deterministic acceleration value (PGA)

* See Section 11.4.7

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

While the information presented on this website is believed to be correct, ATC and its sponsors and contributors assume no responsibility or liability for its accuracy. The material presented in the report should not be used or relied upon for any specific application without competent examination and verification of its accuracy, suitability and applicability by engineers or other licensed professionals. ATC does not intend that the use of this information replace the sound judgment of such competent professionals, having experience and knowledge in the field of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the results of the report provided by this website. Users of the information from this website assume all liability arising from such use. Use of the output of this website does not imply approval by the governing building code bodies responsible for building code approval and interpretation for the building site described by latitude/longitude location in the report.

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (upd

Latitude

Decimal degrees

45.306859

Longitude

Decimal degrees, negative values for western longitudes

-122.940625

Site Class

259 m/s (Site class D)

Spectral Period

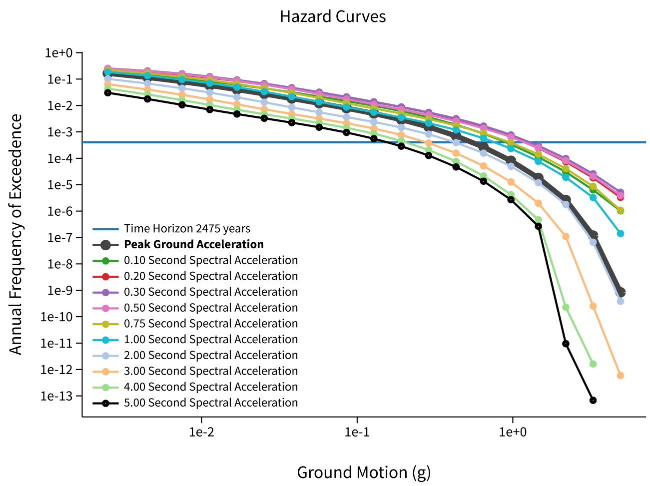
Peak Ground Acceleration

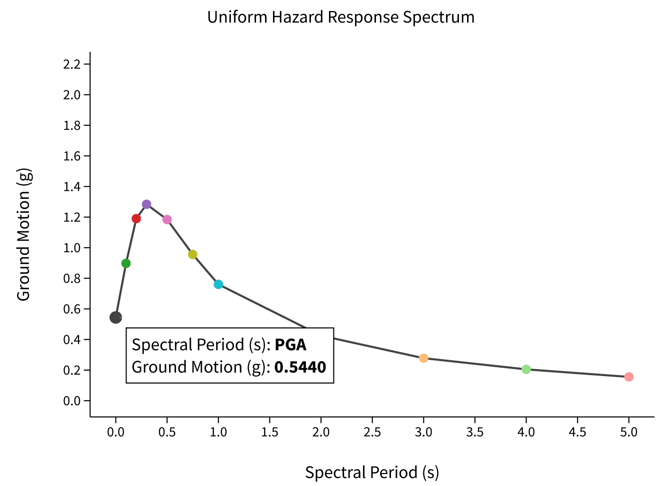
Time Horizon

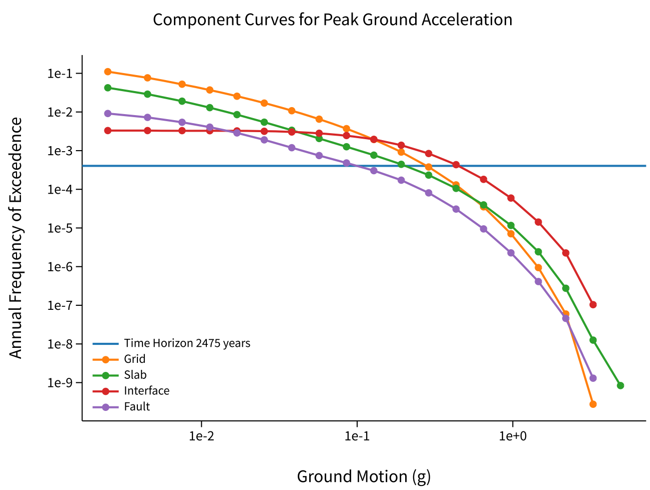
Return period in years

2475

Hazard Curve





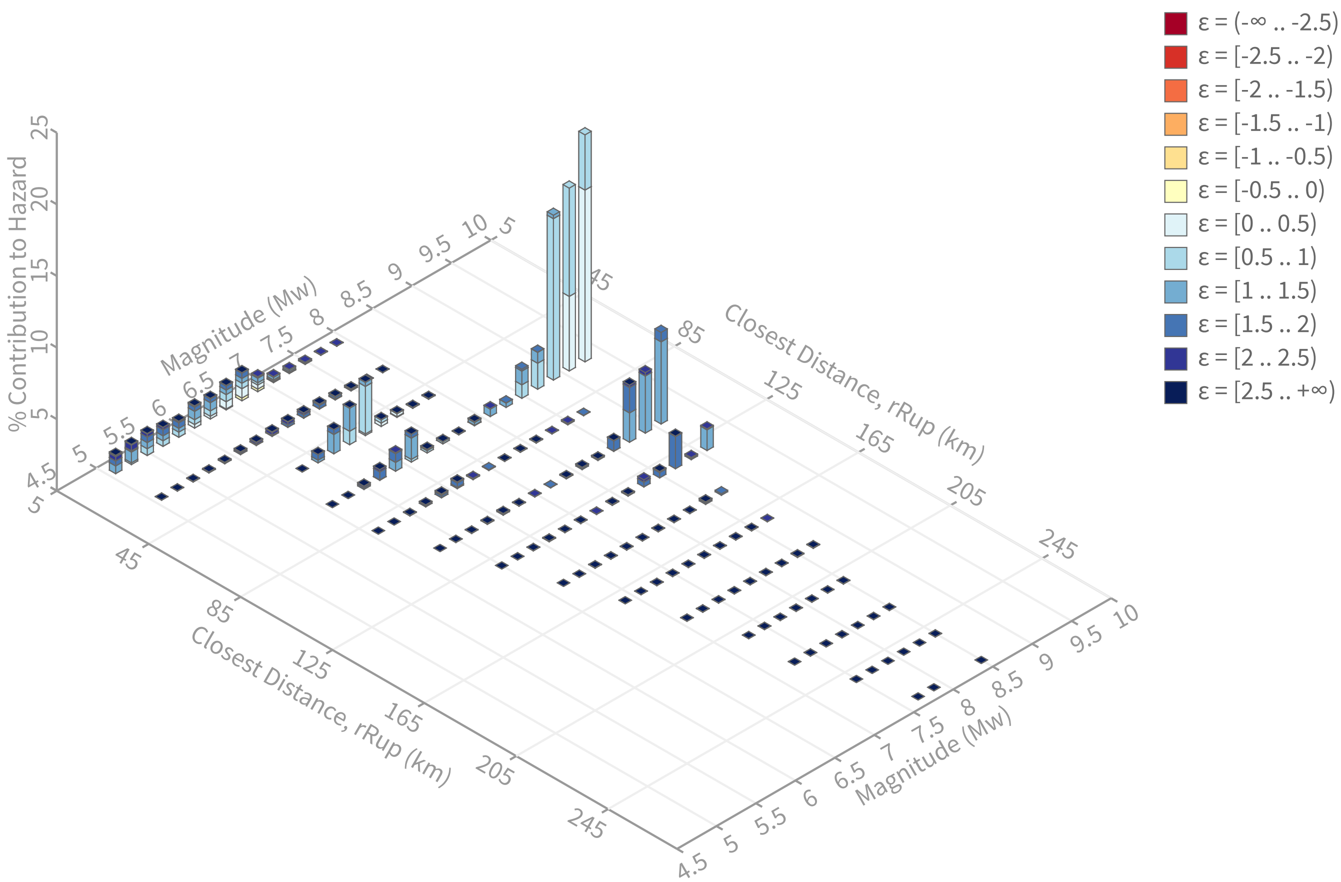


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.54401823 g

Recovered targets

Return period: 2466.6527 yrs
Exceedance rate: 0.00040540771 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.51 %

Mean (over all sources)

m: 8.17
r: 65.91 km
ε_0: 1.02 σ

Mode (largest m-r bin)

m: 9.34
r: 63.87 km
ε_0: 0.39 σ
Contribution: 15.85 %

Mode (largest m-r-ε_0 bin)

m: 9.34
r: 63.87 km
ε_0: 0.29 σ
Contribution: 11.99 %

Discretization

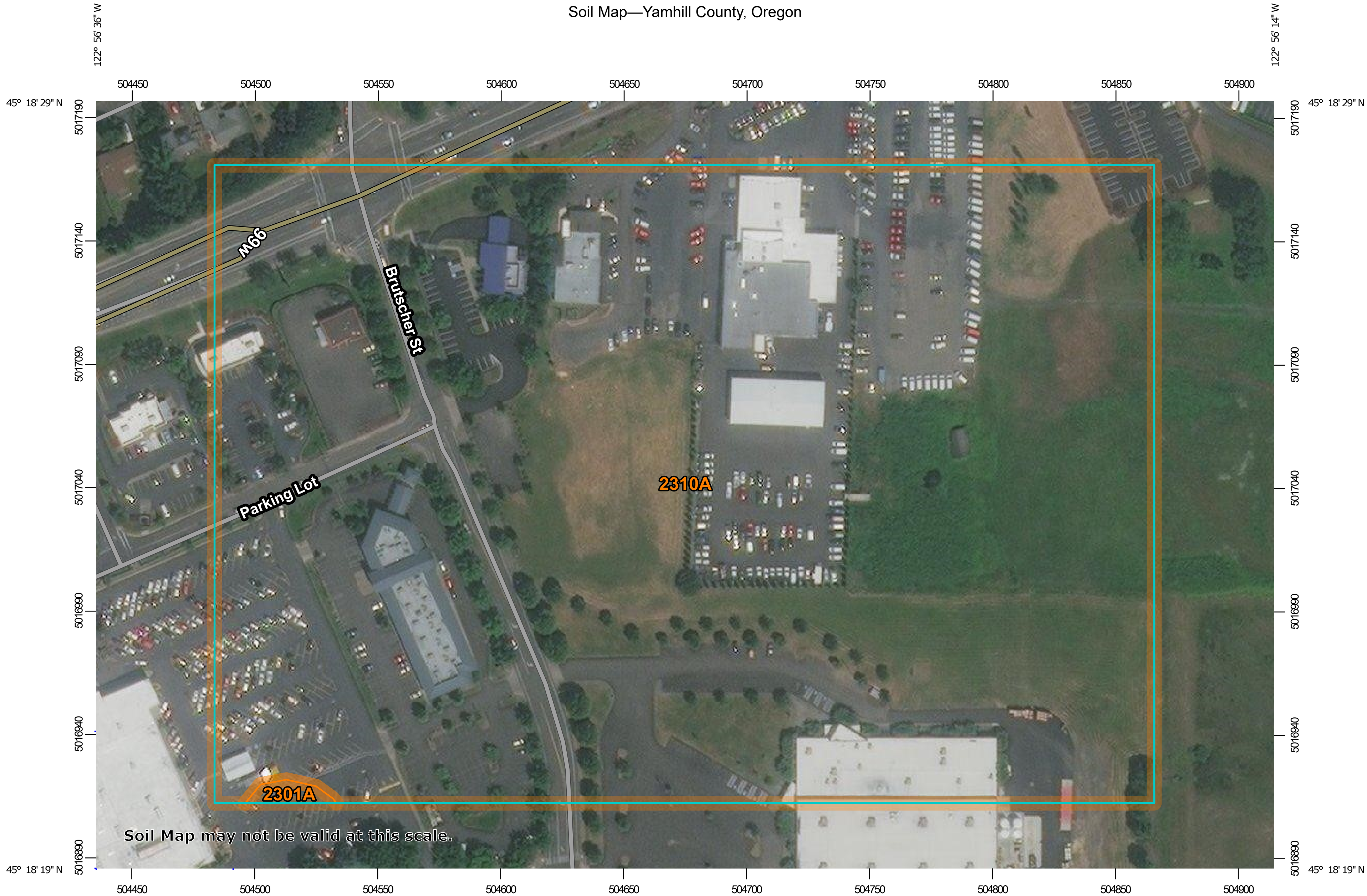
r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

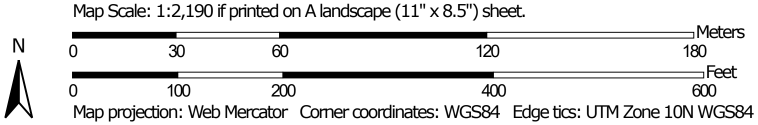
ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
sub0_ch_bot.in	Interface							38.40
Cascadia Megathrust - whole CSZ Characteristic		63.87	9.10	0.56	123.599°W	45.501°N	293.06	38.40
sub0_ch_mid.in	Interface							14.09
Cascadia Megathrust - whole CSZ Characteristic		112.69	8.92	1.41	124.330°W	45.489°N	281.09	14.09
coastalOR_deep.in	Slab							8.17
coastalOR_deep.in	Slab							5.09
sub0_ch_top.in	Interface							3.26
Cascadia Megathrust - whole CSZ Characteristic		127.44	8.83	1.63	124.549°W	45.485°N	279.52	3.26
WUSmap_2014_fixSm.ch.in (opt)	Grid							2.63
noPuget_2014_fixSm.ch.in (opt)	Grid							2.63
sub2_ch_bot.in	Interface							2.44
Cascadia Megathrust - Goldfinger Case C Characteristic		74.96	8.73	1.02	123.702°W	45.000°N	240.53	2.44
WUSmap_2014_fixSm.gr.in (opt)	Grid							2.34
noPuget_2014_fixSm.gr.in (opt)	Grid							2.34
sub1_ch_bot.in	Interface							1.74
Cascadia Megathrust - Goldfinger Case B Characteristic		63.29	8.86	0.72	123.599°W	45.501°N	293.06	1.74
Geologic Model Partial Rupture	Fault							1.48
sub1_GRb0_bot.in	Interface							1.33
Cascadia floater over southern zone - Goldfinger Case B		68.55	8.48	1.05	123.599°W	45.501°N	293.06	1.33
Geologic Model Full Rupture	Fault							1.05



Map Scale: 1:2,190 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 1:24,000.

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: Yamhill County, Oregon
Survey Area Data: Version 7, Sep 10, 2019

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

Date(s) aerial images were photographed: Aug 19, 2015—Sep 13, 2016

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.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
2301A	Amity silt loam, 0 to 3 percent slopes	0.1	0.2%
2310A	Woodburn silt loam, 0 to 3 percent slopes	24.5	99.8%
Totals for Area of Interest		**24.5**	**100.0%**

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Test Pit TP-2

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Test Pit TP-3

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Test Pit TP-4