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Exhibit 3.6

GeoPacific Engineering, Inc.

Real-World Geotechnical Solutions
Investigation • Design • Construction Support

Geotechnical Investigation

Hall Boulevard Apartments
6780 Hall Boulevard
Beaverton, Oregon

GeoPacific Engineering, Inc. Job No. 19-5395
January 8, 2020

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January 8, 2020
Project No. 19-5395

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SUBJECT: **GEOTECHNICAL INVESTIGATION**
SW HALL BOULEVARD APARTMENTS
6780 HALL BOULEVARD
BEAVERTON, OREGON 97008

1 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-7178, dated December 5, 2019, and your subsequent authorization of our proposal and *General Conditions for Geotechnical Services*.

Site Location:	6780 Hall Boulevard Beaverton, Oregon 97008 (see Figure 1)
Civil Engineer:	NW Engineers Mathew Newman 3409 NE John Olsen Avenue Hillsboro, Oregon 97124
Jurisdictional Agency:	Beaverton, Oregon

2 SITE AND PROJECT DESCRIPTION

The subject site is located at 6780 SW Hall Boulevard in Beaverton, Oregon and consists of tax parcel R194275, totaling approximately 0.46 acres in size. Topography onsite is gently sloping to the northeast with site elevations ranging from 250 to 243 feet amsl. The single-family home which existed onsite was recently demolished. Vegetation consists primarily of short grasses, landscaping, shrubs and medium to large trees along the perimeter of the site.

GeoPacific understands site development will include construction of a new 10-unit apartment building with associated new private drives, installation of new underground utilities, and stormwater infiltration facilities. The apartment building will be three levels, constructed on typical spread foundations with square column footings, continuous strip footings, and crawl spaces.

3 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 subject site is underlain by Quaternary age (last 1.6 million years) surficial catastrophic flood deposits associated with repeated glacial outburst flooding of the Willamette Valley (Madin, 2004). The last of these outburst floods occurred about 10,000 years ago. The subject site is underlain by the coarse-grained facies of these outburst floods (Beeson et al., 1991). These deposits typically consist of dense gravel.

Underlying the alluvium is the Miocene aged (about 14.5 to 16.5 million years ago) Columbia River Basalt Formation (Beeson et al, 1989). These basalts are a thick sequence of lava flows which form the crystalline basement of the Tualatin Valley. The basalts are composed of dense, finely crystalline rock that is commonly fractured along blocky and columnar vertical joints. Individual basalt flow units typically range from 25 to 125 feet thick and interflow zones are typically vesicular, scoriaceous, brecciated, and sometimes include sedimentary rocks.

4 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 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 about 6.3 miles

northeast of the site. The Oatfield Fault occurs along the western side of the Portland Hills and is about 3.9 miles northeast of the site. The accuracy of the fault mapping is stated to be within 500 meters (Wong, et al., 2000). 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 14.7 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 FIELD EXPLORATION AND SUBSURFACE CONDITIONS

Our site-specific exploration for this report was conducted on December 26, 2019. Three exploratory test pits, designated TP-1 through TP-3, were excavated at the site to depths between 10 and 12 feet below ground surface (bgs). The approximate locations of our test pit explorations are indicated on Figures 2 and 3. Test pits were located in the field by pacing or taping distances from apparent property corners and other site features shown on the plans provided and their locations should be considered approximate.

During the explorations, GeoPacific observed and recorded pertinent soil information such as color, stratigraphy, strength, and soil moisture content. Soils were classified in general accordance with the Unified Soil Classification System (USCS). Soil samples obtained during the exploration were placed in relatively air-tight plastic bags. Upon test completion, the test pits were backfilled loosely with onsite soil. Summary exploration logs are attached. The stratigraphic contacts shown on the exploration 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

Undocumented Fill: The ground surface in test pits TP-1 and TP-2 was underlain by 14 to 16 inches of soft to medium stiff, dark brown Clayey SILT (ML). The undocumented fill contained fine roots and debris, and was surfaced with grass as the location of test pit TP-1 and with a two-inch-thick layer of crushed aggregate at the location of test pit TP-2.

Topsoil Horizon: The ground surface in test pit TP-3 was underlain by four inches of soft, dark brown Clayey SILT (ML). The topsoil contained fine to medium roots and was surfaced with grass.

Catastrophic Flood Deposits: Underlying the undocumented fill in test pits TP-1 and TP-2, and the topsoil in test pit TP-3, we observed Catastrophic Flood Deposits. At the location of test pit TP-1, the uppermost portion of the Catastrophic Flood Deposit soils consisted of medium stiff, low plasticity brown SILT (ML). The low plasticity silt extended to a depth of approximately 3 feet bgs. In test pits TP-2 and TP-3, and underlying the low plasticity silt in TP-1, Catastrophic Flood Deposits generally consisted of moderate plasticity, elastic, brown SILT (MH). The medium plasticity silt extended beyond the maximum depth of investigation in our test pit explorations.

5.2 Shrink-Swell Potential

The fine-grained soils present in the upper several feet of the ground surface displayed low to moderate plasticity characteristics. The shrink-swell potential of near surface soils are not anticipated to require special design measures where structures are proposed. However, the soil type is moisture sensitive, and will be difficult to work with during periods of wet weather.

5.3 Groundwater and Soil Moisture

On December 26, 2019, observed soil moisture conditions were generally very moist at surface, and moist below the topsoil and undocumented fill material. No groundwater seepage was encountered in explorations TP-2 and TP-3. Light groundwater seepage was encountered at 9.5 feet bgs in test pit TP-1, which was below the depth of infiltration testing. According to the *Estimated Depth to Groundwater in the Portland, Oregon Area*, (United States Geological Survey, Snyder, 2020 website), groundwater is expected to be present at depths ranging from approximately 65 to 70 feet bgs. Groundwater conditions are anticipated to vary depending on the season, local subsurface conditions, changes in site use, and other factors. Seeps and springs may exist in areas not explored and may become evident during site grading.

5.4 Infiltration Testing

On-site soil infiltration testing was performed using the encased falling head procedure at a depth of 6 feet bgs in test pit TP-1. Soils in test pit TP-1 were observed and sampled to characterize the subsurface profile. The test pit soils were presoaked for a period of two hours before taking measurements. During testing, the water level was measured to the nearest 0.01 foot (1/8 inch) from a fixed point at regular intervals until three successive measurements showed a consistent infiltration rate. Table 1 summarizes the infiltration test results.

Table 1 – Summary of Infiltration Test Results

Test Location	Infiltration Testing Method	Depth (feet)	Soil Type	Infiltration Rate (in/hr)
TP-1	Encased Falling Head	6	Elastic SILT (MH)	0.0

Infiltration rates have been reported without applying a factor of safety.

6 CONCLUSIONS AND DESIGN 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. In our opinion, the most significant geotechnical issue for the proposed development is site preparation due to the presence of undocumented fill material, and remaining building debris from the recently demolished onsite structures. The following report sections provide recommendations for site development and construction in accordance with the current applicable codes and local standards of practice.

6.1 Site Preparation Recommendations

Areas of proposed structures, and areas to receive fill should be cleared of vegetation and any organic and inorganic debris. During our site investigation, portions or the existing building slab remained onsite. During site preparation, concrete, slabs, bricks, and other building debris should be removed from the site. Organic materials from clearing should either be removed from the site or placed as landscape fill in areas not planned for structures. Organic-rich topsoil should then be stripped from construction areas of the site or where engineered fill is to be placed. The estimated

average necessary depth of removal is 6-12 inches. However, deeper stripping to remove large tree roots or other organics may be necessary in portions of the site.

The final depth of soil removal will be determined by a site inspection after the stripping/excavation has been performed. Stripped topsoil should be stockpiled only in designated areas and stripping operations should be observed and documented by the geotechnical engineer or their representative.

Undocumented fill and all subsurface structures (tile drains, basements, driveway and landscaping fill, old utility lines, septic leach fields, wells, etc.) should be removed and the excavations backfilled with engineered fill. We observed a brick-lined well in the northeastern potion of the site (see photographic log). The well extended below a depth of 10 feet bgs, and contained water at the time of the investigation. GeoPacific recommends that the drywell be removed and the excavation and surrounding voids backfilled with engineered fill as described in the *Engineered Fill* section. We observed undocumented fill material in test pits TP-1 and TP-2 to depths of 1.3 and 1.2 feet bgs, respectively. Undocumented fill may also be encountered in areas not explored by our explorations.

Following removal of surficial debris, topsoil horizon, and undocumented fill, exposed subgrade soils should be evaluated by GeoPacific. For large areas, this evaluation is normally performed by proof-rolling the exposed subgrade with a fully loaded scraper or dump truck. For smaller areas where access is restricted, the subgrade should be evaluated by probing the soil with a steel probe. Soft/loose soils identified during subgrade preparation should be compacted to a firm and unyielding condition or over-excavated and replaced with engineered fill, as described below. The depth of overexcavation, if required, should be evaluated by GeoPacific at the time of construction.

6.2 Engineered Fill

All grading for the proposed construction should be performed as engineered grading in accordance with the applicable building code at the time of construction with the exceptions and additions noted herein. Site grading should be conducted in accordance with the requirements outlined in the 2018 International Building Code (IBC), Proposed fill placement areas should be prepared as described in the *Site Preparation Recommendations* section. Surface soils should then be scarified and recompacted before structural fill placement. Site preparation, soil stripping, and grading activities should be observed and documented by a geotechnical engineer or their representative. Proper test frequency and earthwork documentation usually requires daily observation and testing during stripping, rough grading, and engineered fill placement.

We anticipate that onsite native soils consisting primarily of silt may be suitable 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 before 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. 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 their 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.

During our site investigation performed on December 26th, 2019, GeoPacific observed a brick-lined drywell on the northeastern portion of the site. Based on our experience in the site area, it is possible that more old dry wells may be present on site. The following recommendations are provided for removal and backfill of existing drywells onsite. The drywell(s) should be excavated and debris removed from the site. GeoPacific should observe the bottom and sides of the excavation prior to backfilling. Deeper portions of dry wells should be backfilled with controlled density fill (CDF), which is essentially a lean mix concrete consisting of water, sand and cement. We recommend use of "excavatable" CDF so that future excavations can be made through the dry well backfill if any new utilities or other excavations are needed in the affected areas. Above a depth of about 8 feet, at the contractor's option, backfill may consist of granular soils such as "reject rock," recycled concrete or similar material approved by GeoPacific. The granular backfill should be placed in lifts no thicker than about 18 inches and compacted with a "hoe-pac" excavator attachment to a minimum of 95 percent of Standard Proctor (ASTM D698). This backfill specification should also be used for any basements or other depressions that require fill during the demolition process.

Site earthwork may be impacted by shallow perched 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 on-site soils can generally be excavated using conventional heavy equipment. Maintaining 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 silt 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 during the wet weather season and should be anticipated in excavations and utility trenches. Vibrations created by traffic and construction equipment may cause some caving and raveling of excavation walls. In such an event, lateral support for the excavation walls should be provided by the contractor to prevent loss of ground support and possible distress to existing or previously constructed structural improvements.

PVC pipe should be installed in accordance with the procedures specified in ASTM D2321 and jurisdictional 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 or approved equivalent 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 or approved native granular 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. Large vibrating compaction equipment use 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 that may be considered highly susceptible to erosion. In our opinion, the primary concern regarding erosion potential will occur during construction in areas that have been stripped of vegetation. Erosion at the site during construction can be minimized by implementing the project erosion control plan, which should include judicious use of straw wattles, 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 surface water run-off and prevent surface water ponding;
- 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 fill placement 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 wattles, 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

The proposed structures may be supported on shallow foundations bearing on medium stiff, native soils and/or engineered fill, appropriately designed and constructed as recommended in this report. Foundation design, construction, and setback requirements should conform to the applicable building code at the time of construction. To maximize bearing strength and protect against frost heave, spread footings should be embedded at a minimum depth of 18 inches below exterior grade. If soft soil conditions are encountered at footing subgrade elevation, they should be removed and replaced with compacted crushed aggregate.

The anticipated allowable soil bearing pressure is 1,500 lbs/ft^2 for footings bearing on medium stiff, native soil and/or engineered fill. In areas where greater bearing pressure is desired, an allowable bearing pressure of 2,000 lbs/ft^2 may be used for footings bearing on at least 12 inches of engineered fill consisting of ¾”-0 crushed aggregate or approved granular material compacted to at least 95 percent of the maximum dry density determined by ASTM D698 (Standard Proctor) or equivalent, or if the footings are embedded to a depth of 3 feet below the ground surface. Where needed, the backfill underneath the footings should consist of 1½”-0 crushed aggregate compacted to at least 95 percent of its maximum dry density as determined by ASTM D698 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 heavier loads, the geotechnical engineer should be consulted. The coefficient of friction between on-site soil and poured-in-place concrete may be taken as 0.42, which includes no factor of safety. The maximum anticipated total and differential footing movements are 1 inch and ¾ inch over a span of 20 feet, respectively. We anticipate that the majority of the estimated settlement will occur during construction, as loads are applied. Excavations near structural footings should not extend within a 1H:1V plane projected downward from the bottom edge of footings.

Footing excavations should penetrate through topsoil and any loose 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 before placing reinforcing steel bars. Due to the moisture sensitivity of on-site native soils, foundations constructed during the wet weather season may require overexcavation of footings and backfill with compacted, crushed aggregate.

6.7 Flexible Pavement Design

We understand that site development includes drive lanes and a parking lot. We assume that traffic will primarily consist of light duty residential cars, heavy shipping vehicles, weekly trash and recycling pickups, and occasional fire and maintenance trucks up to 75,000 lbs and point loads up to 12,000 lbs. We assumed an 18-kip ESAL count of 28,161 over 20 years. For the proposed pavement sections, we conservatively assume that the subgrade with exhibit a resilient modulus of at least 6,000 psi which correlates to a California Bearing Ratio (CBR) of 4. Table 2 presents our flexible pavement design input factors.

Table 2 – Design Input Factors for Pavement Sections

Input Parameter	Design Value
18-kip ESAL Initial Performance Period (20 Years)	28,161
Initial Serviceability	4.2
Terminal Serviceability	2.5
Reliability Level	90 Percent
Overall Standard Deviation	0.44
Roadbed Soil Resilient Modulus (PSI)	6,000
Structural Number	2.08

Table 3 presents our recommended minimum dry-weather pavement section with estimated structural coefficients. Pavement design calculations are attached to this report.

Table 3 - Recommended Minimum Dry-Weather Pavement Sections (HMAC)

Material Layer	Section Thickness (in.)	Structural Coefficient	Compaction Standard
Asphaltic Concrete (AC)	3	0.4	92% of Rice Density AASHTO T-209
Crushed Aggregate Base ¾”-0 (leveling course)	2	0.1	95% of Modified Proctor AASHTO T-180
Crushed Aggregate Base 1.5”-0 (base course)	8	0.1	95% of Modified Proctor AASHTO T-180
Subgrade	6,000 PSI		95% of Standard Proctor AASHTO T-99 or equivalent
Calculated Structural Number	**2.20**		

Subgrade should be ripped or tilled to a depth of 12 inches, moisture conditioned, root-picked, and compacted in-place before the crushed aggregate pavement base is placed. Any pockets of organic debris or loose fill encountered during ripping or tilling should be removed and replaced with engineered fill. 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 before paving.

If pavement areas will 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 subgrade soils are expected to be moisture sensitive and will make the site a difficult wet weather construction project. General recommendations for wet weather pavement sections are provided below.

When placing 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.

6.8 Wet Weather Construction Pavement Section

Based on our site review, we recommend a wet weather section with a minimum subgrade deepening of 6 inches to accommodate a working subbase of additional 1½”-0 crushed rock. Geotextile fabric, Mirafi 500x or equivalent, should be placed on subgrade soils before placing base rock.

In some instances, it may be preferable to use Special Treated Base (STB) in combination with overexcavation and increasing the thickness of the rock section. GeoPacific should be consulted for additional recommendations regarding use of STB in wet weather pavement sections if this alternative is desired. Cement treating the subgrade may also be considered instead of overexcavation. For planning purposes, we anticipate that onsite soil treatment would involve mixing cement powder to approximately 6 percent cement content and a mixing depth on the order of 12 inches.

By implementing 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. Construction in wet weather is risky and pavement subgrade performance depends on a number of factors including the weather conditions, the contractor’s methods, and the amount of traffic the road is subjected to. 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 before paving, the soft spots should be overexcavated 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 overcompacting the base course materials, which could create pumping or unstable subgrade soil conditions. Heavy and/or vibratory compaction efforts should be applied with caution. After the crushed rock is placed and compacted to 95 percent of Modified Proctor (ASTM D1557), 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.

6.9 Concrete Slab-on-Grade Floors

Areas beneath concrete slab-on-grade floors should be prepared as recommended in the *Site Preparation Recommendations* section. Care should be taken during foundation and floor slab excavation 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 concrete slab-on-grade floor evaluation using the beam on elastic foundation method, a subgrade reaction modulus of 150 kcf (87 pci) should be assumed for the medium stiff, fine-grained soils anticipated to be present in the upper four feet at the site. This value assumes the concrete slab system is designed and constructed as recommended herein, with a minimum thickness of 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 D698 (Standard 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. 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.

6.10 Footing and Roof Drains

The outside edge of perimeter walls should be provided with a drainage system consisting of 3-inch diameter, slotted, flexible plastic pipe embedded in a minimum of 1 ft^3 per lineal foot of clean, free-draining gravel or 1 1/2” - 3/4” 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. Water collected from the footing 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 foundation drains in order to reduce the potential for clogging. The footing 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.

Footing drains are recommended to prevent detrimental effects of surface water runoff on foundations – not to dewater groundwater. Footing 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.

6.11 Permanent Below-Grade Walls

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

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

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

We assume relatively level ground surface below the base of the walls. As such, we recommend passive earth pressure of 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), depending on anticipated traffic loads.

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 drainpipe 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 drainpipe 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.

6.12 Stormwater Management

We understand that plans for project development may include stormwater management facilities and incorporating subsurface stormwater disposal is desired. Based on the results of our infiltration testing, stormwater infiltration and disposal onsite may not be feasible. An infiltration test was conducted at the location of test pit TP-1, which indicated a hydraulic conductivity which was not measurable in the field (0.0 inches per hour).

Stormwater management systems should be constructed as specified by the designer and/or in accordance with the applicable stormwater design codes. The infiltration rates presented in this report do not incorporate a factor of safety. Stormwater exceeding soil infiltration and/or soil storage capacities will need to be directed to a suitable surface discharge location, away from structures.

Infiltration test methods and procedures attempt to simulate the as-built conditions of the planned disposal system; however, due to natural variations in soil properties, actual infiltration rates may vary from the measured and/or recommended design rates. All systems should be constructed such that potential overflow is discharged in a controlled manner away from structures, and all systems should include an adequate factor of safety. Infiltration rates presented in this report should not be applied to inappropriate or complex hydrological models, such as a closed basin,

without extensive further studies. Evaluating environmental implications of stormwater disposal at this site are beyond the scope of this study.

6.13 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* to *severe* 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 2020). We recommend Site Class D be used for design per the OSSC and as defined in ASCE 7. Design values determined for the site using the ATC online Hazards by Location tool are summarized in Table 4 and are based upon existing soil conditions.

Table 4 - Recommended Earthquake Ground Motion Parameters (ASCE 7-16)

Parameter	Value
Location (Lat, Long), degrees	45.471, -122.804
Probabilistic Ground Motion Values, 2% Probability of Exceedance in 50 yrs	
Peak Ground Acceleration PGA_M	0.478 g
Short Period, S_s	0.872 g
1.0 Sec Period, S_1	0.402 g
Soil Factors for Site Class D:	
F_a	1.151
F_v	1.898*
$SD_s = 2/3 \times F_a \times S_s$	0.669 g
$SD_1 = 2/3 \times F_v \times S_1$	0.509 g*
Seismic Design Category	D

* The Fv value reported in the above table is a straight-line interpolation of mapped spectral response acceleration at 1-second period, S1 per Table 1613.2.3(2) of OSSC 2019 with the assumption that Exception 2 of ASCE 7-16 Chapter 11.4.8 is met. SD1 is based on the Fv value. The structural engineer should evaluate exception 2 and determine whether or not the exception is met. If Exception 2 is not met, and the long-period site coefficient (Fv) is required for design, GeoPacific Engineering can be consulted to provide a site-specific procedure as per ASCE 7-16, Chapter 21.

6.14 Soil Liquefaction

The DOGAMI, Oregon HazVu: 2020 Statewide GeoHazards Viewer indicates that the site is in an area with *moderate* risk for soil liquefaction during an earthquake. Soil liquefaction is a phenomenon wherein saturated soil deposits temporarily lose strength and behave as a liquid in response to ground shaking caused by strong earthquakes. Soil liquefaction is generally limited to loose sands and granular soils located below the water table, and fine-grained soils with a plasticity index less than 8; however, some studies have shown there to be liquefaction potential in fine-grained soils with a plasticity index as high as 15. The measured plasticity index of soils in the location of test pit TP-1 was greater than 30.

The subsurface profile observed within our explorations and our experience with geologic conditions in the site vicinity indicate that the site is underlain by medium plasticity silt which is not considered susceptible to liquefaction. Static groundwater was not encountered in our explorations, excavated to depths of up to 12 feet. According to the *Estimated Depth to Groundwater in the Portland, Oregon Area, (United States Geological Survey, Snyder, 2020 website)*, static groundwater is expected to be present at depths of 65 to 70 feet bgs in the vicinity of the site. Based on the results of our subsurface investigation and our understanding of the geologic conditions in the site vicinity, it is our opinion that the risk of liquefaction on the site is very low and that no special measures are needed to address liquefaction.

7 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. 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.

REGISTERED PROFESSIONAL ENGINEER
14743PE
OREGON
JAN. 23, 1990
JAMES D. IMBRIE

EXPIRES: 06/30/2021

Thomas Torkelson, E.I.T.
Geotechnical Staff

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

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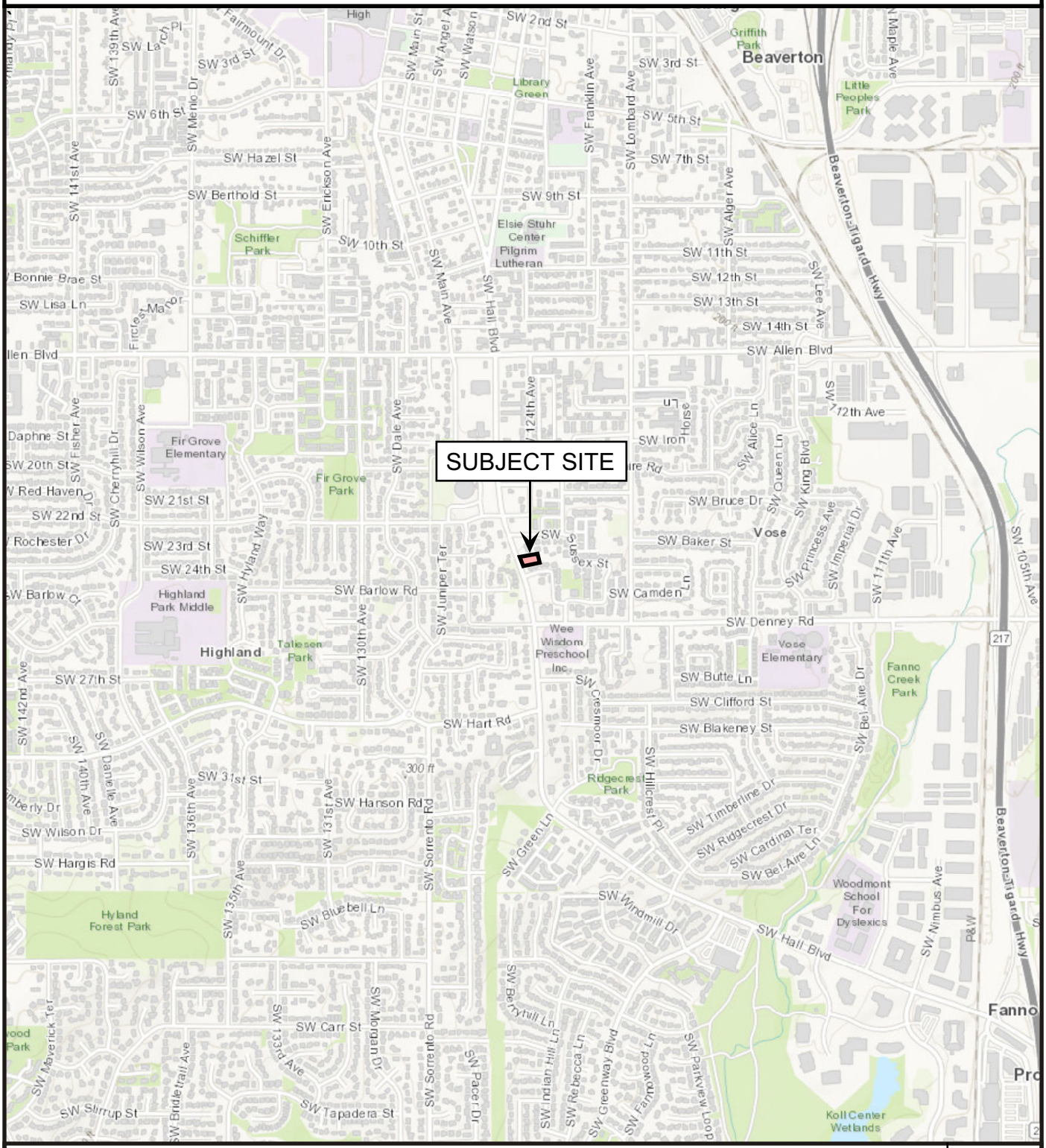
FIGURES

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

SITE VICINITY MAP



Base map: DOGAMI HAZVU Maps 2020
Date: 01/02/2019
Drawn by: TJT

1:24,000

0 0.13 0.25 0.5 mi

0 0.23 0.45 0.9 km

North

Project: Hall Boulevard Apartments Beaverton, Oregon	Project No. 19-5395	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: Base Map Obtained From Google Earth 2020

TP-1
Test Pit Designation and Approximate Location

(1.3') Observed Depth of Undocumented Fill or Topsoil, Feet

APPROXIMATE SCALE (FEET)
20 0 20 40

Drawn by: TJT
Date: 1/02/2020

North

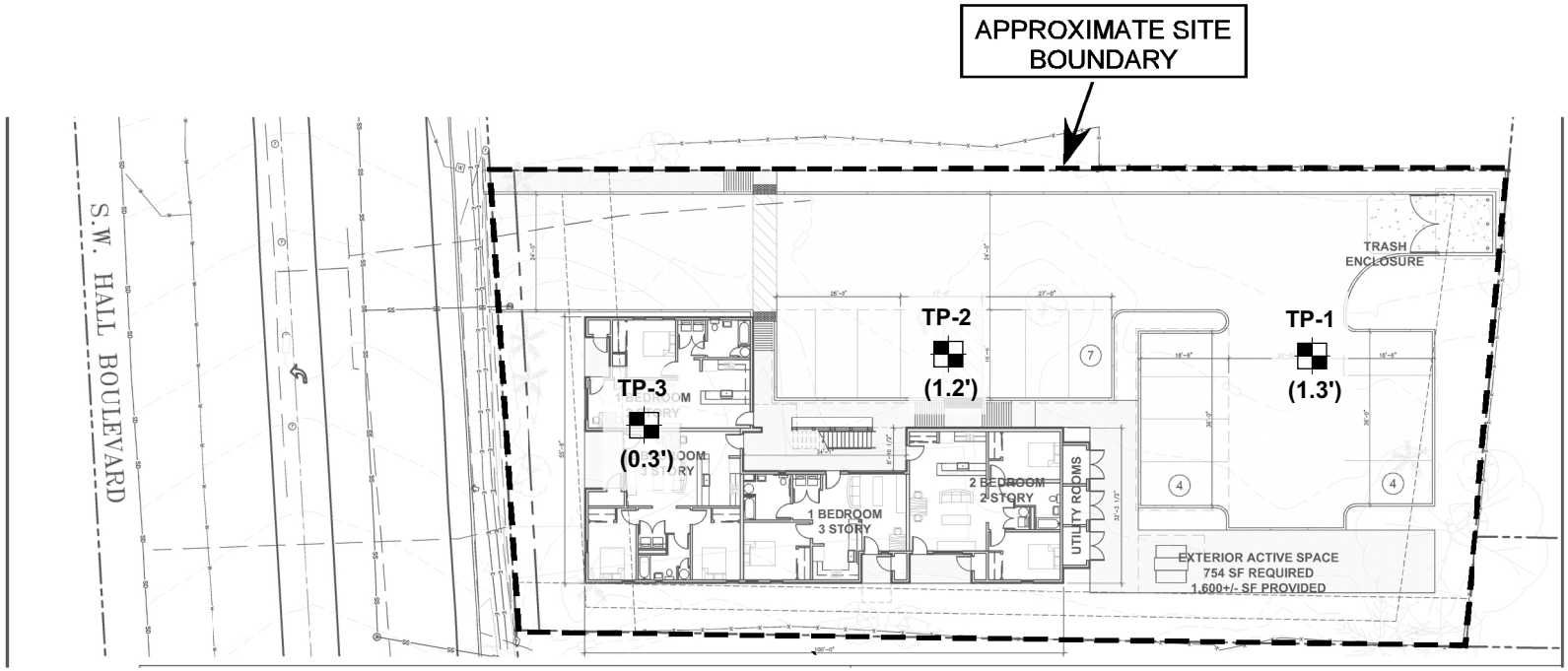
Project: Hall Boulevard Apartments
6780 Hall Boulevard
Beaverton, Oregon 97005

Project No. 19-5395

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: Base Map Provided by Livermore Architecture, Inc.

TP-1

Test Pit Designation and Approximate Location

(1.3') Observed Depth of Undocumented Fill or Topsoil, Feet

APPROXIMATE SCALE (FEET)

20 0 20 40

Drawn by: TJT

Date: 1/02/2020

North

Project: Hall Boulevard Apartments
6780 Hall Boulevard
Beaverton, Oregon 97008

Project No. 19-5395

FIGURE 3

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

EXPLORATION LOGS

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Portland, Oregon 97224
Tel (503) 598-8445
Fax (503) 941-9281

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

TEST PIT LOG

Project: Hall Boulevard Apartments
Beaverton, Oregon

Project No. 19-5395

Test Pit No. **TP-1**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	Fines Content (%)	Moisture Content (%)	Water Bearing Zone	Material Description
1	1.0					0-16" Clayey SILT (ML), dark brown, soft to medium stiff, very moist, with roots and landscaping debris, surfaced with grass. (Undocumented Fill).
2	1.5					SILT (ML), brown, medium stiff, low plasticity, micaceous, moist. (Catastrophic Flood Deposits).
3	4.0					Elastic SILT (MH), brown, stiff, moderate plasticity, micaceous, moist. (Catastrophic Flood Deposits).
4	4.0			22.8		
5						
6		100 to 1,000 g	98.2	23.6		Infiltration testing conducted at 6 feet bgs. Measured infiltration rate = 0.0 inches per hour [Plasticiy Index = 30.9, Liquid Limit = 64.6]
7						
8				28.5		
9						
10				33.7		Light groundwater seepage, less than 3 gallons per minute observed at 9.5 feet bgs.
11						Test pit terminated at 10.0 feet bgs.
12						Infiltration testing conducted at 6.0 feet bgs. Measured infiltration rate = 0.0 inches per hour.
13						Light groundwater seepage encountered at 9.5 feet bgs.
14						No caving observed.
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: 12/26/2019
Logged By: TJT
Surface Elevation:

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

TEST PIT LOG

Project: Hall Boulevard Apartments
Beaverton, Oregon

Project No. 19-5395

Test Pit No. **TP-2**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	Fines Content (%)	Moisture Content (%)	Water Bearing Zone	Material Description
						0-14" Clayey SILT (ML), dark brown, soft to medium stiff, very moist, with fine to medium roots and trace debris, surfaced with 2 inches of crushed aggregate. (Undocumented Fill).
1	1.0					
						Elastic SILT (MH), brown, stiff, moderate plasticity, micaceous, moist. (Catastrophic Flood Deposits).
2	1.5					
3	4.0					
4	4.0					Grades to very stiff at 4 feet bgs.
5						
6						
7						
8						
9						
10		100 to 1,000 g				
11						Test pit terminated at 11.0 feet bgs.
12						No groundwater or seepage encountered.
13						No caving observed.
14						
15						
16						
17						

LEGEND

Date Excavated: 12/26/2019
Logged By: TJT
Surface Elevation:

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

TEST PIT LOG

Project: Hall Boulevard Apartments
Beaverton, Oregon

Project No. 19-5395

Test Pit No. **TP-3**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	Fines Content (%)	Moisture Content (%)	Water Bearing Zone	Material Description
						0-4" Clayey SILT (ML), dark brown, soft, very moist, with fine to medium roots, surfaced with grass. (Topsoil).
1	1.5					Elastic SILT (MH), brown, stiff, moderate plasticity, micaceous, moist. (Catastrophic Flood Deposits).
2	1.0					
3	3.5					Grades to very stiff at 3 feet bgs.
4	4.0					
5						
6						
7						
8						Grades to very moist at 8 feet bgs.
9						
10						
11		100 to 1,000 g				
12						Test pit terminated at 12.0 feet bgs.
13						No groundwater or seepage encountered.
14						No caving observed.
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

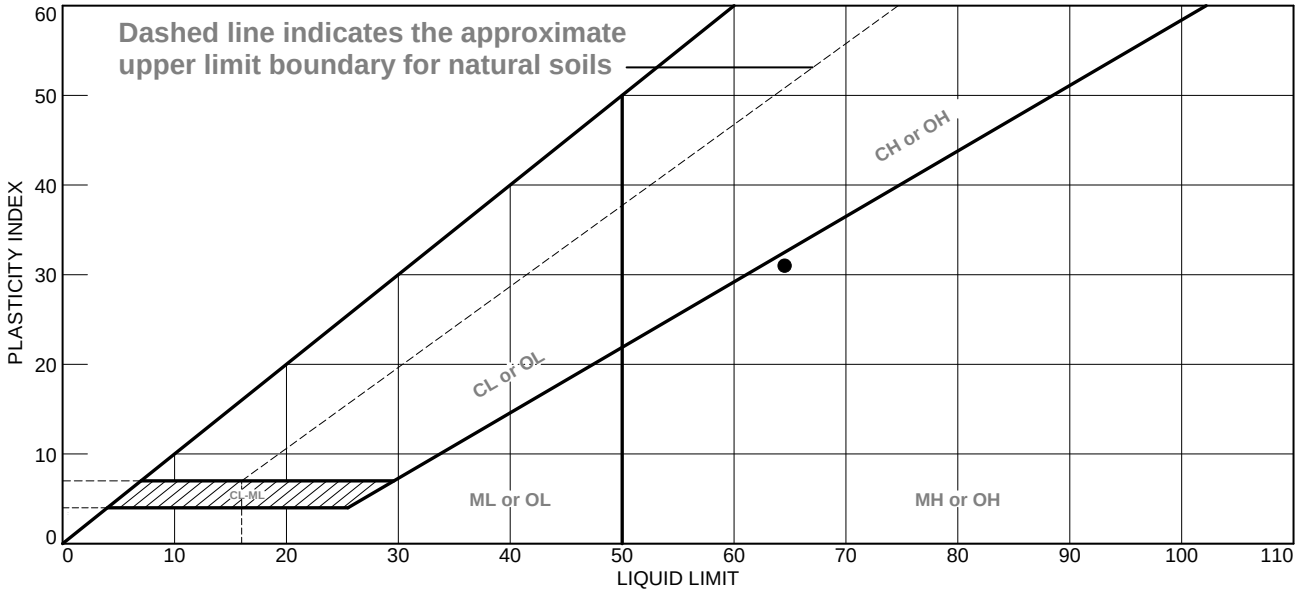
Date Excavated: 12/26/2019
Logged By: TJT
Surface Elevation:

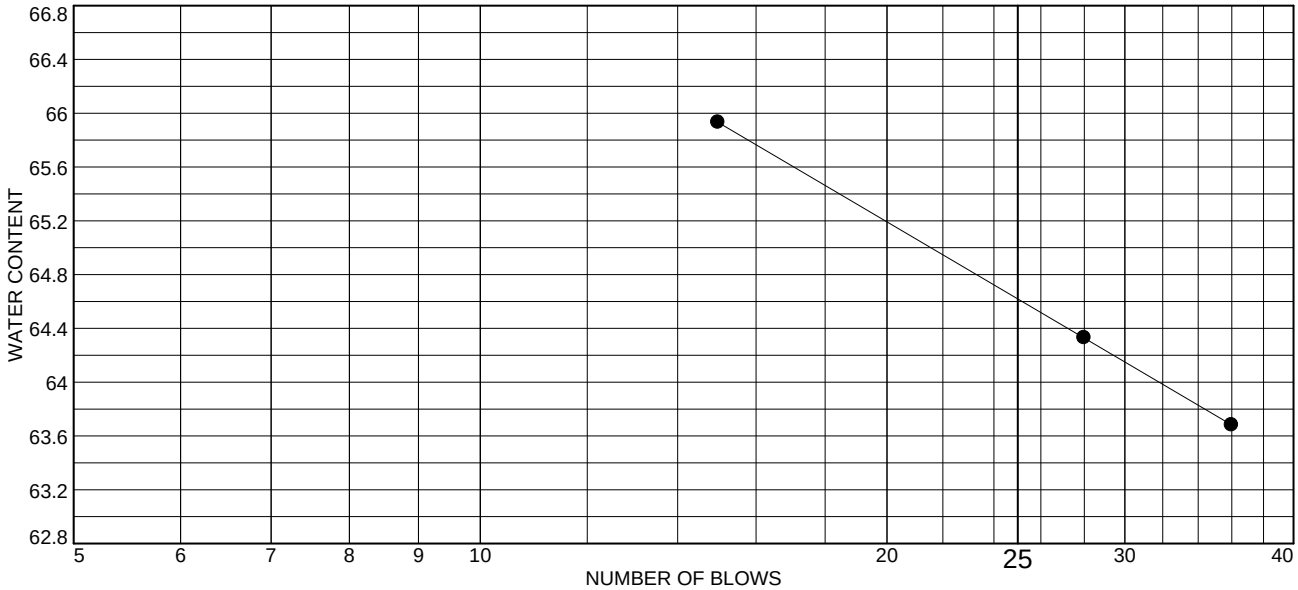
GeoPacific Engineering, Inc.
Real-World Geotechnical Solutions
Investigation • Design • Construction Support

LABORATORY TEST RESULTS

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

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Elastic Silt	64.6	33.7	30.9	99.7	98.2	MH

Project No. 19-5395 **Client:** Evergreen NW, Inc.

Project: Hall Boulevard Apartments

Location: TP-1
Sample Number: S19-311 **Depth:** 6'

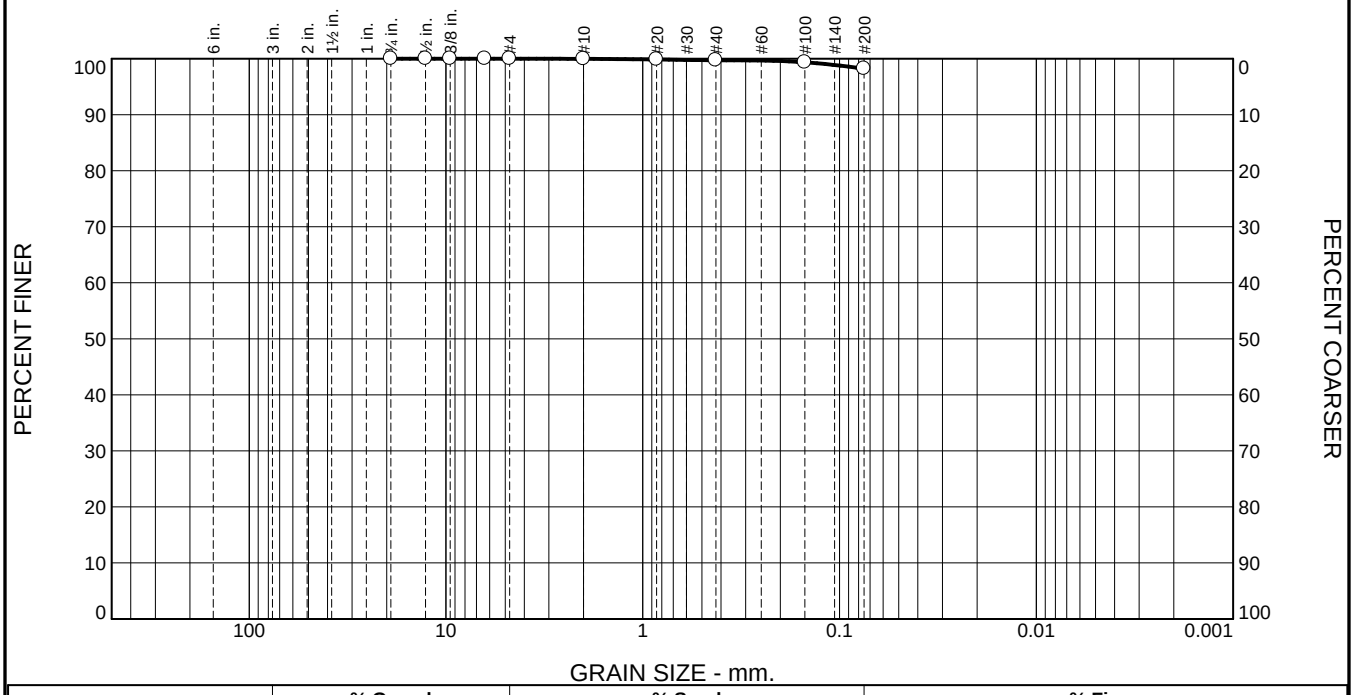
Remarks:

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	0.3	1.5	98.2	

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	100.0		
#20	99.9		
#40	99.7		
#100	99.4		
#200	98.2		

* (no specification provided)

Material Description

Elastic Silt

Atterberg Limits (ASTM D 4318)

PL= 33.7 **LL=** 64.6 **PI=** 30.9

Classification

USCS (D 2487)= MH **AASHTO (M 145)=** A-7-5(38)

Coefficients

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

Remarks

Moisture 23.6%

Date Received: _____ **Date Tested:** 12/31/2019

Tested By: SJC

Checked By: _____

Title: _____

Location: TP-1
Sample Number: S19-311 **Depth:** 6'

Date Sampled: 12/26/2019

GEOPACIFIC ENGINEERING, INC.

Client: Evergreen NW, Inc.
Project: Hall Boulevard Apartments

Project No: 19-5395 **Figure**

Project Name: Hall Boulevard Apartments
Client: Evergreen NW, Inc.
Date Sampled: 12/26/2019
Sampled By: TJT

Project No.: 19-5395
Date Tested: 12/30/2019
Tested By: SJC

Moisture Content

Sample ID:	**S19-310**
Location:	**TP-1**
Depth (ft.):	4'
Tare #:	8
Tare (g):	266.2
Tare + Wet (g):	490.7
Tare + Dry (g):	449.0
Moisture (%):	**22.8**
Sample ID:	**S19-311**
Location:	**TP-1**
Depth:	6'
Tare #:	2
Tare (g):	684.8
Tare + Wet (g):	1034.0
Tare + Dry (g):	967.3
Moisture (%):	**23.6**
Sample ID:	**S19-312**
Location:	**TP-1**
Depth:	8'
Tare #:	13
Tare (g):	264.3
Tare + Wet (g):	489.1
Tare + Dry (g):	439.3
Moisture (%):	**28.5**
Sample ID:	**S19-313**
Location:	**TP-1**
Depth:	10'
Tare #:	12
Tare (g):	270.8
Tare + Wet (g):	647.6
Tare + Dry (g):	552.7
Moisture (%):	**33.7**
Sample ID:	
Location:	
Depth:	
Tare #:	
Tare (g):	
Tare + Wet (g):	
Tare + Dry (g):	
Moisture (%):	

Moisture Content

Sample ID:	
Location:	
Depth (ft.):	
Tare #:	
Tare (g):	
Tare + Wet (g):	
Tare + Dry (g):	
Moisture (%):	
Sample ID:	
Location:	
Depth:	
Tare #:	
Tare (g):	
Tare + Wet (g):	
Tare + Dry (g):	
Moisture (%):	
Sample ID:	
Location:	
Depth:	
Tare #:	
Tare (g):	
Tare + Wet (g):	
Tare + Dry (g):	
Moisture (%):	
Sample ID:	
Location:	
Depth:	
Tare #:	
Tare (g):	
Tare + Wet (g):	
Tare + Dry (g):	
Moisture (%):	
Sample ID:	
Location:	
Depth:	
Tare #:	
Tare (g):	
Tare + Wet (g):	
Tare + Dry (g):	
Moisture (%):	

SITE RESEARCH

Hazards by Location

Search Information

Address:	6780 SW Hall Blvd, Beaverton, OR 97008, USA
Coordinates:	45.4710055, -122.80416020000001
Elevation:	247 ft
Timestamp:	2020-01-02T22:08:14.773Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D

Basic Parameters

Name	Value	Description
S_S	0.872	MCE_R ground motion (period=0.2s)
S_1	0.402	MCE_R ground motion (period=1.0s)
S_{MS}	1.004	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	0.669	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1.151	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.885	Coefficient of risk (0.2s)
CR_1	0.867	Coefficient of risk (1.0s)
PGA	0.398	MCE_G peak ground acceleration
F_{PGA}	1.202	Site amplification factor at PGA
PGA_M	0.478	Site modified peak ground acceleration
T_L	16	Long-period transition period (s)
SsRT	0.872	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.985	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.402	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.464	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.603	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

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.

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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 (update) (v4.2.0)

Spectral Period

Peak Ground Acceleration

Latitude

Decimal degrees

45.471

Time Horizon

Return period in years

2475

Longitude

Decimal degrees, negative values for western longitudes

-122.805

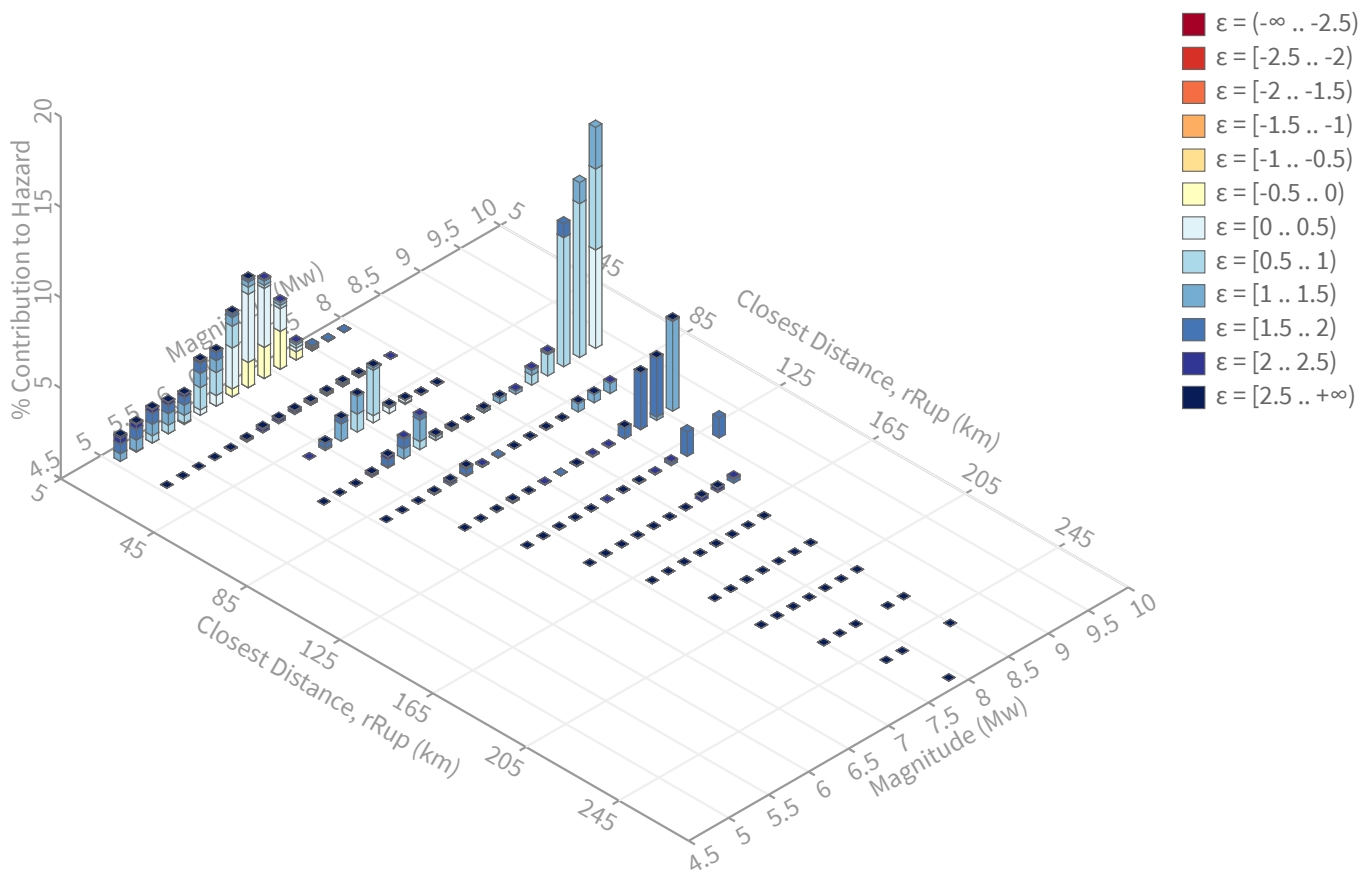
Site Class

760 m/s (B/C boundary)

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.43250045 g

Recovered targets

Return period: 2531.4614 yrs
Exceedance rate: 0.00039502873 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.5 %

Mean (over all sources)

m: 7.78
r: 56.2 km
ε_0: 0.93 σ

Mode (largest m-r bin)

m: 9.34
r: 70.89 km
ε_0: 0.6 σ
Contribution: 12.2 %

Mode (largest m-r-ε_0 bin)

m: 9.01
r: 70.86 km
ε_0: 0.66 σ
Contribution: 8.5 %

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							29.21
Cascadia Megathrust - whole CSZ Characteristic		70.89	9.10	0.72	123.599°W	45.501°N	273.42	29.21
sub0_ch_mid.in	Interface							10.95
Cascadia Megathrust - whole CSZ Characteristic		119.79	8.93	1.47	124.330°W	45.489°N	271.53	10.95
Geologic Model Partial Rupture	Fault							8.52
Portland Hills		8.90	6.74	0.11	122.698°W	45.524°N	54.69	8.43
coastalOR_deep.in	Slab							7.73
Geologic Model Full Rupture	Fault							3.75
Portland Hills		8.84	7.01	-0.03	122.698°W	45.524°N	54.69	3.67
WUSmap_2014_fixSm.ch.in (opt)	Grid							3.60
PointSourceFinite: -122.805, 45.502		6.16	5.81	0.75	122.805°W	45.502°N	0.00	1.03
noPuget_2014_fixSm.ch.in (opt)	Grid							3.60
PointSourceFinite: -122.805, 45.502		6.16	5.81	0.75	122.805°W	45.502°N	0.00	1.03
WUSmap_2014_fixSm.gr.in (opt)	Grid							3.22
noPuget_2014_fixSm.gr.in (opt)	Grid							3.22
coastalOR_deep.in	Slab							3.01
sub0_ch_top.in	Interface							2.33
Cascadia Megathrust - whole CSZ Characteristic		136.77	8.83	1.75	124.549°W	45.485°N	271.26	2.33
Zeng Model Partial Rupture	Fault							1.57
Portland Hills		8.90	6.74	0.11	122.698°W	45.524°N	54.69	1.55
Geologic Model Small Mag	Fault							1.53
sub2_ch_bot.in	Interface							1.37
Cascadia Megathrust - Goldfinger Case C Characteristic		92.34	8.74	1.23	123.702°W	45.000°N	233.62	1.37
sub1_ch_bot.in	Interface							1.31
Cascadia Megathrust - Goldfinger Case B Characteristic		70.26	8.86	0.84	123.599°W	45.501°N	273.42	1.31
WUSmap_2014_fixSm_M8.in (opt)	Grid							1.12
noPuget_2014_fixSm_M8.in (opt)	Grid							1.12

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Investigation • Design • Construction Support

PHOTOGRAPHIC LOG

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HALL BOULEVARD APARTMENTS
GEOTECHNICAL SITE INVESTIGATION
PHOTOGRAPHIC LOG

View of Property from Hall Boulevard, Facing East

Brick-Lined Well on Northeastern Portion of Property (Location on Figure 2)

HALL BOULEVARD APARTMENTS
GEOTECHNICAL SITE INVESTIGATION
PHOTOGRAPHIC LOG

Test Pit TP-2, 11 Feet bgs, Facing North

Test Pit TP-2, Facing Northwest

HALL BOULEVARD APARTMENTS
GEOTECHNICAL SITE INVESTIGATION
PHOTOGRAPHIC LOG

Test Pit TP-3, Facing East

Seepage in Test Pit TP-1, 9.5 Feet bgs