APPENDIX D – GEOTECHNICAL REPORT

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Investigation • Design • Construction Support

Geotechnical Engineering Report

Project Information:	**Lava Drive Apartments** **GeoPacific Project № 23-6332** **May 30, 2023**
Site Location:	**1600 SE Lava Drive** **Clackamas County Taxlot: 11E35AB 100 & 502** **Milwaukie, OR 97206**
Client:	**WDC Properties** **2330 NW 31st Avenue** **Portland, OR 97210** **Email: fstock@wdcproperties.com**

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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 contract dated , dated May 24, 2023, and your subsequent authorization of our proposal and *General Conditions for Geotechnical Services*.

2.0 SITE AND PROJECT DESCRIPTION

The site is located to the south of SE Lava Drive west of the intersection with SE Riverway Lane in the City of Milwaukie, Oregon. The eastern portion of the property is currently occupied by a single-family residence and associated driveway. The western portion of the site is currently undeveloped. The site is gently sloping down to the east with site elevations ranging from 62 to 67. Vegetation onsite consists of short grasses, shrubs, and medium-sized trees. The site is bordered by single-family residences to the south and west, by SE Lava Drive to the north, and by SE Riverway Lane to the east.

It is our understanding that a to 3-story apartment building will be constructed in the eastern portion of the site. Associated parking areas, driveways, and underground utilities are also planned. It is anticipated that the structures will be founded on conventional shallow foundations. A grading plan has not yet been provided for our review. However, we anticipate that cuts and fills will be on the order of 4 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 subject site is underlain by the Quaternary age (last 1.6 million years) Catastrophic Flood Deposits associated with repeated glacial outburst flooding of the Willamette Valley (Madin, 1990). The last of these outburst floods occurred about 10,000 years ago. These deposits typically consist of sand to coarse gravel and cobbles. Regional studies indicate that the thickness of the Catastrophic Flood Deposites in the vicinity of the subject site is approximately 60 feet (Madin, 1990).

Regional geologic mapping indicates the Catastrophic Flood Deposits are underlain by Eocene age (34 to 55 million years ago) Basalt of Waverly Heights (Beeson et al., 1989 and Madin, 1990). Basalt of Waverly Heights are a dense, vesicular, and finely crystalline rock with secondary mineralization. Interflow zones are well developed, vesicular, and commonly include sedimentary deposits. The Basalt of Waverly Heights can be distinguished from the Columbia River Basalt

Group by its darker color, secondary mineralization within vesicles, and mineralogical composition. The top of the Waverly Heights Basalt typically includes a highly weathered rock/residual soil layer up to 30 feet thick which is generally thin or absent in areas of erosional scour that occurred during catastrophic flooding events (Beeson et al., 1989).

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 Grant Butte and Damascus-Trickle Creek Fault 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 approximately 0.8 miles northeast of the site. The East Bank Fault is oriented roughly parallel to the Portland Hills Fault, on the east bank of the Willamette River, and is located approximately 4.8 miles north of the site. The Oatfield Fault occurs along the western side of the Portland Hills and is approximately 1.3 miles southwest of the site. The Oatfield Fault is considered to be potentially seismogenic (Wong, et al., 2000). Madin and Mabey (1996) indicate the Portland Hills Fault Zone has experienced Late Quaternary (last 780,000 years) fault movement; however, movement has not been detected in the last 20,000 years. 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 Grant Butte and Damascus-Trickle Creek Fault Zone

The Grant Butte fault zone was mapped along the north side of Mt. Scott and Powell Butte by Madin (1990). The fault is approximately 8.6 miles northeast of the subject site and extends eastward to Grant Butte on the basis of mapping by CH2M Hill and others (1991) and informally named the Grant Butte fault (Cornforth and Geomatrix, 1992). The Damascus-Trickle Creek fault zone displaces Pliocene and possibly Pleistocene sediments in the vicinity of Boring, Oregon (Madin,1992; Lite, 1992). Relatively short faults define a 17-km-long fault zone that is apparently linked to the Grant Butte fault on the basis of stratigraphic relationships showing middle and late Pleistocene activity. Geomatrix (1995) assigns a probability of 0.5 for activity on structures within these fault zones.

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 May 24, 2023. A total of two exploratory test pits (TP-1 and TP-2) were excavated at the site using a backhoe to maximum depths of 7.25 feet below existing ground surface (bgs). Explorations were conducted under the full-time observation of a GeoPacific engineer. 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). At the completion of each test, the test pits were loosely backfilled with onsite soils.

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 in the following Soils Descriptions section.

5.1 Soil Descriptions

Topsoil: At the ground surface in all test pit locations, we observed organic SILT (ML-OL) which was brown and contained fine roots. This topsoil layer generally extended to depths of approximately 6 inches bgs. Topsoil depths are likely to increase where trees are present.

Catastrophic Flood Deposits: Underlying topsoil in all test pit locations, we encountered Catastrophic Flood Deposit soils. The upper portion of these soils typically consisted of native SILT

(ML) that was stiff and brown. In test pit TP-1, at a depth of approximately 6 feet bgs, the SILT (ML) graded to Silty COBBLES (GM) which were grayish brown and medium dense. Catastrophic Flood Deposits extended beyond the 7-foot maximum depth of exploration in test pit TP-1 and to a depth of approximately 7 feet bgs in test pit TP-2.

Basalt of Waverly Heights: Underlying the Catastrophic Flood Deposits in test pit TP-2, we encountered medium hard BASALT belonging to the Basalt of Waverly Heights formation. The BASALT extended beyond the 7.25-foot maximum depth of exploration in test pit TP-2.

5.2 Shrink-Swell Potential

Low-plasticity fine-grained soils and course-grained soils were encountered within the upper 7.25 feet of the test pit explorations conducted at the site. Based upon our observations and our local experience with the soil layers in the vicinity of the subject site, 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 to foundations as a result of potential soil expansion at this site.

5.3 Groundwater and Soil Moisture

On May 24, 2023, observed soil moisture conditions were generally moist. We did not encounter groundwater seepage within our explorations. According to a groundwater map of the Portland area, groundwater is expected within the site vicinity at a depth of approximately 20 feet bgs (Snyder 2008). It is anticipated that groundwater conditions will vary depending on the season, local subsurface conditions, changes in site utilization, and other factors. Perched groundwater may be encountered in localized areas. Seeps and springs may exist in areas not explored and may become evident during site grading.

5.1 Infiltration Testing

We performed soil infiltration testing within test pit TP-1 using the open-hole falling-head method. The approximate location of TP-1 is indicated on Figures 2 and 3. The test location was pre saturated prior to testing. During testing, we measured the water level to the nearest 0.01 foot (1/8 inch) from a fixed point and the change in water level was recorded at regular intervals until three successive measurements showing a consistent infiltration rate were achieved. The measured rates for these tests reflect vertical flow pathways. At a depth of approximately 6 feet bgs in test pit TP-1, the soils exhibited an infiltration rate of 2 inches per hour. Infiltration rates have been reported without applying a factor of safety. A factor of safety of 4 should be used in design.0.5

6.0 CONCLUSIONS AND RECOMMENDATIONS

Our site investigation indicates that the proposed development appears to be geotechnically feasible, provided that the recommendations of this report are incorporated into the design and construction phases of the project. The main geotechnical concern associated with the proposed site development is the presence of low-permeability soils in the near-surface soil profile. 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

Areas of proposed construction and areas to receive fill should be cleared of any organic and inorganic debris, disturbed soil, and loose stockpiled soils. Inorganic debris and organic materials from clearing should be removed from the site. Organic-rich soils and root zones should then be stripped from construction areas of the site or where engineered fill is to be placed. The average depth of stripping of existing organic topsoil is estimated to be approximately 6 inches at the site but may be deeper in the vicinity of trees and bushes.

The final depth of soil removal should be determined by the geotechnical engineer or designated representative during site inspection while stripping/excavation is being performed. Stripped topsoil should be removed from areas proposed for placement of engineered fill and structures. 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.

In areas of roadways, structures, or where engineered fill material is proposed, undocumented fills and any subsurface structures (dry wells, basements, driveway and landscaping fill, old utility lines, septic leach fields, etc.) should be completely removed and the excavations backfilled with engineered fill.

Site earthwork may be impacted by wet weather conditions. Stabilization of subgrade soils may require aeration and re-compaction. If subgrade soils are found to be difficult to stabilize, over-excavation, placement of granular soils, or cement treatment of subgrade soils may be feasible options. GeoPacific should be onsite to observe preparation of subgrade soil conditions prior to placement of engineered fill.

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 2021 International Building Code (IBC), and 2022 Oregon Structural Specialty Code (OSSC), Chapter 18 and Appendix J. Areas proposed for fill placement should be prepared as described in the section of this report titled *Site Preparation*. 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 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 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 to a depth of approximately 7 feet can generally be excavated using conventional heavy equipment. Below 7 feet bgs in test pit TP-2, we encountered medium-hard basaltic bedrock, which may present difficulties if excavations are planned below 7 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 silt soils in our explorations 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 applicable city and county 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, AASHTO T-99) 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 and topographic 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 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 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 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

We anticipate that the homes will be one to two stories tall, constructed with typical spread foundations and wood framing. We assume that the maximum structural loading on column footings and continuous strip footings will be on the order of 10 to 35 kips, and 2 to 4 kips respectively. We anticipate maximum cuts and fills will be on the order of 4 feet or less.

The proposed structures may be supported on shallow foundations bearing on native soils and/or engineered fill, appropriately designed and constructed as recommended in this report. Foundation design, construction, and setback requirements should conform to the applicable building code at the time of construction. For maximization of bearing strength and protection against frost heave, spread footings should be embedded at a minimum depth of 12 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 2,500 lbs/ft^2 for footings bearing on competent, native soil and/or engineered fill. The recommended maximum allowable bearing pressure may be increased by 1/3 for short-term transient conditions such as wind and seismic loading. The coefficient of friction between on-site soil and poured-in-place concrete may be taken as 0.42, which includes no factor of safety. The maximum anticipated total and differential footing movements (generally from soil expansion and/or settlement) are 1 inch and ¾ inch over a span of 20 feet, respectively. We anticipate that the majority of the estimated settlement will occur during construction, as loads are applied. Excavations near structural footings should not extend within a 1H:1V plane projected downward from the bottom edge of footings.

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

6.7 Concrete Slabs-on-Grade

Preparation of areas beneath concrete slab-on-grade floors should be performed as described in the *Site Preparation* and *Spread Foundations* sections of this report. 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 stiff, fine-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 3/4”-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.

6.8 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 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 52 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 Stormwater Management

We understand that plans for project development may include stormwater management facilities, and that it may be desired to incorporate subsurface disposal of stormwater. The native SILT with Sand (ML) and Silty COBBLES (GM) observed in the upper 7 feet of native soils within our explorations exhibited an infiltration rate of approximately 2 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. If a pervious pavement section is utilized onsite, a drainage pipe connected to a suitable outlet such as a stormwater facility or city stormwater system may be necessary to meet rainfall demands.

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.

7.0 SEISMIC DESIGN

The Oregon Department of Geology and Mineral Industries (DOGAMI), Oregon HazVu: 2022 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 2021 International Building Code (IBC) with applicable Oregon Structural Specialty Code (OSSC) revisions (current 2022). We recommend Site Class C 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 2022 Seismic Design Maps Summary Report are summarized in Table 1 and are based upon SPT blow counts from boring log data and soil conditions observed during field explorations.

Table 1: Recommended Earthquake Ground Motion Parameters (ASCE-7-16)

Parameter	Value
Location (Lat, Long), degrees	45.446, -122.646
Probabilistic Ground Motion Values, 2% Probability of Exceedance in 50 yrs	
Peak Ground Acceleration PGA_M	0.479 g
Short Period, S_s	0.886 g
1.0 Sec Period, S_1	0.392 g
Soil Factors for Site Class C:	
F_a	1.200
F_v	1.500
$SD_s = 2/3 \times F_a \times S_s$	0.709 g
$SD_1 = 2/3 \times F_v \times S_1$	0.392 g
Seismic Design Category	D

7.1 Soil Liquefaction

The Oregon Department of Geology and Mineral Industries (DOGAMI), Oregon HazVu: 2022 Statewide GeoHazards Viewer indicates that the site is not mapped as having risk of soil liquefaction during an earthquake. Soil liquefaction is a phenomenon wherein saturated soil deposits temporarily lose strength and behave as a liquid in response to ground shaking caused by strong earthquakes. Soil liquefaction is generally limited to loose sands and granular soils located below the water table, and fine-grained soils with a plasticity index less than 15.

Static groundwater was not encountered in our explorations, excavated to depths of up to 7.25 feet. Static groundwater is expected to be present at approximately 20 feet bgs in the vicinity of the site. Based on the mapped depth to groundwater, it is our opinion that the risk of damage to the proposed structures due to soil liquefaction is very low and that no special measures are needed to address the effects of liquefaction for the proposed development.

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.

REGISTERED PROFESSIONAL ENGINEER
14743PE
OREGON
JAN. 23, 1990
JAMES D. IMBRIE
EXPIRES: 06/30/2025

Alexandria B. Campbell, E.I.
Engineering Staff

James D. Imbrie, G.E.
Principal 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 Engineer	
2	Fill removal from site or sorting and stockpiling	Prior to mass stripping	Technician/ Geotechnical Engineer	
3	Stripping, aeration, and root-picking operations	During stripping	Technician	
4	Compaction testing of engineered fill	During filling, tested every 2 vertical feet minimum	Technician	
5	Foundation Subgrade Compaction	During foundation preparation, prior to placement of forms	Technician/ Geotechnical Engineer	
6	Compaction testing of trench backfill	During backfilling, tested every 2 to 4 vertical feet for every 200 linear feet	Technician	
7	Street subgrade inspection	Prior to placing base course	Technician	
8	Base course compaction	Prior to paving, tested every 100 - 200 linear feet	Technician	
9	Base course proof roll	Prior to paving	Technician	
10	Asphalt Compaction	During paving, tested every 100 linear feet	Technician	
11	Final Geotechnical Engineer's Report	Completion of project	Geotechnical Engineer	

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FIGURES

GEOPACIFIC

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

SITE VICINITY MAP

Base map obtained from Oregon HazVu street map

Date: 05/30/23

Drawn by: ABC

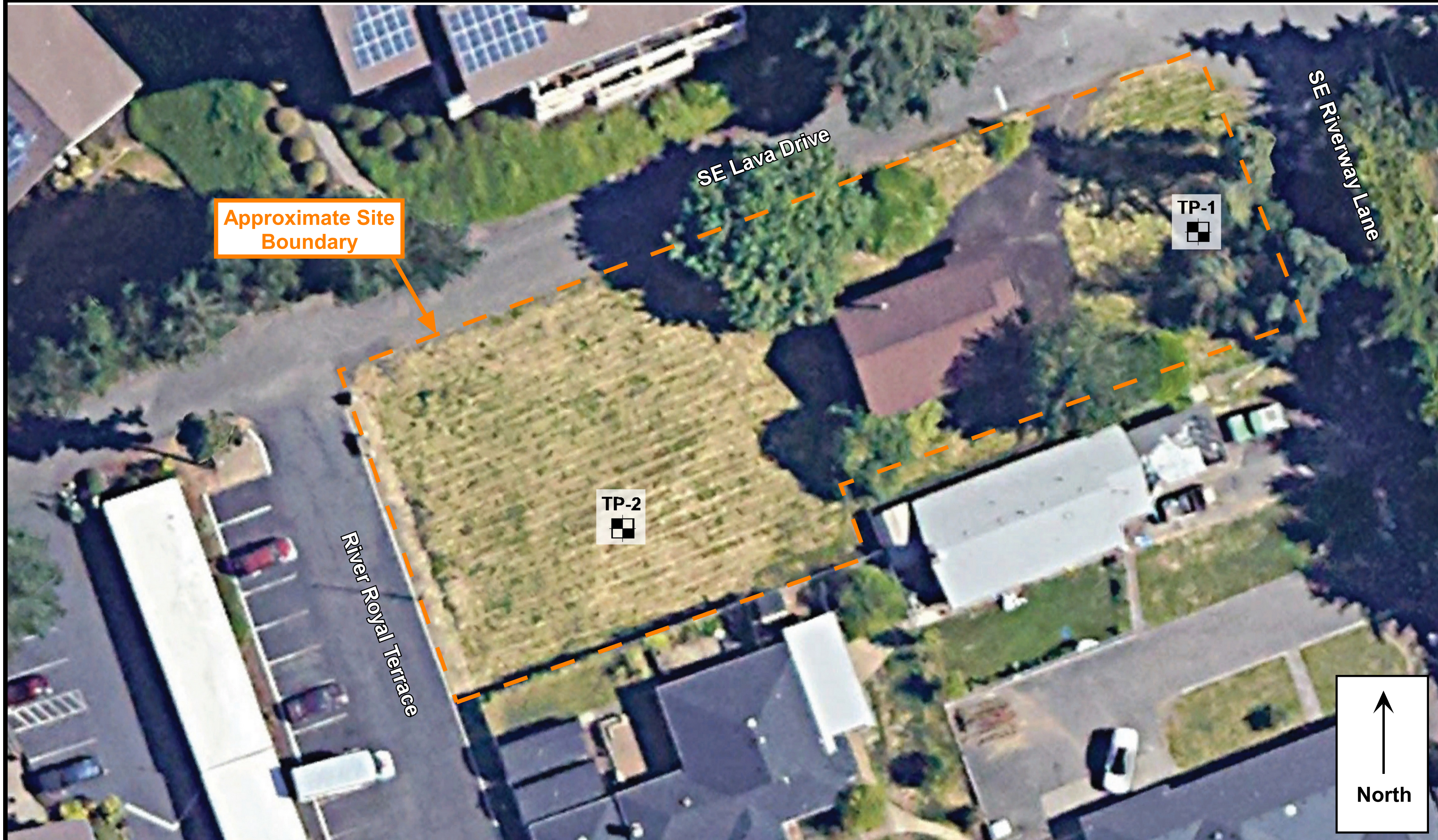
0 600

Approximate Scale: 1" = 600'

Project: Lava Drive Apartments Milwaukie, Oregon	Project No. 23-6332	FIGURE 1

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

SITE AERIAL AND EXPLORATION LOCATIONS



Legend: TP-1 Test Pit Designation and Approximate Location

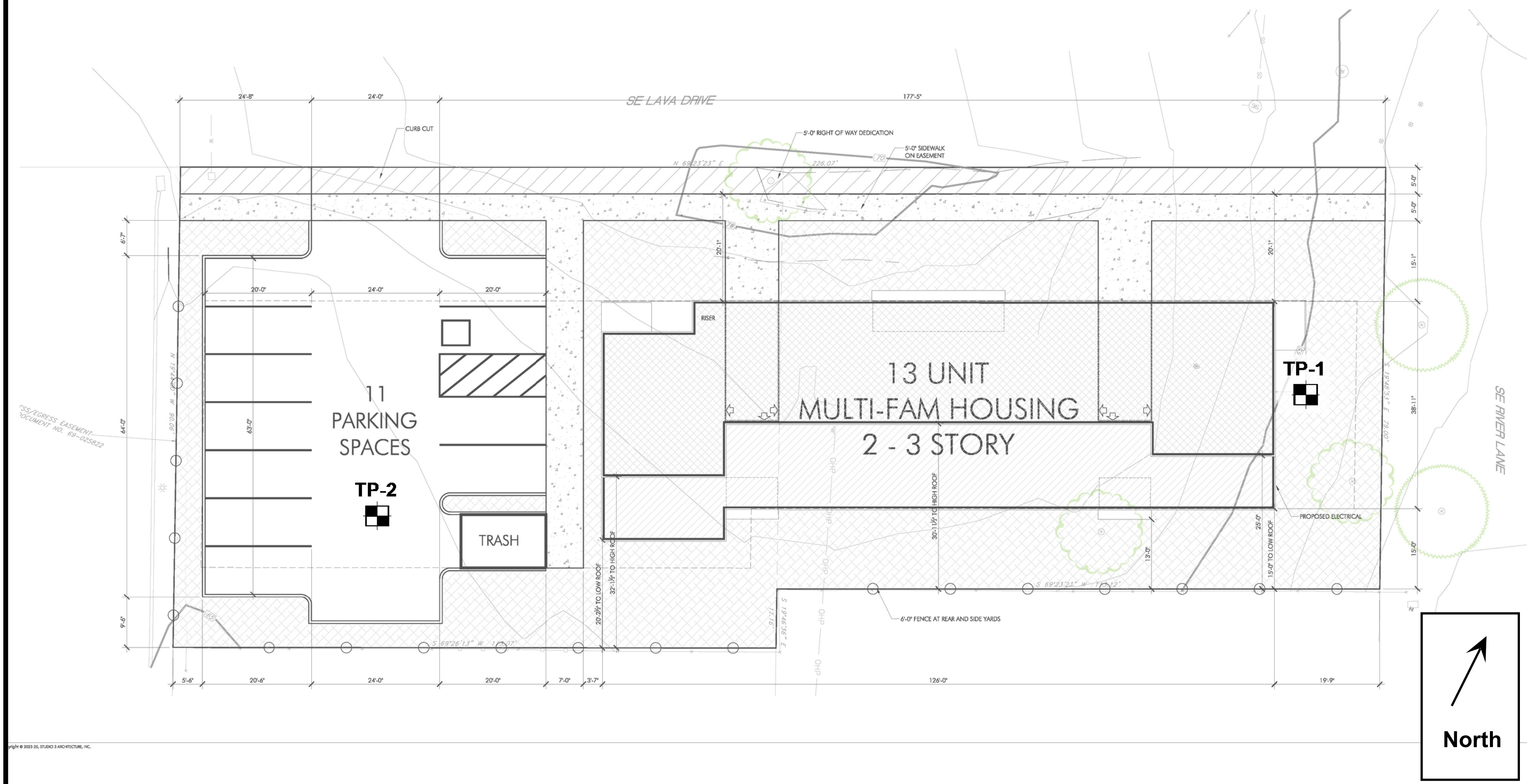
0 40'
APPROXIMATE SCALE 1"=40'

Date: 05/30/23
Drawn by: ABC

Project: Lava Drive Apartments Milwaukie, Oregon	Project No. 23-6332	FIGURE 2

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

SITE PLAN AND EXPLORATION LOCATIONS



Legend: **TP-1** Test Pit Designation and Approximate Location

0 30'
APPROXIMATE SCALE 1"=30'

Date: 05/30/23
Drawn by: ABC

Project: Lava Drive Apartments
Milwaukie, Oregon

Project No. 23-6332

FIGURE 3

EXPLORATION LOGS

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

TEST PIT LOG

Project: Lava Drive Apartments, 1600 SE Lava Drive, Milwaukie, Oregon	Project No. 23-6332	Test Pit No. **TP-1**

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
0–0.5							6" Organic SILT (OL-ML), brown, moist, fine roots throughout (Topsoil)
0.5–6							SILT (ML), brown, stiff, moist (Catastrophic Flood Deposits) Infiltration test conducted at 6 feet bgs. Infiltration rate observed as 2.0 in/hr
6–7							Silty COBBLES (GM), grayish brown, medium dense, moist (Catastrophic Flood Deposits)
7							Test pit terminated at 7 feet bgs. No groundwater seepage 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: 05/24/2023
Logged By: ABC
Surface Elevation: 183 Feet

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

TEST PIT LOG

Project: Lava Drive Apartments 1600 SE Lava Drive Milwaukie, Oregon	Project No. 23-6332	Test Pit No. **TP-2**

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							6" Organic SILT (OL-ML), brown, moist, fine roots throughout (Topsoil)
0.5–6.5							SILT (ML), brown, stiff, moist (Catastrophic Flood Deposits)
6.5–7.25							BASALT, gray, medium hard (R3), moist (Basalt of Waverly Heights Formation)
							Test pit terminated at 7.25 feet bgs. No groundwater seepage 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: 05/24/2023
Logged By: ABC
Surface Elevation: 183 Feet

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SITE RESEARCH

This is a beta release of the new ATC Hazards by Location website. Please contact us with feedback.

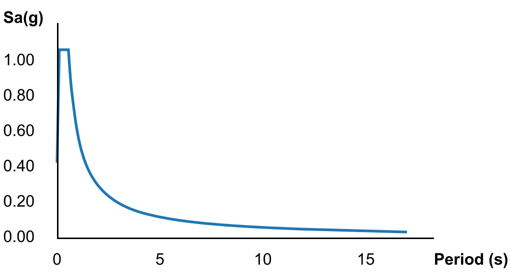
The ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

ATC Hazards by Location

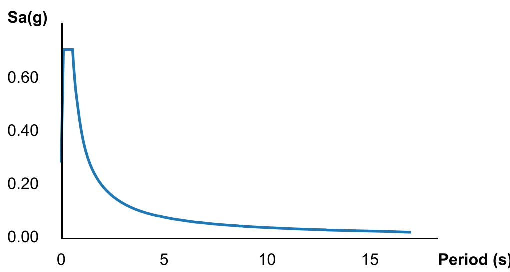
Search Information

Address:	1600 SE Lava Dr, Milwaukie, OR 97222, USA
Coordinates:	45.4463562, -122.6464097
Elevation:	68 ft
Timestamp:	2023-05-30T17:26:20.288Z
Hazard Type:	Seismic
Reference Document:	NEHRP-2015
Risk Category:	II
Site Class:	C

MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	0.886	MCE_R ground motion (period=0.2s)
S_1	0.392	MCE_R ground motion (period=1.0s)
S_{MS}	1.064	Site-modified spectral acceleration value
S_{M1}	0.588	Site-modified spectral acceleration value
S_{DS}	0.709	Numeric seismic design value at 0.2s SA
S_{D1}	0.392	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	D	Seismic design category
F_a	1.2	Site amplification factor at 0.2s
F_v	1.5	Site amplification factor at 1.0s
CR_S	0.89	Coefficient of risk (0.2s)
CR_1	0.871	Coefficient of risk (1.0s)
PGA	0.399	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.479	Site modified peak ground acceleration
T_L	16	Long-period transition period (s)
SsRT	0.886	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.997	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.392	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.451	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)

S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

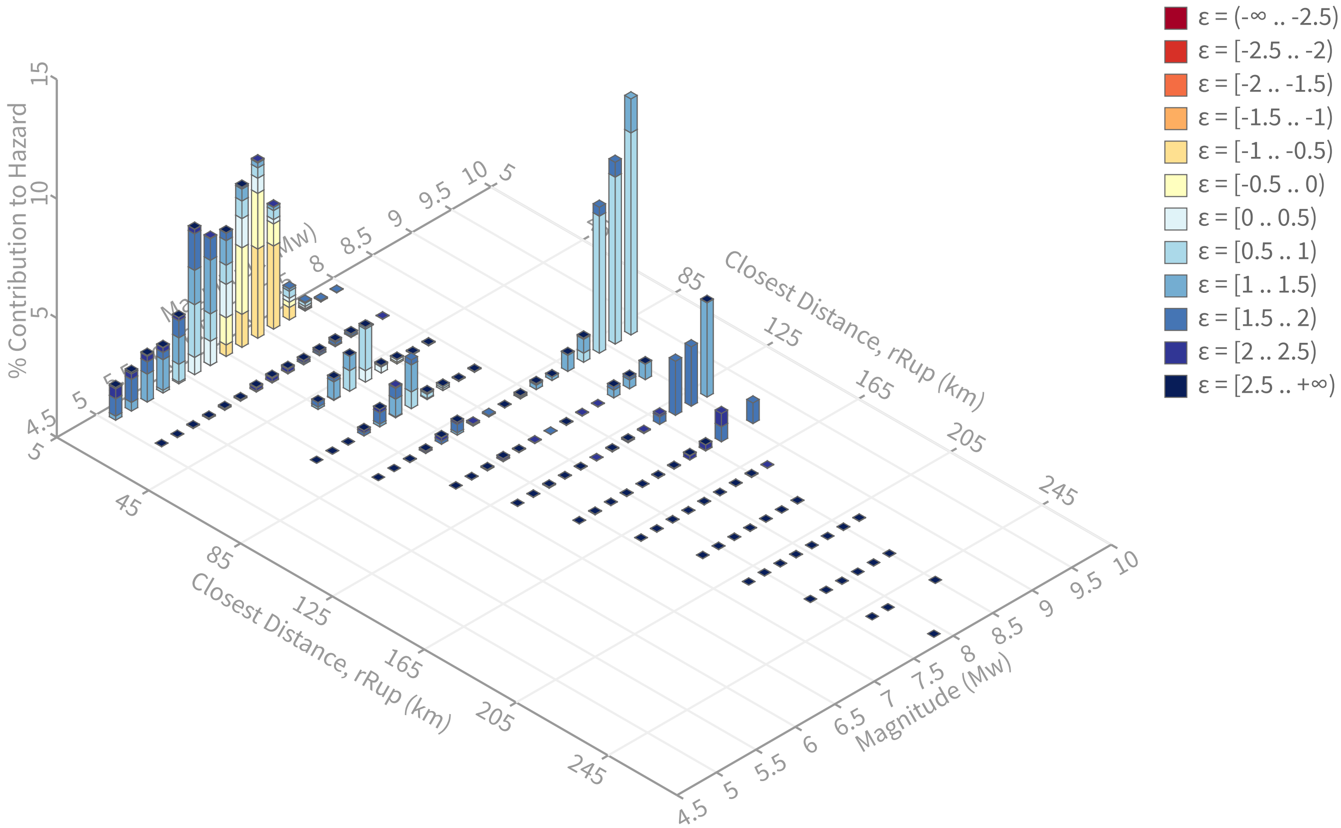
The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

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.



ε = (-∞ .. -2.5)
ε = [-2.5 .. -2)
ε = [-2 .. -1.5)
ε = [-1.5 .. -1)
ε = [-1 .. -0.5)
ε = [-0.5 .. 0)
ε = [0 .. 0.5)
ε = [0.5 .. 1)
ε = [1 .. 1.5)
ε = [1.5 .. 2)
ε = [2 .. 2.5)
ε = [2.5 .. +∞)
% Contribution to Hazard
Closest Distance, rRup (km)
Magnitude (Mw)

Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.42099019 g

Recovered targets

Return period: 2534.2251 yrs
Exceedance rate: 0.00039459794 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.39 %

Mean (over all sources)

m: 7.5
r: 50.37 km
ε_0: 0.87 σ

Mode (largest m-r bin)

m: 9.34
r: 82.27 km
ε_0: 0.72 σ
Contribution: 9.89 %

Mode (largest m-r-ε_0 bin)

m: 9.34
r: 82.27 km
ε_0: 0.62 σ
Contribution: 8.47 %

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							23.20
Cascadia Megathrust - whole CSZ Characteristic		82.27	9.11	0.84	123.599°W	45.501°N	275.05	23.20
Geologic Model Partial Rupture	Fault							9.83
Portland Hills		2.87	6.75	-0.27	122.620°W	45.436°N	120.49	9.75
sub0_ch_mid.in	Interface							8.44
Cascadia Megathrust - whole CSZ Characteristic		132.27	8.93	1.59	124.330°W	45.489°N	272.68	8.44
coastalOR_deep.in	Slab							7.53
Geologic Model Small Mag	Fault							6.92
Grant Butte 50		8.64	6.19	1.41	122.544°W	45.476°N	67.48	2.99
Bolton		5.05	6.16	0.59	122.663°W	45.402°N	194.86	1.80
Grant Butte 65		8.64	6.19	1.41	122.544°W	45.476°N	67.48	1.00
Geologic Model Full Rupture	Fault							5.18
Portland Hills		1.22	7.00	-0.58	122.620°W	45.436°N	120.49	5.11
WUSmap_2014_fixSm.ch.in (opt)	Grid							3.99
PointSourceFinite: -122.646, 45.505		7.88	5.90	0.97	122.646°W	45.505°N	0.00	1.82
noPuget_2014_fixSm.ch.in (opt)	Grid							3.99
PointSourceFinite: -122.646, 45.505		7.88	5.90	0.97	122.646°W	45.505°N	0.00	1.82
WUSmap_2014_fixSm.gr.in (opt)	Grid							3.67
PointSourceFinite: -122.646, 45.505		8.00	5.84	1.02	122.646°W	45.505°N	0.00	1.71
noPuget_2014_fixSm.gr.in (opt)	Grid							3.67
PointSourceFinite: -122.646, 45.505		8.00	5.84	1.02	122.646°W	45.505°N	0.00	1.71
coastalOR_deep.in	Slab							1.86
Zeng Model Partial Rupture	Fault							1.81
Portland Hills		2.87	6.75	-0.27	122.620°W	45.436°N	120.49	1.79
sub0_ch_top.in	Interface							1.76
Cascadia Megathrust - whole CSZ Characteristic		149.29	8.84	1.87	124.549°W	45.485°N	272.32	1.76
Zeng Model Small Mag	Fault							1.26
WUSmap_2014_fixSm_M8.in (opt)	Grid							1.22
noPuget_2014_fixSm_M8.in (opt)	Grid							1.22
sub2_ch_bot.in	Interface							1.15
Cascadia Megathrust - Goldfinger Case C Characteristic		100.78	8.74	1.31	123.702°W	45.000°N	239.40	1.15

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
sub1_ch_bot.in	Interface							1.01
Cascadia Megathrust - Goldfinger Case B Characteristic		81.63	8.86	0.97	123.599°W	45.501°N	275.05	1.01

Exhibit L – Geotechnical Report

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Geotechnical Engineering Report

Project Information:	**Lava Drive Apartments** **GeoPacific Project № 23-6332** **May 30, 2023**
Site Location:	**1600 SE Lava Drive** **Clackamas County Taxlot: 11E35AB 100 & 502** **Milwaukie, OR 97206**
Client:	**WDC Properties** **2330 NW 31st Avenue** **Portland, OR 97210** **Email: fstock@wdcproperties.com**

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

TABLE OF CONTENTS

List of Appendices

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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 contract dated , dated May 24, 2023, and your subsequent authorization of our proposal and *General Conditions for Geotechnical Services*.

2.0 SITE AND PROJECT DESCRIPTION

The site is located to the south of SE Lava Drive west of the intersection with SE Riverway Lane in the City of Milwaukie, Oregon. The eastern portion of the property is currently occupied by a single-family residence and associated driveway. The western portion of the site is currently undeveloped. The site is gently sloping down to the east with site elevations ranging from 62 to 67. Vegetation onsite consists of short grasses, shrubs, and medium-sized trees. The site is bordered by single-family residences to the south and west, by SE Lava Drive to the north, and by SE Riverway Lane to the east.

It is our understanding that a to 3-story apartment building will be constructed in the eastern portion of the site. Associated parking areas, driveways, and underground utilities are also planned. It is anticipated that the structures will be founded on conventional shallow foundations. A grading plan has not yet been provided for our review. However, we anticipate that cuts and fills will be on the order of 4 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 subject site is underlain by the Quaternary age (last 1.6 million years) Catastrophic Flood Deposits associated with repeated glacial outburst flooding of the Willamette Valley (Madin, 1990). The last of these outburst floods occurred about 10,000 years ago. These deposits typically consist of sand to coarse gravel and cobbles. Regional studies indicate that the thickness of the Catastrophic Flood Deposites in the vicinity of the subject site is approximately 60 feet (Madin, 1990).

Regional geologic mapping indicates the Catastrophic Flood Deposits are underlain by Eocene age (34 to 55 million years ago) Basalt of Waverly Heights (Beeson et al., 1989 and Madin, 1990). Basalt of Waverly Heights are a dense, vesicular, and finely crystalline rock with secondary mineralization. Interflow zones are well developed, vesicular, and commonly include sedimentary deposits. The Basalt of Waverly Heights can be distinguished from the Columbia River Basalt

Group by its darker color, secondary mineralization within vesicles, and mineralogical composition. The top of the Waverly Heights Basalt typically includes a highly weathered rock/residual soil layer up to 30 feet thick which is generally thin or absent in areas of erosional scour that occurred during catastrophic flooding events (Beeson et al., 1989).

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 Grant Butte and Damascus-Trickle Creek Fault 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 approximately 0.8 miles northeast of the site. The East Bank Fault is oriented roughly parallel to the Portland Hills Fault, on the east bank of the Willamette River, and is located approximately 4.8 miles north of the site. The Oatfield Fault occurs along the western side of the Portland Hills and is approximately 1.3 miles southwest of the site. The Oatfield Fault is considered to be potentially seismogenic (Wong, et al., 2000). Madin and Mabey (1996) indicate the Portland Hills Fault Zone has experienced Late Quaternary (last 780,000 years) fault movement; however, movement has not been detected in the last 20,000 years. 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 Grant Butte and Damascus-Trickle Creek Fault Zone

The Grant Butte fault zone was mapped along the north side of Mt. Scott and Powell Butte by Madin (1990). The fault is approximately 8.6 miles northeast of the subject site and extends eastward to Grant Butte on the basis of mapping by CH2M Hill and others (1991) and informally named the Grant Butte fault (Cornforth and Geomatrix, 1992). The Damascus-Trickle Creek fault zone displaces Pliocene and possibly Pleistocene sediments in the vicinity of Boring, Oregon (Madin,1992; Lite, 1992). Relatively short faults define a 17-km-long fault zone that is apparently linked to the Grant Butte fault on the basis of stratigraphic relationships showing middle and late Pleistocene activity. Geomatrix (1995) assigns a probability of 0.5 for activity on structures within these fault zones.

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 May 24, 2023. A total of two exploratory test pits (TP-1 and TP-2) were excavated at the site using a backhoe to maximum depths of 7.25 feet below existing ground surface (bgs). Explorations were conducted under the full-time observation of a GeoPacific engineer. 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). At the completion of each test, the test pits were loosely backfilled with onsite soils.

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 in the following Soils Descriptions section.

5.1 Soil Descriptions

Topsoil: At the ground surface in all test pit locations, we observed organic SILT (ML-OL) which was brown and contained fine roots. This topsoil layer generally extended to depths of approximately 6 inches bgs. Topsoil depths are likely to increase where trees are present.

Catastrophic Flood Deposits: Underlying topsoil in all test pit locations, we encountered Catastrophic Flood Deposit soils. The upper portion of these soils typically consisted of native SILT

(ML) that was stiff and brown. In test pit TP-1, at a depth of approximately 6 feet bgs, the SILT (ML) graded to Silty COBBLES (GM) which were grayish brown and medium dense. Catastrophic Flood Deposits extended beyond the 7-foot maximum depth of exploration in test pit TP-1 and to a depth of approximately 7 feet bgs in test pit TP-2.

Basalt of Waverly Heights: Underlying the Catastrophic Flood Deposits in test pit TP-2, we encountered medium hard BASALT belonging to the Basalt of Waverly Heights formation. The BASALT extended beyond the 7.25-foot maximum depth of exploration in test pit TP-2.

5.2 Shrink-Swell Potential

Low-plasticity fine-grained soils and course-grained soils were encountered within the upper 7.25 feet of the test pit explorations conducted at the site. Based upon our observations and our local experience with the soil layers in the vicinity of the subject site, 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 to foundations as a result of potential soil expansion at this site.

5.3 Groundwater and Soil Moisture

On May 24, 2023, observed soil moisture conditions were generally moist. We did not encounter groundwater seepage within our explorations. According to a groundwater map of the Portland area, groundwater is expected within the site vicinity at a depth of approximately 20 feet bgs (Snyder 2008). It is anticipated that groundwater conditions will vary depending on the season, local subsurface conditions, changes in site utilization, and other factors. Perched groundwater may be encountered in localized areas. Seeps and springs may exist in areas not explored and may become evident during site grading.

5.1 Infiltration Testing

We performed soil infiltration testing within test pit TP-1 using the open-hole falling-head method. The approximate location of TP-1 is indicated on Figures 2 and 3. The test location was pre saturated prior to testing. During testing, we measured the water level to the nearest 0.01 foot (1/8 inch) from a fixed point and the change in water level was recorded at regular intervals until three successive measurements showing a consistent infiltration rate were achieved. The measured rates for these tests reflect vertical flow pathways. At a depth of approximately 6 feet bgs in test pit TP-1, the soils exhibited an infiltration rate of 2 inches per hour. Infiltration rates have been reported without applying a factor of safety. A factor of safety of 4 should be used in design.0.5

6.0 CONCLUSIONS AND RECOMMENDATIONS

Our site investigation indicates that the proposed development appears to be geotechnically feasible, provided that the recommendations of this report are incorporated into the design and construction phases of the project. The main geotechnical concern associated with the proposed site development is the presence of low-permeability soils in the near-surface soil profile. 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

Areas of proposed construction and areas to receive fill should be cleared of any organic and inorganic debris, disturbed soil, and loose stockpiled soils. Inorganic debris and organic materials from clearing should be removed from the site. Organic-rich soils and root zones should then be stripped from construction areas of the site or where engineered fill is to be placed. The average depth of stripping of existing organic topsoil is estimated to be approximately 6 inches at the site but may be deeper in the vicinity of trees and bushes.

The final depth of soil removal should be determined by the geotechnical engineer or designated representative during site inspection while stripping/excavation is being performed. Stripped topsoil should be removed from areas proposed for placement of engineered fill and structures. 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.

In areas of roadways, structures, or where engineered fill material is proposed, undocumented fills and any subsurface structures (dry wells, basements, driveway and landscaping fill, old utility lines, septic leach fields, etc.) should be completely removed and the excavations backfilled with engineered fill.

Site earthwork may be impacted by wet weather conditions. Stabilization of subgrade soils may require aeration and re-compaction. If subgrade soils are found to be difficult to stabilize, over-excavation, placement of granular soils, or cement treatment of subgrade soils may be feasible options. GeoPacific should be onsite to observe preparation of subgrade soil conditions prior to placement of engineered fill.

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 2021 International Building Code (IBC), and 2022 Oregon Structural Specialty Code (OSSC), Chapter 18 and Appendix J. Areas proposed for fill placement should be prepared as described in the section of this report titled *Site Preparation*. 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 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 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 to a depth of approximately 7 feet can generally be excavated using conventional heavy equipment. Below 7 feet bgs in test pit TP-2, we encountered medium-hard basaltic bedrock, which may present difficulties if excavations are planned below 7 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 silt soils in our explorations 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 applicable city and county 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, AASHTO T-99) 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 and topographic 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 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 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 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

We anticipate that the homes will be one to two stories tall, constructed with typical spread foundations and wood framing. We assume that the maximum structural loading on column footings and continuous strip footings will be on the order of 10 to 35 kips, and 2 to 4 kips respectively. We anticipate maximum cuts and fills will be on the order of 4 feet or less.

The proposed structures may be supported on shallow foundations bearing on native soils and/or engineered fill, appropriately designed and constructed as recommended in this report. Foundation design, construction, and setback requirements should conform to the applicable building code at the time of construction. For maximization of bearing strength and protection against frost heave, spread footings should be embedded at a minimum depth of 12 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 2,500 lbs/ft^2 for footings bearing on competent, native soil and/or engineered fill. The recommended maximum allowable bearing pressure may be increased by 1/3 for short-term transient conditions such as wind and seismic loading. The coefficient of friction between on-site soil and poured-in-place concrete may be taken as 0.42, which includes no factor of safety. The maximum anticipated total and differential footing movements (generally from soil expansion and/or settlement) are 1 inch and ¾ inch over a span of 20 feet, respectively. We anticipate that the majority of the estimated settlement will occur during construction, as loads are applied. Excavations near structural footings should not extend within a 1H:1V plane projected downward from the bottom edge of footings.

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

6.7 Concrete Slabs-on-Grade

Preparation of areas beneath concrete slab-on-grade floors should be performed as described in the *Site Preparation* and *Spread Foundations* sections of this report. 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 stiff, fine-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 3/4”-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.

6.8 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 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 52 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 Stormwater Management

We understand that plans for project development may include stormwater management facilities, and that it may be desired to incorporate subsurface disposal of stormwater. The native SILT with Sand (ML) and Silty COBBLES (GM) observed in the upper 7 feet of native soils within our explorations exhibited an infiltration rate of approximately 2 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. If a pervious pavement section is utilized onsite, a drainage pipe connected to a suitable outlet such as a stormwater facility or city stormwater system may be necessary to meet rainfall demands.

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.

7.0 SEISMIC DESIGN

The Oregon Department of Geology and Mineral Industries (DOGAMI), Oregon HazVu: 2022 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 2021 International Building Code (IBC) with applicable Oregon Structural Specialty Code (OSSC) revisions (current 2022). We recommend Site Class C 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 2022 Seismic Design Maps Summary Report are summarized in Table 1 and are based upon SPT blow counts from boring log data and soil conditions observed during field explorations.

Table 1: Recommended Earthquake Ground Motion Parameters (ASCE-7-16)

Parameter	Value
Location (Lat, Long), degrees	45.446, -122.646
Probabilistic Ground Motion Values, 2% Probability of Exceedance in 50 yrs	
Peak Ground Acceleration PGA_M	0.479 g
Short Period, S_s	0.886 g
1.0 Sec Period, S_1	0.392 g
Soil Factors for Site Class C:	
F_a	1.200
F_v	1.500
$SD_s = 2/3 \times F_a \times S_s$	0.709 g
$SD_1 = 2/3 \times F_v \times S_1$	0.392 g
Seismic Design Category	D

7.1 Soil Liquefaction

The Oregon Department of Geology and Mineral Industries (DOGAMI), Oregon HazVu: 2022 Statewide GeoHazards Viewer indicates that the site is not mapped as having risk of soil liquefaction during an earthquake. Soil liquefaction is a phenomenon wherein saturated soil deposits temporarily lose strength and behave as a liquid in response to ground shaking caused by strong earthquakes. Soil liquefaction is generally limited to loose sands and granular soils located below the water table, and fine-grained soils with a plasticity index less than 15.

Static groundwater was not encountered in our explorations, excavated to depths of up to 7.25 feet. Static groundwater is expected to be present at approximately 20 feet bgs in the vicinity of the site. Based on the mapped depth to groundwater, it is our opinion that the risk of damage to the proposed structures due to soil liquefaction is very low and that no special measures are needed to address the effects of liquefaction for the proposed development.

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.

REGISTERED PROFESSIONAL ENGINEER
14743PE
OREGON
JAN. 23, 1990
JAMES D. IMBRIE
EXPIRES: 06/30/2025

Alexandria B. Campbell, E.I.
Engineering Staff

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Principal 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 Engineer	
2	Fill removal from site or sorting and stockpiling	Prior to mass stripping	Technician/ Geotechnical Engineer	
3	Stripping, aeration, and root-picking operations	During stripping	Technician	
4	Compaction testing of engineered fill	During filling, tested every 2 vertical feet minimum	Technician	
5	Foundation Subgrade Compaction	During foundation preparation, prior to placement of forms	Technician/ Geotechnical Engineer	
6	Compaction testing of trench backfill	During backfilling, tested every 2 to 4 vertical feet for every 200 linear feet	Technician	
7	Street subgrade inspection	Prior to placing base course	Technician	
8	Base course compaction	Prior to paving, tested every 100 - 200 linear feet	Technician	
9	Base course proof roll	Prior to paving	Technician	
10	Asphalt Compaction	During paving, tested every 100 linear feet	Technician	
11	Final Geotechnical Engineer's Report	Completion of project	Geotechnical Engineer	

FIGURES

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

SITE VICINITY MAP

Base map obtained from Oregon HazVu street map

Date: 05/30/23

Drawn by: ABC

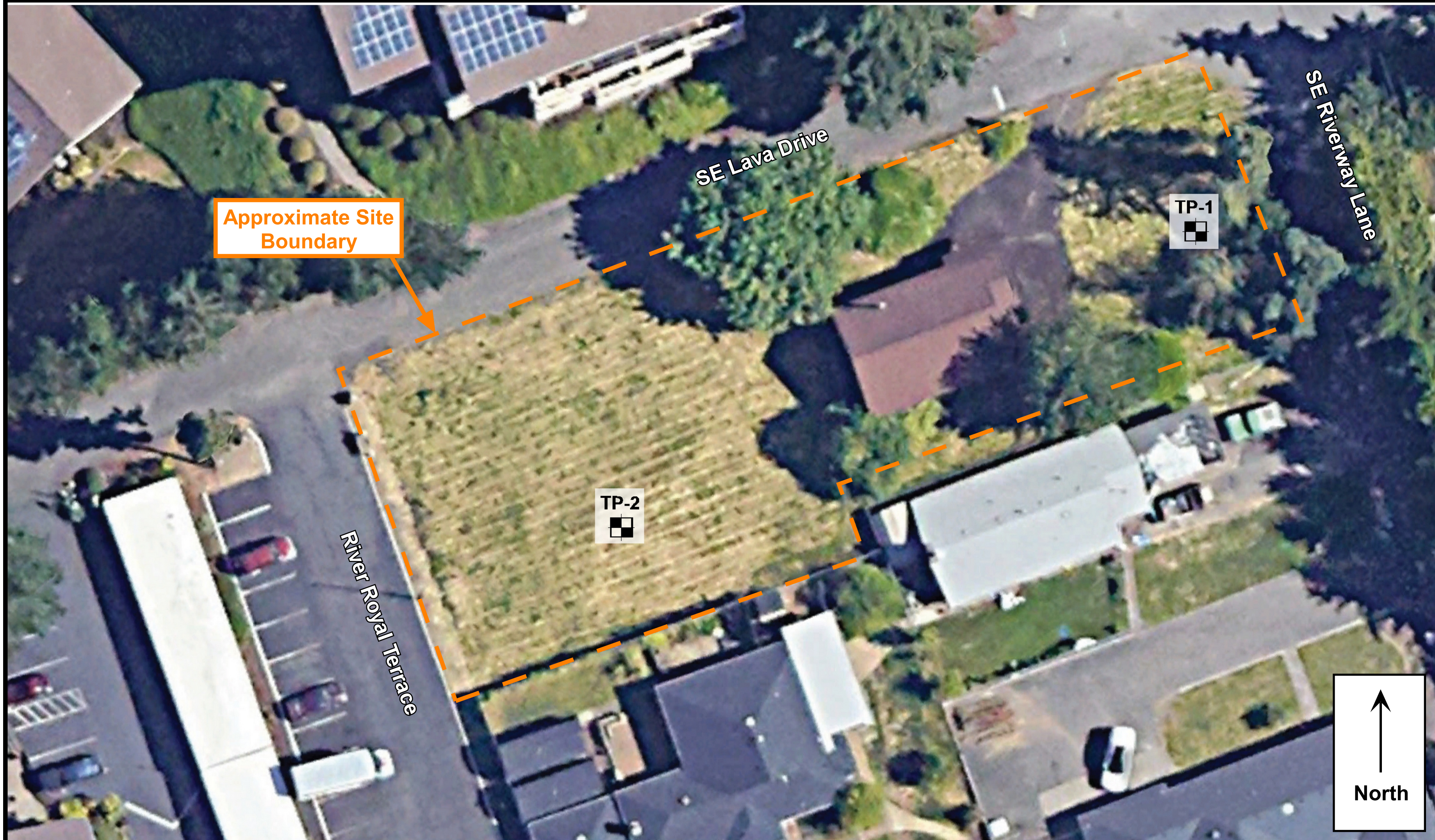
0 600

Approximate Scale: 1" = 600'

Project: Lava Drive Apartments Milwaukie, Oregon	Project No. 23-6332	FIGURE 1

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

SITE AERIAL AND EXPLORATION LOCATIONS



Legend: TP-1 Test Pit Designation and Approximate Location

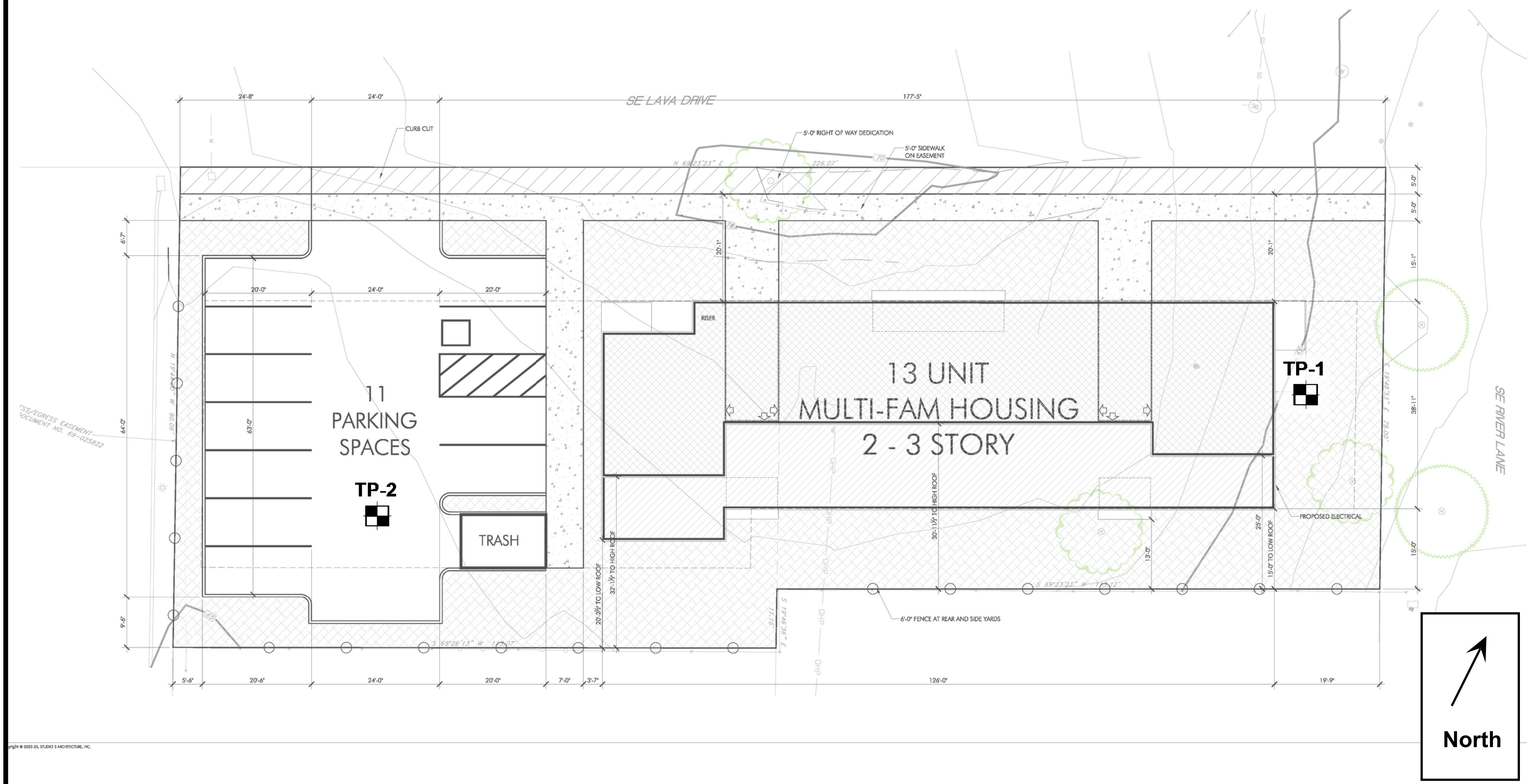
0 40'
APPROXIMATE SCALE 1"=40'

Date: 05/30/23
Drawn by: ABC

Project: Lava Drive Apartments Milwaukie, Oregon	Project No. 23-6332	FIGURE 2

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

SITE PLAN AND EXPLORATION LOCATIONS



Legend: TP-1 Test Pit Designation and Approximate Location

0 30'
APPROXIMATE SCALE 1"=30'

Date: 05/30/23
Drawn by: ABC

Project: Lava Drive Apartments Milwaukie, Oregon	Project No. 23-6332	FIGURE 3

Real-World Geotechnical Solutions
Investigation • Design • Construction Support

EXPLORATION LOGS

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

TEST PIT LOG

Project: Lava Drive Apartments, 1600 SE Lava Drive, Milwaukie, Oregon	Project No. 23-6332	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							6" Organic SILT (OL-ML), brown, moist, fine roots throughout (Topsoil)
0.5–6							SILT (ML), brown, stiff, moist (Catastrophic Flood Deposits) Infiltration test conducted at 6 feet bgs. Infiltration rate observed as 2.0 in/hr
6–7							Silty COBBLES (GM), grayish brown, medium dense, moist (Catastrophic Flood Deposits)
7							Test pit terminated at 7 feet bgs. No groundwater seepage 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: 05/24/2023
Logged By: ABC
Surface Elevation: 183 Feet

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

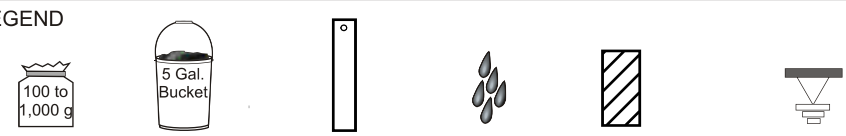
TEST PIT LOG

Project: Lava Drive Apartments 1600 SE Lava Drive Milwaukie, Oregon	Project No. 23-6332	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
0–0.5							6" Organic SILT (OL-ML), brown, moist, fine roots throughout (Topsoil)
0.5–6.5							SILT (ML), brown, stiff, moist (Catastrophic Flood Deposits)
6.5–7.25							BASALT, gray, medium hard (R3), moist (Basalt of Waverly Heights Formation)
							Test pit terminated at 7.25 feet bgs. No groundwater seepage observed

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

LEGEND



Bag Sample | Bucket Sample | Shelby Tube Sample | Seepage | Water Bearing Zone | Water Level at Abandonment

Date Excavated: 05/24/2023
Logged By: ABC
Surface Elevation: 183 Feet

GEOPACIFIC

Real-World Geotechnical Solutions
Investigation • Design • Construction Support

SITE RESEARCH

This is a beta release of the new ATC Hazards by Location website. Please contact us with feedback.

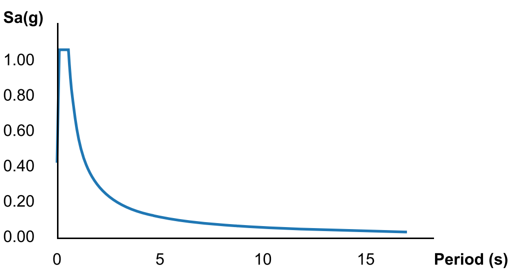
The ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

Hazards by Location

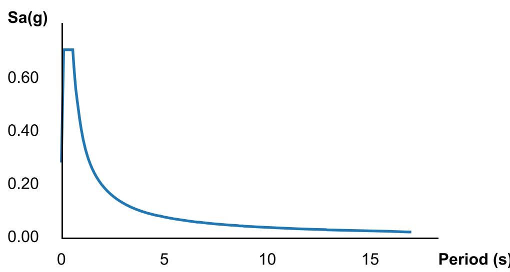
Search Information

Address:	1600 SE Lava Dr, Milwaukie, OR 97222, USA
Coordinates:	45.4463562, -122.6464097
Elevation:	68 ft
Timestamp:	2023-05-30T17:26:20.288Z
Hazard Type:	Seismic
Reference Document:	NEHRP-2015
Risk Category:	II
Site Class:	C

MCER Horizontal Response Spectrum



Design Horizontal Response Spectrum



Basic Parameters

Name	Value	Description
S_S	0.886	MCE_R ground motion (period=0.2s)
S_1	0.392	MCE_R ground motion (period=1.0s)
S_{MS}	1.064	Site-modified spectral acceleration value
S_{M1}	0.588	Site-modified spectral acceleration value
S_{DS}	0.709	Numeric seismic design value at 0.2s SA
S_{D1}	0.392	Numeric seismic design value at 1.0s SA

Additional Information

Name	Value	Description
SDC	D	Seismic design category
F_a	1.2	Site amplification factor at 0.2s
F_v	1.5	Site amplification factor at 1.0s
CR_S	0.89	Coefficient of risk (0.2s)
CR_1	0.871	Coefficient of risk (1.0s)
PGA	0.399	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.479	Site modified peak ground acceleration
T_L	16	Long-period transition period (s)
SsRT	0.886	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.997	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.392	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.451	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)

S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

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.

Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

Disclaimer

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

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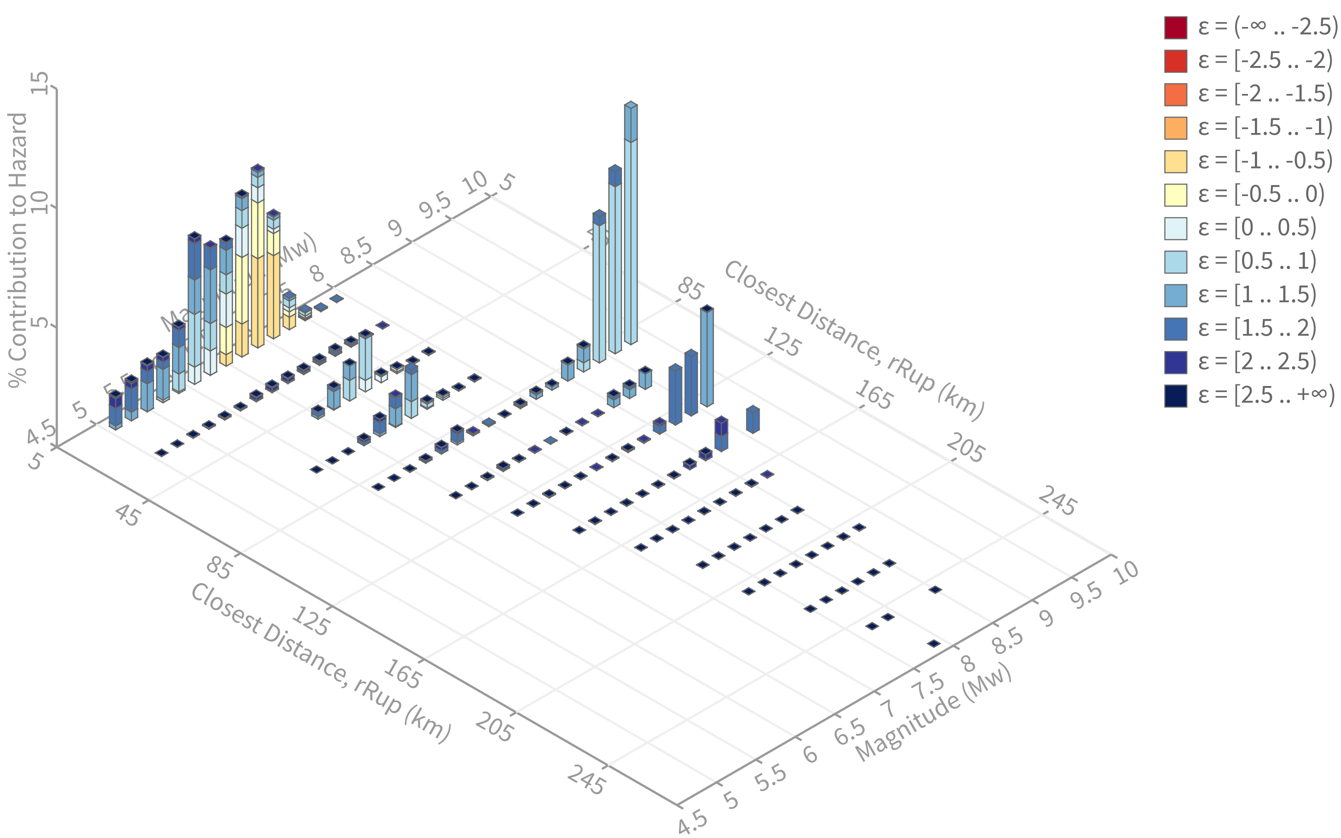
% Contribution to Hazard
Closest Distance, rRup (km)
Magnitude (Mw)
ε = (-∞ .. -2.5)
ε = [-2.5 .. -2)
ε = [-2 .. -1.5)
ε = [-1.5 .. -1)
ε = [-1 .. -0.5)
ε = [-0.5 .. 0)
ε = [0 .. 0.5)
ε = [0.5 .. 1)
ε = [1 .. 1.5)
ε = [1.5 .. 2)
ε = [2 .. 2.5)
ε = [2.5 .. +∞)

Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.42099019 g

Recovered targets

Return period: 2534.2251 yrs
Exceedance rate: 0.00039459794 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.39 %

Mean (over all sources)

m: 7.5
r: 50.37 km
ε_0: 0.87 σ

Mode (largest m-r bin)

m: 9.34
r: 82.27 km
ε_0: 0.72 σ
Contribution: 9.89 %

Mode (largest m-r-ε_0 bin)

m: 9.34
r: 82.27 km
ε_0: 0.62 σ
Contribution: 8.47 %

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							23.20
Cascadia Megathrust - whole CSZ Characteristic		82.27	9.11	0.84	123.599°W	45.501°N	275.05	23.20
Geologic Model Partial Rupture	Fault							9.83
Portland Hills		2.87	6.75	-0.27	122.620°W	45.436°N	120.49	9.75
sub0_ch_mid.in	Interface							8.44
Cascadia Megathrust - whole CSZ Characteristic		132.27	8.93	1.59	124.330°W	45.489°N	272.68	8.44
coastalOR_deep.in	Slab							7.53
Geologic Model Small Mag	Fault							6.92
Grant Butte 50		8.64	6.19	1.41	122.544°W	45.476°N	67.48	2.99
Bolton		5.05	6.16	0.59	122.663°W	45.402°N	194.86	1.80
Grant Butte 65		8.64	6.19	1.41	122.544°W	45.476°N	67.48	1.00
Geologic Model Full Rupture	Fault							5.18
Portland Hills		1.22	7.00	-0.58	122.620°W	45.436°N	120.49	5.11
WUSmap_2014_fixSm.ch.in (opt)	Grid							3.99
PointSourceFinite: -122.646, 45.505		7.88	5.90	0.97	122.646°W	45.505°N	0.00	1.82
noPuget_2014_fixSm.ch.in (opt)	Grid							3.99
PointSourceFinite: -122.646, 45.505		7.88	5.90	0.97	122.646°W	45.505°N	0.00	1.82
WUSmap_2014_fixSm.gr.in (opt)	Grid							3.67
PointSourceFinite: -122.646, 45.505		8.00	5.84	1.02	122.646°W	45.505°N	0.00	1.71
noPuget_2014_fixSm.gr.in (opt)	Grid							3.67
PointSourceFinite: -122.646, 45.505		8.00	5.84	1.02	122.646°W	45.505°N	0.00	1.71
coastalOR_deep.in	Slab							1.86
Zeng Model Partial Rupture	Fault							1.81
Portland Hills		2.87	6.75	-0.27	122.620°W	45.436°N	120.49	1.79
sub0_ch_top.in	Interface							1.76
Cascadia Megathrust - whole CSZ Characteristic		149.29	8.84	1.87	124.549°W	45.485°N	272.32	1.76
Zeng Model Small Mag	Fault							1.26
WUSmap_2014_fixSm_M8.in (opt)	Grid							1.22
noPuget_2014_fixSm_M8.in (opt)	Grid							1.22
sub2_ch_bot.in	Interface							1.15
Cascadia Megathrust - Goldfinger Case C Characteristic		100.78	8.74	1.31	123.702°W	45.000°N	239.40	1.15

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
sub1_ch_bot.in	Interface							1.01
Cascadia Megathrust - Goldfinger Case B Characteristic		81.63	8.86	0.97	123.599°W	45.501°N	275.05	1.01