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Geotechnical Engineering Report

Project Information:	**Banks Subdivision** **GeoPacific Project № 22-6070** **July 11, 2022**
Site Location:	**Clackamas County Tax Lot 2N43600 600** **South of NW Cedar Canyon Road and** **North of Banks Sunset Park** **Banks, Oregon**
Client:	**David Weekly Homes** **Max Bondar** **1905 NW 169th Place, #102** **Beaverton, OR 97006** **Phone: (503) 516-5078** **mbondar@dwhomes.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, assess potential geologic hazards at the property, and to provide geotechnical recommendations for site development. This geotechnical study was performed in accordance with GeoPacific Proposal No. P-8117, dated May 27, 2022, and your subsequent authorization of our proposal and *General Conditions for Geotechnical Services*.

2.0 SITE AND PROJECT DESCRIPTION

The subject site is located to the south of NW Cedar Canyon Road and to the north of Banks Sunset Park in the City of Banks, Oregon. The site is approximately 20.3 acres in size and consists of Tax Lot 600 of Tax Map 2N43600. Topography on the site is gently sloping down to the west, with site elevations ranging from approximately 195 to 215 feet amsl. The majority of the site consists of an agricultural field. However, there are several single-family residences and outbuildings in the northern portion of the site, near NW Cedar Canyon Road. Vegetation around the single-family residences and outbuildings includes some small to large trees, while vegetation in the rest of the site generally consists of agricultural plantings.

It is our understanding that the proposed development will consist of the construction of single-family homes, new streets, a stormwater facility, and associated underground utilities. A grading plan has not been provided for our review; however, we anticipate maximum cuts and fills will be on the order of about 5 feet or less. A commercial/industrial parcel may be developed at a future date in the southwest portion of the site, but is not considered to be part of the current plans for development.

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 Quaternary age (last 1.6 million years) Willamette Formation, a catastrophic flood deposit 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 horizontally layered, micaceous, silt to coarse sand forming poorly-defined to distinct beds less than 3 feet thick. Regional studies indicate that the thickness of the Willamette Formation in the vicinity of the subject site is approximately 50 feet (Madin, 1990). The Willamette Formation is underlain by Miocene aged (about 14.5 to 16.5 million years ago) Columbia River Basalt, a thick sequence of lava flows which forms the crystalline basement of the basin.

Adjacent to the subject site, geologic maps indicate the presence of Holocene age (last 10 thousand years) alluvial deposits consisting of unconsolidated sand, gravel, and silt deposited along channels and flood plains of the modern drainage system (Madin, 1998).

4.0 REGIONAL SEISMIC SETTING

At least one major fault zone capable of generating damaging earthquakes, the Cascadia Subduction Zone, is thought to exist in the vicinity of the subject site.

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 June 9 and 14, 2022. A total of twenty exploratory test pits (TP-1 through TP-20) were excavated at the site using a backhoe to maximum depths of 11 feet below existing ground surface (bgs). Explorations were conducted under the full-time observation of a GeoPacific engineering staff member. 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 Horizon: At the ground surface in test pits TP-1, TP-3, and TP-4 was organic SILT (ML-OL) which was brown and contained fine roots. The organic topsoil generally extended to depths between 5 and 7 and inches bgs. Topsoil depths are likely to increase where trees are present. In test pits TP-5 through TP-20, we observed a 6-inch thick root mat layer, as described in the *Tilled Zone* section below.

Undocumented Fill: At the ground surface in test pits TP-2 and below the topsoil in test pit TP-4, we encountered undocumented fill material. In test pit TP-2, the undocumented fill consisted of gray and brown Silty GRAVEL (GM) and extended to a depth of approximately 1 foot bgs. In test pit TP-4, the undocumented fill was comprised of dark brown SILT (ML) which was stiff to very stiff. The undocumented fill encountered in test pit TP-4 extended to a depth of approximately 4 feet bgs.

Tilled Zone: At the ground surface in test pits TP-5 through TP-20, we observed a tilled zone of material consisting of disturbed SILT (ML). The tilled zone typically extended to approximately 15 inches bgs, with some areas extending to up to 18 inches bgs. In our explorations, the tilled zone had a root mat at the ground surface. The root mat was generally 6 inches thick. Below the root mat, the soil in the tilled zone was generally light brown to brown and generally exhibited orange and gray mottling.

Catastrophic Flood Deposits: Underlying the topsoil, undocumented fill, or tilled zone, soils typically consisted of SILT (ML) that was medium stiff to very stiff and light brown to brown and generally exhibited orange and gray mottling. These soil types were considered to be Catastrophic Flood Deposits and extended beyond the 11-foot maximum depth of exploration.

5.2 Shrink-Swell Potential

Medium stiff to very stiff, low plasticity fine-grained soils were encountered within the upper 11 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 of foundations as a result of potential soil expansion at this site.

5.3 Groundwater and Soil Moisture

On June 9 and 14, 2022, observed soil moisture conditions ranged from damp to very moist. Layers of perched groundwater seepage were encountered at various depths, ranging from 5 to 10 feet bgs. According to local well logs, groundwater has been documented within the site vicinity at depths ranging from 10 to 40 feet bgs. 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.

6.0 PORTABLE DYNAMIC CONE PENETROMETER TESTING

On June 14, 2022, soil strength tests were conducted at four locations within the alignment of the future interior roadways using a portable dynamic cone penetrometer (PDCP), provided by GeoPacific. A summary of subgrade soil test data is presented in Table 1. PDCP data is attached to the appendix of this report.

Table 1 - PDCP Field Test Results for Interior Roadways

Field Test Designation	Depth Interval of Test (ft)	Average Penetration Per Blow (mm)	M_R - Subgrade Resilient Modulus (psi)
PDCP-1	1.0 - 3.4	40.0	5232
PDCP-2	0.8 – 3.4	44.2	5032
PDCP-3	0.8 – 3.4	45.3	4985
PDCP-4	0.9 – 3.4	52.5	4705

7.0 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 two geotechnical concerns associated with the proposed site development are:

1) Site preparation, due to the presence of a tilled zone layer and undocumented fill material

2) Site grading and utility trench excavation, due to the presence of perched groundwater seepage at relatively shallow depths.

In test pits TP-5 through TP-20, we encountered a tilled zone layer. The upper 6 inches of the tilled zone layer generally consisted of a root mat, which will need to be stripped off and will not be suitable for reuse as engineered fill. Below the root mat, to a depth of approximately 18 inches below the existing ground surface, the tilled zone consisted of disturbed soil and is not considered suitable for the support of roadways, fill material, or structures in its existing condition.

In areas of roadways, structures, or where engineered fill material is proposed, the portion of the tilled zone layer below the root mat will need to be ripped/tilled, moisture-conditioned, and recompacted. Alternatively, it could be removed and replaced with engineered fill material. For construction in the wet weather season, it may be advantageous to amend the layer of disturbed native soil by tilling in cement powder.

We encountered undocumented fill material in test pits TP-2 and TP-4. The undocumented fill material extended to a depth of 1 foot in test pit TP-2 and 4 feet in test pit TP-4. Where it is thicker than approximately 1 foot, the undocumented fill material will need to be removed and replaced with engineered fill. Where the undocumented fill material is 1 foot thick or less, it may be feasible to rip/till, moisture-condition, and recompact it.

We encountered perched groundwater seepage in several of our explorations, at depths ranging from 5 to 10 feet. Site grading may be impacted by perched groundwater seepage. Also, excavations for utility trenches may be complicated by perched groundwater seepage.

Subsurface drainage tile systems are commonly encountered in agricultural areas and are usually removed during the site preparation stage of construction, if encountered. Drain tiles missed during site preparation could potentially introduce water into utility trenches or daylight beneath structures.

7.1 Site Preparation

Areas of proposed construction and areas to receive fill should be cleared of any organic and inorganic debris, 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. Following removal of topsoil, the existing ground surface should be aerated, scarified and recompacted in areas proposed for placement of engineered fill and structures.

In areas of roadways, structures, or where engineered fill material is proposed, the portion of the tilled zone layer below the root mat will need to be ripped/tilled, moisture-conditioned, and recompacted. Alternatively, it could be removed and replaced with engineered fill material. For construction in the wet weather season, it may be advantageous to amend the layer of disturbed native soil by tilling in cement powder.

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.

Where/if encountered, 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. We encountered undocumented fill material in test pits TP-2 and TP-4. The undocumented fill material extended to a depth of 1 foot in test pit TP-2 and 4 feet in test pit TP-4. Where it is thicker than approximately 1 foot, the undocumented fill material will need to be removed and replaced with engineered fill. Where the undocumented fill material is 1 foot thick or less, it may be feasible to rip/till, moisture-condition, and recompact it.

Subsurface drainage tile systems are commonly encountered in agricultural areas and are usually removed during the site preparation stage of construction, if encountered. Drain tiles missed during site preparation could potentially introduce water into utility trenches or daylight beneath structures.

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

7.2 Engineered Fill

All grading for the proposed construction should be performed as engineered grading in accordance with the applicable building code at the time of construction with the exceptions and additions noted herein. Site grading should be conducted in accordance with the requirements outlined in the 2018 International Building Code (IBC), and 2019 Oregon Structural Specialty Code (OSSC), Chapter 18 and Appendix J. Areas proposed for fill placement should be prepared as described in *Site Preparation Recommendations*. Surface soils should be aerated, scarified and recompacted prior to placement of structural fill. Site preparation, soil stripping, and grading activities should be observed and documented by a geotechnical engineer or his representative. Proper test frequency and earthwork documentation usually requires daily observation and testing during stripping, rough grading, and placement of engineered fill.

Onsite native soils appear to be suitable for use as engineered fill. Soils containing greater than 5 percent organic content should not be used as structural fill. Imported fill material must be approved by the geotechnical engineer prior to being imported to the site. Oversize material greater than 6 inches in size should not be used within 3 feet of foundation footings, and material greater than 12 inches in diameter should not be used in engineered fill.

Engineered fill should be compacted in horizontal lifts not exceeding 12 inches using standard compaction equipment. We recommend that engineered fill be compacted to at least 95 percent of the maximum dry density determined by ASTM D698 (Standard Proctor) or equivalent. Soils should be moisture conditioned to within two percent of optimum moisture. Field density testing should conform to ASTM D2922 and D3017, or D1556. All engineered fill should be observed and tested by the project geotechnical engineer or his representative. Typically, one density test is performed for at least every 2 vertical feet of fill placed or every 500 yd^3, whichever requires more testing. Because testing is performed on an on-call basis, we recommend that the earthwork contractor be held contractually responsible for test scheduling and frequency.

Site earthwork may be impacted by shallow groundwater, soil moisture and wet weather conditions. Earthwork in wet weather would likely require extensive use of additional crushed aggregate, cement or lime treatment, or other special measures, at considerable additional cost compared to earthwork performed under dry-weather conditions.

7.3 Excavating Conditions and Utility Trench Backfill

We anticipate that onsite soils can generally be excavated using conventional heavy equipment. Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor. Actual slope inclinations at the time of construction should be determined based on safety requirements and actual soil and groundwater conditions. All temporary cuts in excess of 4 feet in height should be sloped in accordance with U.S. Occupational Safety and Health Administration (OSHA) regulations (29 CFR Part 1926) or be shored. The existing native 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.

7.4 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.

7.5 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.

7.6 Spread Foundations

We anticipate that the homes will be one to two stories, constructed with typical spread foundations and wood framing, with maximum structural loading on column footings and continuous strip footings 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 6 feet or less.

The proposed structures may be supported on shallow foundations bearing on medium stiff to stiff, native soils and/or engineered fill, appropriately designed and constructed as recommended in this report. Foundation design, construction, and setback requirements should conform to the applicable building code at the time of construction. 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 1,500 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. For loads heavier than 35 kips, the geotechnical engineer should be consulted. If heavier loads than described above are proposed, it may be necessary to over-excavate point load areas and replace with additional compacted crushed aggregate to achieve a higher allowable bearing capacity. The coefficient of friction between on-site soil and poured-in-place concrete may be taken as 0.42, which includes no factor of safety. The maximum anticipated total and differential footing movements (generally from soil expansion and/or settlement) are 1 inch and ¾ inch over a span of 20 feet, respectively. We anticipate that the majority of the estimated settlement will occur during construction, as loads are applied. Excavations near structural footings should not extend within a 1H:1V plane projected downward from the bottom edge of footings.

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

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

7.7 Concrete Slabs-on-Grade

Preparation of areas beneath concrete slab-on-grade floors should be performed as described in the Site *Preparation Recommendations* 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.

7.8 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.

7.9 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.

7.10 Pavement Design

It is our understanding that new public streets are planned as part of the proposed development. GeoPacific may be consulted to provide pavement design calculations and construction recommendations when traffic loading data is available.

8.0 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 2018 International Building Code (IBC) with applicable Oregon Structural Specialty Code (OSSC) revisions (current 2019). We anticipate the proposed residential structures will have a fundamental period of vibration equal to or less than 0.5 seconds. If this is the case, the structures will be exempt from the requirement to perform a site response analysis, per section 20.3.1 of ASCE 7 and the site class may be determined according to Table 20.3-1. Provided that this is the case, the site class is determined to be Site Class D,

Design values determined for the site using the ATC (Applied Technology Council) *ASCE7-16 Hazards by Location online Tool* website are summarized in Table 4. Site class determination is based upon extrapolated CPT data, and soil conditions observed during field explorations.
ummary Report are summarized in Table 2 and are based upon observed existing soil conditions.

Table 2 - Recommended Earthquake Ground Motion Parameters (ASCE-7-16)

Parameter	Value
Location (Lat, Long), degrees	45.6178088, --122.1144457
Probabilistic Ground Motion Values, 2% Probability of Exceedance in 50 yrs	
Site Modified Peak Ground Acceleration PGA_M	0.502 g
Short Period, S_s	0.921 g
1.0 Sec Period, S_1	0.455 g
Soil Factors for Site Class D:	
F_a	1.132
* F_v	1.845
SD_s = 2/3 x F_a x S_s	0.695 g
*SD_1 = 2/3 x F_v x S_1	0.560 g
Seismic Design Category	D

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

8.1 Soil Liquefaction

The Oregon Department of Geology and Mineral Industries (DOGAMI), Oregon HazVu: 2022 Statewide GeoHazards Viewer indicates that the site is mapped as being at *high* 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.

The subsurface profile observed within our explorations and our experience with geologic conditions in the site vicinity indicate that the site is underlain by medium stiff to very stiff silt. Static groundwater was not encountered in our explorations, excavated to depths of up to 11 feet. Static groundwater is expected to be present at depths of 10 to 40 feet bgs in the vicinity of the site.

Since single-family residences are typically lightly loaded and relatively flexible, it is standard engineering practice that special design or construction measures are not required for single-family residences in order to protect life safety due to liquefaction. However, it should be noted that in the event of a large earthquake, some damage might occur to the proposed structures due to differential settlement and/or lateral spreading resulting from soil liquefaction.

It is our understanding that for construction of single-family structures, special design or construction measures are not required by code to mitigate the effects of liquefaction. However, GeoPacific may be consulted to perform further study of seismic hazards on the site if desired. If commercial/industrial, multi-family residential, high occupancy, or critical structures were to be incorporated into plans for site development, further study and evaluation of seismic hazards would be required by code to evaluate the potential adverse effects due to liquefaction, such as vertical settlement, lateral deformation, and lateral spreading. We anticipate that our additional explorations on the site for the purpose of evaluating seismic hazards would include at least two cone penetrometer tests. It is our understanding that a commercial/industrial parcel may be developed at a future date in the southwest portion of the site, but is not considered to be part of the current plans for development.

9.0 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
90013PE
OREGON
MAY 22, 2015
BENJAMIN G. ANDERSON

Expires: 12/31/23

Alexandria B. Campbell, E.I.
Engineering Staff

Benjamin G. Anderson, P.E.
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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	