HARDMAN GEOTECHNICAL SERVICES INC.
Practical, Cost-Effective Geotechnical Solutions

June 7, 2018
HGSI Project No. 18-2287

Chuck Overton
Windermere Real Estate
588 Pacific Way
P.O. Box 2729
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Copy: Li Alligood, Otak

Via e-mail with hard copies mailed on request

Subject: **PRELIMINARY GEOTECHNICAL ENGINEERING REPORT**
PROPOSED RESIDENTIAL DEVELOPMENT
PALMBERG PROPERTY
GEARHART, OREGON

This report presents the results of a preliminary geotechnical engineering study conducted by Hardman Geotechnical Services Inc. (HGSI) for the above referenced project. The purpose of this study is to evaluate subsurface conditions at the site and to provide geotechnical recommendations for site development. This geotechnical study was performed in accordance with HGSI proposal 18-813, dated February 6, 2018, and your subsequent authorization of our agreement and *General Conditions for Geotechnical Services.* Please note that HGSI's scope of work consisted of evaluating physical geotechnical characteristics of the soil only; evaluation of the potential for contaminated soils or groundwater on site is beyond the scope of this study.

SITE DESCRIPTION AND PROPOSED DEVELOPMENT

Our understanding of the site and project conditions is based on a review of information provided, including a memorandum from Pacific Habitat Services, Inc. addressing wetland issues for the site, and a Conceptual Development Plan prepared by Otak. The total site area is reportedly about 29 acres, although site development will be limited to the northern portion of the site to avoid wetland impacts. The site area contains a 4.5-acre human-made lake, located generally south and west of the planned development area.

The site has been in the Palmberg family for decades and has been used as part of Palmberg Paving Co. operations. Based on a brief review of Google Earth® historical aerial photos going back to July, 2000, the area of the planned development north of the lake was used as a stockpile, production and hauling area as well as staging for equipment and materials, etc. There is a potential for undocumented fill soils to be present in the conceptual development area due to the site's past use. We understand the lake was formed many years ago by gravel removal operations.

The preliminary plan prepared by Otak indicates the conceptual plan to include 25 lots, accessed by a private driveway extending west off of McCormick Gardens Road. The private drive will include several turn-outs and a cul-de-sac at its western terminus. Please note that the final development plan may vary from that described herein.

The proposed development includes grading the site to support the planned single family residential construction, with associated underground utilities and private driveway. Details of the planned structure and street layout, and proposed grading, have not yet been developed.

SITE GEOLOGY AND SEISMICITY

The subject site is underlain by Quaternary age (last 1.6 million years) dune sand (Walsh, 1987; Niem et al., 1983). This geologic unit is described as "active and inactive dune sands forming several prominent north-south beach ridges on Clatsop Spit. Sands are well sorted, fine grained, quartzo-feldspathic with heavy mineral laminae, and cross-bedded." At this site, the dune sands are considered inactive.

Seismicity of the site area is dominated by the Cascadia Subduction Zone, which essentially underlies this portion of coastal Oregon. Earthquake risk from the Cascadia Subduction Zone and other potential seismic source zones are included in the probabilistic earthquake design parameters specified in the current building code (see *Seismic Design* section, below). The site is partially within the mapped "Statutory Tsunami Inundation Zone," as discussed in *Seismic Design*.

FIELD EXPLORATION

The site-specific exploration for this study was conducted on March 23, 2018 and consisted of exploratory test pits conducted using a medium-sized excavator provided by the client. Eight test pits, designated TP-1 through TP-8, were excavated to depths of approximately 3 to 9 feet below ground surface (bgs), at approximate locations shown on Figure 2. It should be noted that exploration locations were determined 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.

Explorations were conducted under the full-time observation of HGSI personnel. Soil samples were classified in the field and representative portions were placed in relatively air-tight plastic bags. These soil samples were then returned to the laboratory for further examination and laboratory testing. Pertinent information including soil sample depths, stratigraphy, soil engineering characteristics, and groundwater occurrence was recorded. Soils were classified in general accordance with the Unified Soil Classification System.

Summary exploration logs are attached. The stratigraphic contacts shown on the individual 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.

SUBSURFACE CONDITIONS

The following discussion is a summary of subsurface conditions encountered in the test pit explorations. For more detailed information regarding subsurface conditions at specific exploration locations, refer to the attached exploration logs. Also, please note that subsurface conditions can vary between exploration locations, as discussed in the *Uncertainty and Limitations* section below.

Soil

On-site soils encountered in the test pits consisted of topsoil, undocumented fill, and native sandy silt to silty sand, as described below.

> ***Topsoil:*** In all of the borings except TP-3, the ground surface was directly underlain by topsoil consisting of brown, low to moderately organic silt with fine roots throughout. Where overlying old fill, topsoil thickness ranged from about 3 to 6 inches. Where overlying native ground, topsoil thickness in the explorations was about 12 to 18 inches. A disturbed/organic zone was encountered in TP-1 to a depth of about 3 feet bgs, in the northwest portion of the site.
>
> ***Undocumented Fill –*** Undocumented fill was encountered in test pits TP-3, TP-4 and TP-7, extending to depths of greater than 3 feet, 1 foot, and greater than 7.5 feet bgs respectively. TP-3 could not be excavated deeper than 3 feet (refusal) with the medium-sized excavator used for this investigation. The fill was not homogeneous, and ranged from Sand with Silt, to Gravelly Silt and Sandy Gravel. It should be noted that areas of fill may be present in areas beyond the test pit locations. The fill was generally soft, except in TP-3 where fill hardness prevented excavation below 3 feet. This fill was likely compacted over the years as TP-3 is located within the area of the former asphalt plant. Test pits TP-3 and TP-7 were terminated in the undocumented fill unit at depths of 3 and 7.5 feet bgs respectively.
>
> ***Native Sandy Silt to Silty Sand:*** Underlying the topsoil layer in all borings we encountered material belonging to the [inactive] dune sand formation. The native dune sand materials encountered in our test pits generally medium stiff / medium dense, interbedded layers of silt with sand, sandy silt, and gravelly silt. All of the test pits except TP-3 and TP-7 were terminated in the dune sand unit.

Groundwater

Groundwater seepage was encountered in all of the test pits except TP-3, TP-6 and TP-7. The depth to groundwater/seepage ranged from about 3 to 6 feet bgs. Caving test pit side walls occurred in most of the test pits, within sandy soil layers near and below the water level. It is anticipated that groundwater conditions will vary depending on the season, local subsurface conditions, changes in site utilization, and other factors.

CONCLUSIONS AND RECOMMENDATIONS

Results of this study indicate that the proposed development is geotechnically feasible, provided the recommendations of this report are followed. The primary constraints to site development are the presence of localized and unpredictable undocumented fills, and relatively shallow groundwater with the potential for caving sands in trench excavations.

The proposed residential structures may be supported on shallow foundations bearing on competent undisturbed native soils and/or engineered fill, designed and constructed as recommended in this report. The recommendations of this report assume the single-family structure will have raised floors and crawlspaces. We suggest that HGSI be consulted during preparation of the grading plan for the project, to ensure that geotechnical issues are addressed and to assist in optimizing the grading plan to minimize the amount of undocumented fill removal needed.

Site Preparation and Undocumented Fill Removal

A significant amount of undocumented fill exists on site. Generally, the near-surface undocumented fill soils are not suitable to support the planned structures and pavements without remedial grading measures. It is possible that portions of the undocumented fill may remain in place below the areas to be developed; however, improvements constructed upon undocumented fill would need to be carefully evaluated and verified in the field during construction.

Additional geotechnical input will be needed during final design, to develop more specific removal and replacement recommendations based on the specifics of site grading. HGSI should observe removal excavations prior to fill placement to verify that removals are adequate and an appropriate bearing stratum is exposed.

Due to the extent of the undocumented fill on site, we recommend that remedial overexcavation / replacement measures be done over the entire site as a mass grading operation, and not on a lot-by-lot basis during house construction. Much of the undocumented fill soils may be reused as engineered fill, provided oversize material (boulders) and highly organic soils are removed.

The areas of the site to be graded should first be cleared of vegetation and any loose debris; and debris from clearing should be removed from the site. Organic-rich topsoil should then be removed to competent native soils, in areas where undocumented fill removal is not performed. We anticipate that the average depth of topsoil stripping will be at least 10 inches over most of the site. The final depth of stripping removal may vary depending on local subsurface conditions and the contractor’s methods, and should be determined on the basis of site observations after the initial stripping has been performed. Stripped organic soil should be stockpiled only in designated areas or removed from the site and stripping operations should be observed and documented by HGSI. Existing subsurface structures (tile drains, old utility lines, septic leach fields, etc.) beneath areas of proposed structures and pavement should be removed and the excavations backfilled with engineered fill.

In construction areas, once stripping and/or undocumented fill removal has been verified, the area should be ripped or tilled to a depth of 12 inches, moisture conditioned, and compacted in-place prior to the placement of engineered fill. Exposed subgrade soils should be evaluated by HGSI.

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

Engineered Fill

On-site native soils are anticipated to be suitable for use as engineered fill during dry weather, provided they are adequately moisture conditioned prior to compacting. Imported fill material should be reviewed 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. Placement of boulders greater than 12 inches in size may be feasible in deeper fill areas, provided the boulders are surrounded in properly compacted engineered fill and boulders are not nested or stacked. Specific recommendations should be provided by HGSI in the field based on the quantity and size of rock materials being generated in the cuts.

Engineered fill should be compacted in horizontal lifts not exceeding 12 inches using heavy vibratory compaction equipment. We recommend that engineered fill be compacted to at least 90% of the maximum dry density determined by Modified Proctor (ASTM D1557) or equivalent. We anticipate that aeration of native soil will be necessary for compaction operations.

Proper test frequency and earthwork documentation usually requires daily observation and testing during stripping, rough grading, and placement of engineered fill. Field density testing should conform to ASTM D2922 and D3017, or D1556. Engineered fill should be periodically observed and tested by HGSI. Typically, one density test is performed for at least every 2 vertical feet of fill placed or every 100 yd^3, whichever requires more testing.

Wet Weather Earthwork

Soils underlying the site are 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 probably require expensive measures such as cement treatment or imported granular material to compact fill 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 fines. 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
- Bales of straw and/or geotextile silt fences should be strategically located to control erosion.

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

Structural Foundations

Based on our understanding of the proposed project and the results of our exploration program, and assuming our recommendations for site preparation are followed, native deposits and/or engineered fill soils will be encountered at or near the foundation level of the proposed structures. These soils are generally stiff to very stiff and should provide adequate support of the structural loads.

Shallow, conventional isolated or continuous spread footings may be used to support the proposed structures, provided they are founded on competent native soils, or compacted engineered fill placed directly upon the competent native soils. We recommend a maximum allowable bearing pressure of 2,000 pounds per square foot (psf) for designing the footings. The recommended maximum allowable bearing pressure may be increased by 1/3 for short term transient conditions such as wind and seismic loading. Minimum footing depths and widths should be determined by the project engineer/architect in accordance with applicable design codes.

Assuming construction is accomplished as recommended herein, and for the foundation loads anticipated, we estimate total settlement of spread foundations of less than about 1 inch and differential settlement between two adjacent load-bearing components supported on competent soil of less than about ½ inch. We anticipate that the majority of the estimated settlement will occur during construction, as loads are applied.

Wind, earthquakes, and unbalanced earth loads will subject the proposed structure to lateral forces. Lateral forces on a structure will be resisted by a combination of sliding resistance of its base or footing on the underlying soil and passive earth pressure against the buried portions of the structure. For use in design, a coefficient of friction of 0.5 may be assumed along the interface between the base of the footing and subgrade soils. Passive earth pressure for buried portions of structures may be calculated using an equivalent fluid weight of 390 pounds per cubic foot (pcf), assuming footings are cast against dense, natural soils or engineered fill. The recommended coefficient of friction and passive earth pressure values do not include a safety factor. The upper 12 inches of soil should be neglected in passive pressure computations unless it is protected by pavement or slabs on grade.

Footing excavations should be trimmed neat and the bottom of the excavation should be carefully prepared. All loose or softened soil should be removed from the footing excavation prior to placing reinforcing steel bars. We recommend that footing excavations be observed by HGSI prior to placing steel and concrete, to verify that the recommendations of this report have been followed, and that an appropriate bearing stratum has been exposed.

Concrete Retaining 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 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 walls, an at-reset equivalent fluid pressure of 54 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 6H, where H is the total height of the wall.

We assume relatively level ground surface below the base of the walls. As such, we recommend passive earth pressure of 390 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 HGSI should be contacted for additional recommendations.

A coefficient of friction of 0.5 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.

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-inch wide zone of crushed drain rock containing less than 5 percent fines 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 sump to remove water from the crushed drain rock zone. The drain pipe should be wrapped in filter fabric (Mirafi 140N or other as approved by the geotechnical engineer) to minimize clogging. The above drainage measures are intended to remove water from behind the wall to prevent hydrostatic pressures from building up. Additional drainage measures may be specified by the project architect or structural engineer, for damp-proofing or other reasons.

HGSI 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.

Footing and Roof Drains

To minimize the fluctuation of soil moisture content near structural foundations, we recommend that the structures be constructed with perimeter footing drains. The outside edge of all perimeter footings should be provided with a drainage system consisting of 3-inch minimum diameter perforated plastic pipe embedded in a minimum of 1 ft^3 per lineal foot of clean, free-draining sand and gravel or 2”-1/2” drain rock. The drain pipe and surrounding drain rock should be wrapped in non-woven geotextile (Mirafi 140N, or approved equivalent) to minimize the potential for clogging and/or ground loss due to piping. Water collected from the footing drains should be directed into the local storm drain system or other suitable outlet. A minimum 0.5 percent fall should be maintained throughout the drain and non-perforated pipe outlet. The footing drains should include clean-outs to allow periodic maintenance and inspection.

Construction should include typical measures for controlling subsurface water beneath the home, 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 homebuyers should be informed and educated that some slow flowing water in the crawlspaces is considered normal and not necessarily detrimental to the home given these other design elements incorporated into its construction. Appropriate design professionals should be consulted regarding crawlspace ventilation, building material selection and mold prevention issues, which are outside HGSI’s area of expertise.

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

Seismic Design

Structures should be designed to resist earthquake loading in accordance with the methodology described in the 2012/2015 International Building Code (IBC) with applicable 2014 Oregon

Structural Specialty Code (OSSC) revisions. We recommend Site Class D be used for design per the OSSC, which references ASCE 7-10, Chapter 20, Table 20.3-1. Design values determined for the site using the USGS (United States Geological Survey) *Earthquake Ground Motion Parameters* utility are summarized on Table 1.

Table 1. Recommended Earthquake Ground Motion Parameters (2012 IBC / 2014 OSSC)

Parameter	Value
Location (Lat, Long), degrees	46.0272, -123.9072
Mapped Spectral Acceleration Values (MCE, Site Class B):	
Short Period, S_s	1.326 g
1.0 Sec Period, S_1	0.680 g
Soil Factors for Site Class D:	
F_a	1.000
F_v	1.500
$SD_s = 2/3 \text{ x } F_a \text{ x } S_s$	0.884 g
$SD_1 = 2/3 \text{ x } F_v \text{ x } S_1$	0.680 g

Portions of the site are within the “Statutory Tsunami Inundation Zone” as mapped by the Oregon Department of Geology and Mineral Industries (DOGAMI, 2018), on their HAZVU website. Figure 3 shows the extent of the mapped inundation zone. In HGSI’s opinion, the site does not have any greater risk for tsunami inundation hazard than other portions of Gearhart at similar elevations. As can be seen on Figure 3, the majority of Gearhart is below the inundation hazard elevation. In our opinion, additional design measures are not warranted for this site due to the potential inundation hazard. However, education of homebuyers, and signage with clearly marked routes for tsunami evacuation, should be considered as part of project design.

Excavating Conditions and Utility Trenches

We anticipate that on-site soils can be excavated to depths of our excavator test pits (9 feet) using conventional heavy equipment such as trackhoes. It should be noted that trench excavations in the sand unit will be subject to caving, particularly for deeper excavations and during wet weather. Perched groundwater conditions were encountered in most of the explorations, at depths ranging from about 3 to 6 feet bgs. Where encountered, the contractor should be prepared to implement an appropriate dewatering system for installation of the utilities. At this time, we anticipate that dewatering systems consisting of ditches, sumps and pumps would be adequate for control of groundwater where encountered during construction conducted during the dry season. Regardless of the dewatering system used, it should be installed and operated such that in-place soils are prevented from being removed along with the groundwater.

Test pit TP-3 met refusal at shallow depths on very dense/hard fill, with the medium-sized excavator used in our exploration. Larger excavation equipment can likely excavate to greater depths in the hard fill materials. There is some potential for concrete and other debris to be buried on the site. The contractor should be informed of the potential for encountering such obstructions and a budget contingency allocated for excavation and removal of oversize materials.

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

Vibrations created by traffic and construction equipment may cause some caving and raveling of excavation walls. In such an event, lateral support for the excavation walls should be provided by the contractor to prevent loss of ground support and possible distress to existing or previously constructed structural improvements.

PVC pipe should be installed in accordance with the procedures specified in ASTM D2321. We recommend that structural trench backfill be compacted to at least 90 percent of the maximum dry density obtained by Modified Proctor (ASTM D1557) or equivalent. Initial backfill lift thicknesses for a 5/8”-0 crushed aggregate base may need to be as great as 4 feet to reduce the risk of flattening underlying flexible pipe. Subsequent lift thickness should not exceed 1 foot. If imported granular fill material is used, then the lifts for large vibrating plate-compaction equipment (e.g. hoe compactor attachments) may be up to 2 feet, provided that proper compaction is being achieved and each lift is tested. Use of large vibrating compaction equipment should be carefully monitored near existing structures and improvements due to the potential for vibration-induced damage.

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

Erosion Control Considerations

During our field exploration program, we did not observe soil types near the ground surface that would be considered highly susceptible to erosion. In our opinion, the primary concern regarding erosion potential will occur during construction, in areas that have been stripped of vegetation. Erosion at the site during construction can be minimized by implementing the project erosion control plan, which should include judicious use of “bio-bags,” silt fences and/or other appropriate technology. Where used, these erosion control devices should be in place and 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.

UNCERTAINTY 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, HGSI should be notified for review of the recommendations of this report, and revision of such if necessary.

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

Within the limitations of scope, schedule and budget, HGSI executed these services in accordance with generally accepted professional principles and practices in the field of geotechnical engineering 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,

HARDMAN GEOTECHNICAL SERVICES INC.

64,422PE
SEPTEMBER 26, 2000
SCOTT L. HARDMAN
6-7-18
EXPIRES: 06-30-2019

Scott L. Hardman, P.E., G.E.
Principal Geotechnical Engineer

Attachments: References
Figure 1 – Vicinity Map
Figure 2 – Site Plan and Exploration Locations
Figure 3 – Mapped Tsunami Inundation Zone
Logs of Test Pits TP-1 through TP-8

REFERENCES

Oregon Department of Geology and Mineral Industries (DOGAMI) HAZVU website, 2018: https://gis.dogami.oregon.gov/maps/hazvu/

Walsh, T.J., 1987, Geologic map of the Astoria and Ilwaco quadrangles, Washington and Oregon: Washington Division of Geology and Earth Resources, Open File Report 87-2, scale 1:100,000

Wells, R.E., Niem, A.R., MacLeod, N.S., Snavely, P.D., and Niem, W.A., 1983, Preliminary geologic map of the west half of the Vancouver (Wa.-Ore.) 1 degree X 2 degree quadrangle, Oregon: U.S. Geological Survey, Open-File Report OF-83-591, scale 1:250,000

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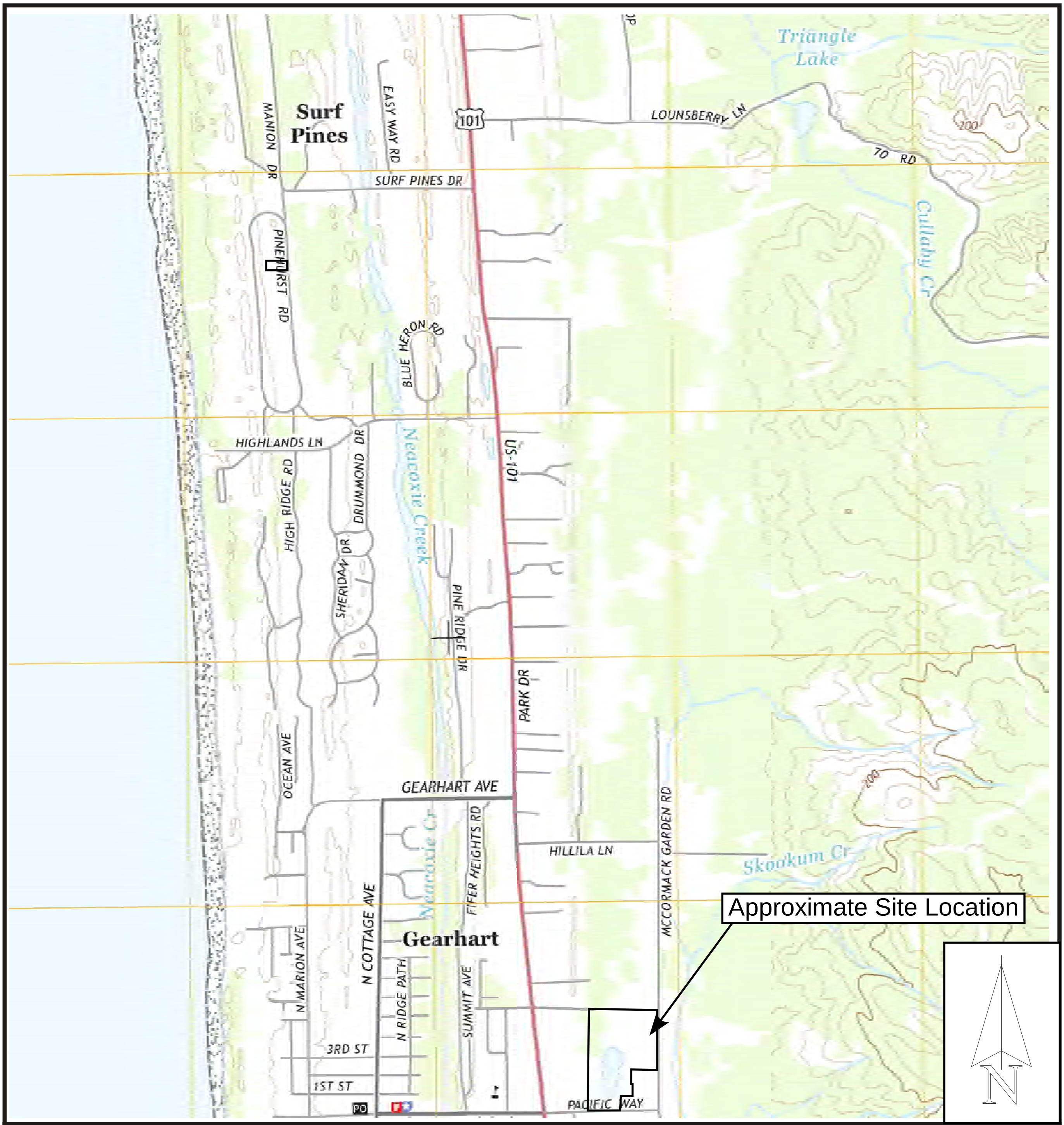
VICINITY MAP



Legend

Approx. Scale: 1 inch = 2000 feet

Base Map: USGS Gearhart, Oregon Quad, from US Topo, 2014

Project: Palmberg Property Gearhart, Oregon	Project No. 18-2287	FIGURE 1

HARDMAN GEOTECHNICAL SERVICES INC.
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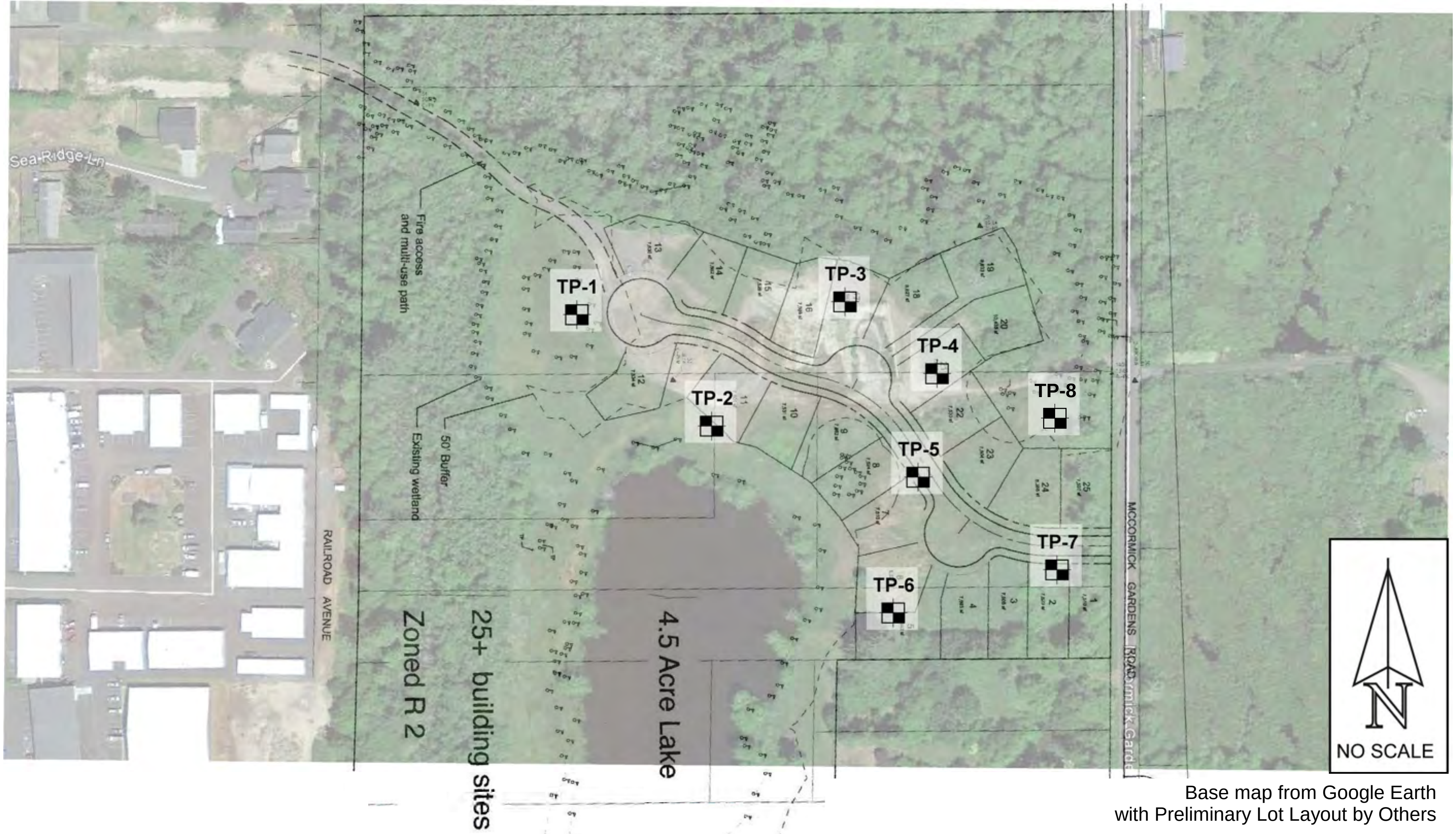
SITE AND EXPLORATION PLAN



Base map from Google Earth
with Preliminary Lot Layout by Others

Legend

TP-8
▚ Test Pit Designation and Approximate Location

Note: Base map and overlay may not be aligned accurately due to scaling, orientation and distortion issues. Use for schematic purposes only.

Project: Palmberg Property Gearhart, Oregon	Project No. 18-2287	FIGURE 2

HGSI HARDMAN GEOTECHNICAL SERVICES INC.
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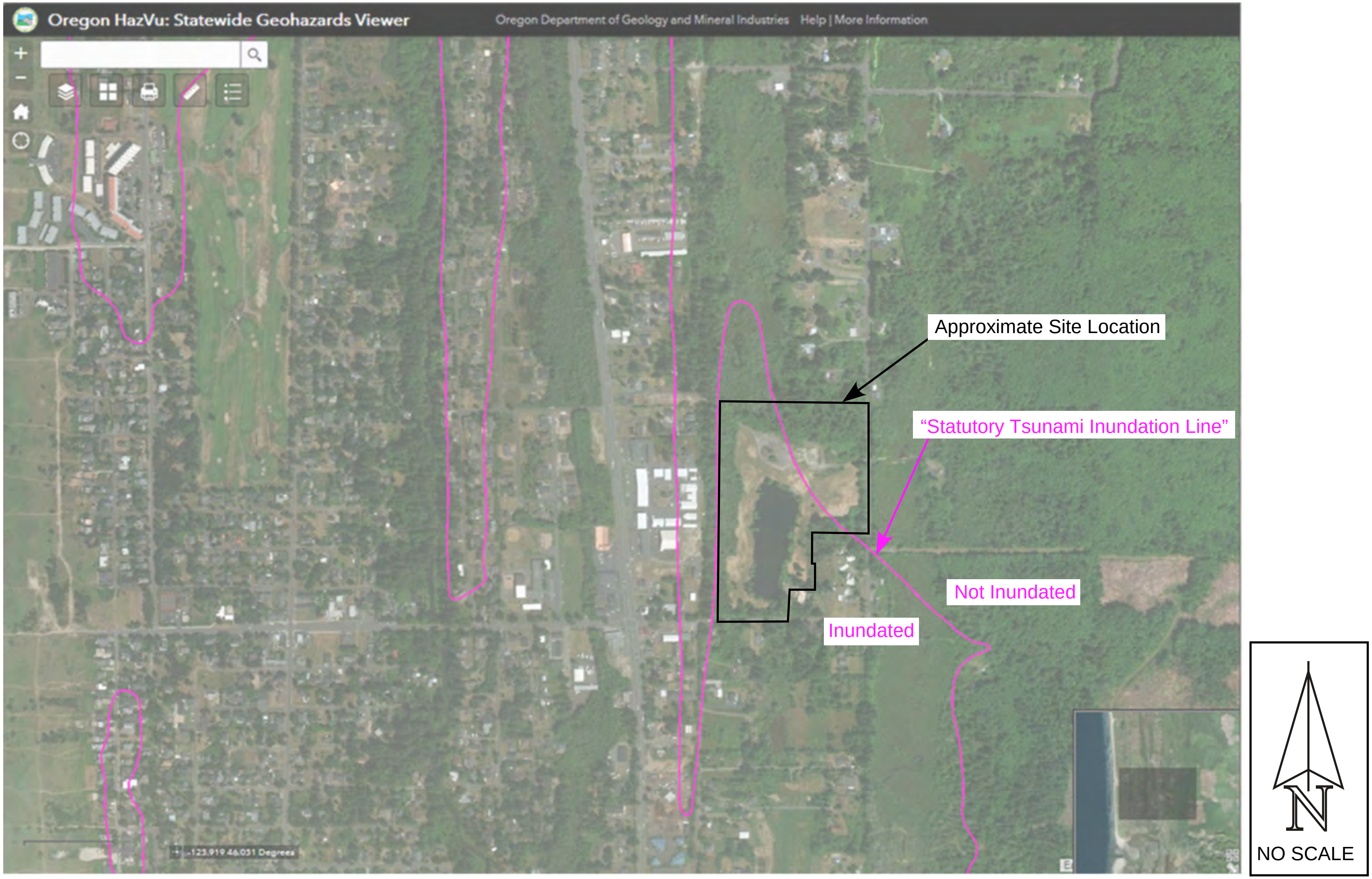
TSUNAMI INUNDATION MAPPING



Base map from DOGAMI “HAZVu” Website

Project: Palmberg Property Gearhart, Oregon	Project No. 18-2287	FIGURE 3

LOG OF BACKHOE / EXCAVATOR TEST PIT

Project: Palmberg Property
Gearhart, Oregon

Project No. 18-2287

Test Pit No. TP- 1

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Interval	Sample Designation	Moisture Content (%)	Groundwater	Material Description
						Soft, highly organic Silt, many roots, dark brown brown, very moist (Topsoil)
1						Soft, Silt, dark brown, moderately organic, some clay (Possible Fill/Disturbed Organic Soil Zone)
2						Approxim.. 18-inch diameter log at 2 feet bgs.
3						
						Medium dense, Silty Sand to Sand with Silt, brown and gray, sand is fine to medium, very moist to wet, micaceous.
4					▽	Moderate to strong seepage at 4 feet bgs.
5						
6						
7						
						Test pit terminated at 7 feet bgs Moderate to strong seepage at 4 feet bgs Some caving of test pit sidewalls
8						
9						
10						
11						
12						
13						
14						
15						
16						
17						

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LEGEND

☒ S-1 — Soil Sample Depth Interval and Designation

▽ — Water Level at Time of Excavation

Date Excavated: 03-23-18

Logged By: SLH

LOG OF BACKHOE / EXCAVATOR TEST PIT

Project: Palmberg Property
Gearhart, Oregon

Project No. 18-2287

Test Pit No. TP- 2

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Interval	Sample Designation	Moisture Content (%)	Groundwater	Material Description
1						Soft, highly organic Silt, many roots, dark brown brown, very moist. Steel beam section near ground surface (Topsoil / Disturbed Zone)
2						
3					▽	Loose to medium dense, Sand with Silt and Gravel, brown, many cobbles to 10-inch size, very moist to wet.
4						Moderate to strong seepage at 3 feet bgs.
5						Medium dense, Sand with Silt, light brown to brown, sand is fine to medium, wet, micaceous.
6						
7						Test pit terminated at 6 feet bgs Moderate to strong seepage at 3 feet bgs Some caving of test pit sidewalls
8						
9						
10						
11						
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17						

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☒ S-1	▽
Soil Sample Depth Interval and Designation	Water Level at Time of Excavation

Date Excavated: 03-23-18

Logged By: SLH

LOG OF BACKHOE / EXCAVATOR TEST PIT

Project: Palmberg Property
Gearhart, Oregon

Project No. 18-2287

Test Pit No. TP- 3

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Interval	Sample Designation	Moisture Content (%)	Groundwater	Material Description
						Dense to very dense, Sand with Silt, light gray, moist. (Fill)
1						
2						Very dense to hard, Sand with Silt and Gravel, red brown and gray, moist, some cobbles. Gravel and cobbles are rounded to subrounded. (Fill)
3						
4						Test pit terminated at 3 feet bgs Refusal to medium excavator
5						
6						
7						
8						
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☒ S-1 — Soil Sample Depth Interval and Designation

▽ — Water Level at Time of Excavation

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LOG OF BACKHOE / EXCAVATOR TEST PIT

Project: Palmberg Property
Gearhart, Oregon

Project No. 18-2287

Test Pit No. TP- 4

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Interval	Sample Designation	Moisture Content (%)	Groundwater	Material Description
						3 inches topsoil/grass over Medium dense, Sandy Gravel, red brown, trace debris (wire), moist. (Fill)
1						Medium dense, Sand with Silt to Silty Sand, red brown mottled with yellow brown, moist, micaceous. Sand is fine to medium.
2						
3						
4						Gradational Contact Grades to Medium dense Silty fine to medium Sand, light gray, very moist to wet, micaceous.
5						
6					▽	Slow to moderate seepage at 6 feet bgs.
7						Test pit terminated at 7 feet bgs Slow to moderate seepage at 6 feet bgs Minor caving below groundwater seepage zone
8						
9						
10						
11						
12						
13						
14						
15						
16						
17						

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☒ S-1 Soil Sample Depth Interval and Designation

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LOG OF BACKHOE / EXCAVATOR TEST PIT

Project: Palmberg Property
Gearhart, Oregon

Project No. 18-2287

Test Pit No. TP- 5

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Interval	Sample Designation	Moisture Content (%)	Groundwater	Material Description
						Soft, Organic Silt, dark brown. (Topsoil)
1						Medium dense, Sand with Silt to Silty Sand, light brown and gray, moist to very moist, micaceous. Sand is fine to medium.
2						
3						
4						
5						
6						Becomes wet at 6 feet bgs.
7						Test pit terminated at 6 feet bgs Soil becomes wet at 6 feet bgs Minor caving at bottom of pit
8						
9						
10						
11						
12						
13						
14						
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16						
17						

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☒ S-1 — Soil Sample Depth Interval and Designation

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LOG OF BACKHOE / EXCAVATOR TEST PIT

Project: Palmberg Property
Gearhart, Oregon

Project No. 18-2287

Test Pit No. TP- 6

Depth (ft)	Pocket Penetrometer (tons/ft^2)	Sample Interval	Sample Designation	Moisture Content (%)	Groundwater	Material Description
						Soft, Organic Silt, dark brown. (Topsoil)
1						Medium dense, Sand with Silt, red brown and yellow brown with some mottling, moist.
2						Medium dense, Sand with Silt to Silty Sand, light brown and gray, moist to very moist, micaceous. Sand is fine to medium.
3						
4						
5						
6						
7						
8						
9						Test pit terminated at 9 feet bgs No seepage or groundwater observed
10						
11						
12						
13						
14						
15						
16						
17						

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Project: Palmberg Property
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Project No. 18-2287

Test Pit No. TP- 7

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Interval	Sample Designation	Moisture Content (%)	Groundwater	Material Description
1						6 inches of Topsoil/Grass over Loose to medium dense, Sand with Silt and Gravel, red brown, brown, gray, and black, moist. (Fill)
2						
3						
4						
5						
6						
7						Organic soil near bottom of test pit; possible old topsoil layer
8						Test pit terminated at 7.5 feet bgs No seepage or groundwater observed
9						
10						
11						
12						
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LOG OF BACKHOE / EXCAVATOR TEST PIT

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Test Pit No. TP- 8

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Interval	Sample Designation	Moisture Content (%)	Groundwater	Material Description
						Soft, Organic Silt, dark brown. Highly organic in upper 12 inches (Topsoil)
1						
2						Medium dense, Sand with Silt, gray to brown, moist, micaceous. Sand is fine to medium.
3						
4					▽	Slow to moderate seepage at 4 feet bgs.
5						
6						
7						
8						Test pit terminated at 7 feet bgs Slow to moderate seepage at 4 feet bgs Caving below groundwater seepage zone
9						
10						
11						
12						
13						
14						
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