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

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HGSI Project No. 16-2028

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**Subject: GEOTECHNICAL ENGINEERING REPORT**
**LOLICH AND BELLAIRS PROPERTIES**
**18185 & 18407 SW SCHOLLS FERRY ROAD**
**BEAVERTON, OREGON**

This report presents the results of a geotechnical engineering study conducted by Hardman Geotechnical Services Inc. (HGSI) for the above-referenced project. The purpose of this study was to evaluate subsurface conditions at the site and to provide geotechnical recommendations for site development.

## SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The site consists of two contiguous tax lots located generally north of SW Scholls Ferry Road and east of SW Strobel Road (see Vicinity Map, Figure 1). These properties are within the South Cooper Mountain Community Plan area and will be within the City of Beaverton, Oregon. The project totals about 28 acres, as summarized below. Please note that the parcel addresses and acreages were taken from the Washington County GIS website and may not be completely accurate.

| Tax Lot No. | Address | Approx. Acreage | House Constructed Date |
|---|---|---|---|
| 2S106-0000500 | 18185 SW Scholls Ferry Rd Bellairs Property | 15.95 | 2012 |
| 2S106-0000600 | 18407 SW Scholls Ferry Rd Lolich Property | 12.00 | 1955 |

Both parcels have existing homes, garages, outbuildings and/or barns. Site slopes are predominantly gentle. Steep fill slopes exist along a natural drainage feature in the southern portion of the site. Vegetation predominantly consists of cultivated fields with some wooded and lawn areas.

The proposed development includes grading the site to support residential lots. Associated underground utilities, roadways and water quality facilities are also planned. We anticipate site development will consist of residential structures up to three stories in height. A site plan and grading plan has not yet been developed. HGSI should review grading plans when they become available in order to provide additional geotechnical recommendations as needed.

## REGIONAL GEOLOGY AND SEISMIC SETTING

The subject site lies within the Portland Basin, a broad structural depression situated between the Coast Range on the west and the Cascade Range on the east. The Portland Basin is a northwest-southwest trending structural basin produced by broad regional downwarping of the area. The Portland Basin is approximately 20 miles wide and 45 miles long and is filled with consolidated and unconsolidated sedimentary rocks of late Miocene, Pliocene and Pleistocene age.

Regional geologic maps indicate the site is underlain by the Columbia River Basalt (Beeson et al., 1989; Madin, 1990). Since deposition, the upper surface of the basalt has typically weathered to in-situ residual soil consisting of reddish clay containing spheroidally weathered basalt fragments.

At least three major fault zones capable of generating damaging earthquakes are known to exist in the region. These include the Portland Hills Fault Zone, Gales Creek-Newberg-Mt. Angel Structural Zone, and the Cascadia Subduction Zone. These potential earthquake source zones are included in the determination of seismic design values for structures, as presented in the *Seismic Design* section.

## FIELD EXPLORATION

### Exploratory Test Pits

The site-specific exploration for this study was conducted on May 26, 2016 and consisted of nine test pits (designated TP-1 through TP-9) excavated to a maximum depth of 10 feet below ground surface (bgs) at the approximate locations shown on the attached Site Plan, 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 obtained from the explorations 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. 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 test pit logs are attached to this report. 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.

### Subgrade Soil Evaluation – DCP Testing

On May 26, 2016, HGSI conducted two Dynamic Cone Penetrometer (DCP) tests to determine the strength parameters of the in-situ soil for support of pavement. Tests were performed at the approximate locations shown on Figure 2. Test equipment and methodology were in general accordance with ASTM Test Method D6951/D6951M – 09, *Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications*. Correlated California Bearing Ratio (CBR) values at the test locations are summarized on Table 1, for the depth intervals indicated. Correlated CBR values were determined using ASTM D6951/D6951M - 09.

**Table 1. DCP Field Test Results and Correlated CBR Values**

| Test Designation | Test Location | Material Tested | Depth Interval (feet) | Average Penetration Per Blow (mm) | Correlated CBR |
|---|---|---|---|---|---|
| DCP-1 | See Figure 2 | Native Silt | 1.1 – 2.9 | 15 | 6.9 |
| DCP-2 | See Figure 2 | Native Silt | 1.3 – 3.1 | 16 | 6.5 |

## SUBSURFACE CONDITIONS

The following discussion is a summary of subsurface conditions encountered in our explorations. For more detailed information regarding subsurface conditions at specific exploration locations, refer to the attached test pit 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 are anticipated to consist of undocumented fill, topsoil, residual soil, and Columbia River Basalt as described below.

***Undocumented Fill –*** Undocumented fill was encountered in Test Pits TP-1, TP-3 and TP-7 to depths of 6, 4.5, and 1.5 feet, respectively. Fill encountered in TP-1 consisted of organic silt with abundant brick and concrete debris. Fill encountered in TP-3 consisted of sparsely organic silt. Fill encountered in TP-7 consisted of silt with sparse gravel. Other localized areas of undocumented fill may be present in the vicinity of the existing structures, driveway, or in other areas beyond our exploration locations.

***Topsoil –*** Topsoil was encountered in all of the test pits and ranged in thickness from 3 to 12 inches. The average depth of topsoil was approximately 6 inches. These soils generally consisted of dark brown, organic silt (OL).

***Residual Soil*** – Underlying the topsoil or fill, test pits encountered residual soil. Residual soils are formed by in-place weathering of the underlying bedrock materials. The residual soil generally consisted of stiff to very stiff, silt, brown with gray and orange mottling. Many test pits encountered trace fine sand at depths of 7 to 8 feet and fine sand content typically increased with depth. Within the residual soil in Test Pit TP-6, gravel sized, highly weathered basalt clasts were commonly encountered.

***Columbia River Basalt Bedrock:*** Test Pit TP-6 was excavated at the toe of a slope that in other areas has exhibited shallow Columbia River Basalt Bedrock. Although a continuous bedrock surface was not encountered on the site in our explorations, TP-6 encountered a 2-foot diameter boulder and another large rock of unknown size. Columbia River Basalt may be encountered below residual soil, particularly along the site's northern boundary.

## Groundwater

During the field exploration, no groundwater or seepage was encountered to the maximum depth of exploration, 10 feet. Perched groundwater conditions often occur over fine-grained native deposits such as those beneath the site, particularly during the wet season. It is anticipated that groundwater conditions will vary depending on the season, local subsurface conditions, changes in site utilization, and other factors. The groundwater conditions reported above are for the specific date and locations indicated, and therefore may not necessarily be indicative of other times and/or locations.

# CONCLUSIONS AND RECOMMENDATIONS

Results of this study indicate that the proposed development is geotechnically feasible, provided that the recommendations of this report are incorporated into the design and construction phases of the project. The primary geotechnical constraint is the presence of undocumented fill along the drainage in the southern portion of the site. At the time of this report, site and grading plans were not yet developed. HGSI should review grading plans when they become available, and provide additional geotechnical recommendations as needed.

Recommendations are presented below regarding slope stability, site preparation, engineered fill, fill slope keying and benching, wet weather earthwork, spread footing foundations, below grade structural retaining walls, concrete slabs-on-grade, perimeter footing drains, seismic design, excavating conditions and utility trench backfill, typical pavement sections, and erosion control considerations.

## Slope Stability

Most of the site is gently sloping and does not have sufficient topographic gradient to have potential for development of unstable slopes in a static condition. No indications of previous slope failure or unstable soils were observed. Based on results of this study it is our opinion that on-site slopes have adequate factors of safety considering gross (overall) stability.

Steep slopes exist near the drainage in the southern portion of the site. These slopes are composed of undocumented fill. If homes, roads, or other settlement sensitive improvements are planned in these areas, undocumented fill should be removed to competent native soil. In this case, the height of the slopes will be significantly reduced and underlying native soils are considered to be competent and resistant to slope failure. If development or grading is not planned within these areas, undocumented fill may remain in place. Existing slopes composed of undocumented fill appear to be reasonably stable. Adequate structural setbacks from undocumented fill slope should be provided. HGSI should review grading plans when they become available to more fully evaluate slope issues based on the site and grading plans.

During and following site development within areas of steep slopes, surface runoff should be collected and storm water should be discharged in a controlled manner. In no case should uncontrolled stormwater runoff be allowed to flow over slopes. It should be noted that this evaluation is based on limited observation of surficial features, the subsurface explorations performed and review of available geologic literature. Deep subsurface explorations and quantification of slope stability factors of safety using numerical methods were beyond the scope of this study.

## Site Preparation

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. We anticipate that the average depth of topsoil stripping will be 6 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.

Undocumented fill was encountered in three test pits (TP-1, TP-3 and TP-7). Test Pit TP-1 and TP-3 were excavated at opposing ends of a long undocumented fill ridge line along the east side of the existing drainage in the southern portion of the site. Based on topographic expression, undocumented fill likely exists the entire length of the fill slope between TP-1 and TP-3. Undocumented fill encountered in TP-7 appears to be surficial and limited to a relatively small area. There is potential for old fills to be present on site in areas beyond our explorations. Where encountered beneath proposed structures, pavements, or other settlement-sensitive improvements, undocumented fill should be removed down to firm inorganic native soils and the removal area backfilled with engineered fill (see below). HGSI should observe removal excavations prior to fill placement to verify that overexcavations are adequate and an appropriate bearing stratum is exposed.

In construction areas, once stripping 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 during dry weather, 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, and during wet weather, 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**

In general, we anticipate that on-site native soils will be suitable for use as engineered fill in dry weather conditions, provided they are relatively free of organics and are properly moisture conditioned for compaction. Undocumented fill soils encountered in Test Pit TP-1 are not considered suitable for use as engineered fill. Undocumented fill in TP-3 and TP-7 may be used as engineered fill provided it is properly moisture conditioned and mixed with non-organic soils, as needed. 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 8 inches using standard compaction equipment. We recommend that engineered fill be compacted to at least 90 percent of the maximum dry density determined by ASTM D1557 (Modified Proctor) or equivalent. On-site soils may be wet or dry of optimum; therefore, we anticipate that moisture conditioning 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 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.

**Fill Slope Keying and Benching**

Although most of the site is gently sloping, steep slopes exist along the drainage in the southern portion of the site. If these areas are to be graded, we recommend that fill slopes be planned no steeper than 2H:1V and be constructed in accordance with the Fill Slope Detail, Figure 3. For fill slopes constructed at 2H:1V or flatter, and comprised of engineered fill placed and compacted as recommended herein, we anticipate that adequate factors of safety against global failure will be maintained.

Prior to placing compacted fill against the existing natural slopes, loose undocumented fill, topsoil, and soft soils should first be removed. Adequate benching should be maintained. Fill slope keyways should be constructed with a minimum depth of 2 feet and minimum width of H/3 (10 feet minimum), where H equals the vertical height between the base and top of the fill slope. Both benches and keyways should be roughly horizontal in the down slope direction. A subdrain should be incorporated in the fill slope keyway, and HGSI should observe the keyway excavations prior to the placement of fill.

Measures should be taken to prevent surficial instability and/or erosion of embankment material. This can be accomplished by conscientious compaction of the embankment fills all the way out to the slope face, by maintaining adequate drainage, and planting the slope face as soon as possible after construction. To achieve the specified relative compaction at the slope face, it may be necessary to overbuild the slopes several feet, and then trim back to design finish grade. In our experience, compaction of slope faces by “track-walking” is generally ineffective and is therefore not recommended.

**Wet Weather Earthwork**

The on-site soils are moisture sensitive and may 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, HGSI should be contacted for additional recommendations.

Under wet weather, the construction area will unavoidably become wet and the condition of fill or native soils exposed will degrade. To limit the impacts of wet weather on the finished building pad surface, consideration may be given to placement of a crushed aggregate pad. Where used, we recommend the working pad be constructed using 1½”–0 crushed aggregate, and should have minimum thickness of at least 12 inches. This thickness is considered adequate to support light construction traffic, but will not be sufficient to support heavy traffic such as loaded dump trucks or other heavy rubber-tired equipment.

**Spread Footing Foundations**

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 spread footings bearing on undisturbed native soils or engineered fill. The recommended maximum allowable bearing pressure may be increased by a factor of 1.33 for short term transient conditions such as wind and seismic loading. All footings should be founded at least 18 inches below the lowest adjacent finished grade. Minimum footing 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 0.45 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. Loose, wet or otherwise softened soil should be removed from the footing excavation prior to placing reinforcing steel bars. Due to the high moisture sensitivity of on-site soils, construction during wet weather may require overexcavation of footings and backfill with compacted, crushed aggregate.

**Below-Grade Structural 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 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 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 5H, where H is the total height of the wall.

We assume relatively level ground surface below the base of the walls. As such, we recommend passive earth pressure of 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- to 18-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.

**Concrete Slabs-on-Grade**

Preparation of areas beneath concrete slab-on-grade floors should be performed as recommended in the *Site Preparation* section. 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 200 kcf (115 pci) should be assumed for the soils anticipated at subgrade depth. This value assumes the concrete slab system is designed and constructed as recommended herein, with a minimum thickness of crushed rock of 8 inches beneath the slab.

Interior slab-on-grade floors should be provided with an adequate moisture break. The capillary break material should consist of ODOT open graded aggregate per ODOT Standard Specifications 02630-2. The minimum recommended thickness of capillary break materials on re-compacted soil subgrade is 8 inches. 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 90% of its maximum dry density as determined by ASTM D1557 or equivalent.

In areas where moisture will be detrimental to floor coverings or equipment inside the proposed structure, appropriate vapor barrier and damp-proofing measures should be implemented. A commonly applied vapor barrier system consists of a 10-mil polyethylene vapor barrier placed directly over the capillary break material. With this type of system, an approximately 2-inch thick layer of sand is often placed over the vapor barrier to protect it from damage, to aid in curing of the concrete, and also to help prevent cement from bleeding down into the underlying capillary break materials. Other damp/vapor barrier systems may also be feasible. Appropriate design professionals should be consulted regarding vapor barrier and damp proofing systems, ventilation, building material selection and mold prevention issues, which are outside HGSI's area of expertise.

**Perimeter Footing Drains**

Due to the potential for perched surface water above fine grained deposits and engineered fill such as those encountered at the site, we recommend the outside edge of perimeter footings be provided with a drainage system consisting of 4-inch minimum diameter perforated PVC pipe embedded in a minimum of 1 $ft^3$ per

lineal foot of clean, free-draining sand and gravel or 1”- ¼” 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.

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 2014 Oregon Residential Specialty Code (ORSC). We recommend Site Class D be used for design per ASCE 7-10, Chapter 20. Design values determined for the site using the USGS (United States Geological Survey) *Seismic Design Tool* utility are summarized below in Table 2.

**Table 2. Recommended Earthquake Ground Motion Parameters (2014 ORSC)**

| Parameter | Value |
|---|---|
| Location (Lat, Long), degrees | 45.4258, -122.8651 |
| Mapped Spectral Acceleration Values (MCE, Site Class B): | |
| Short Period, $S_S$ | 0.956 g |
| Soil Factors for Site Class D: | |
| $F_a$ | 1.118 |
| $SD_S = 2/3 \times F_a \times S_S$ | 0.712 g |
| Seismic Design Category (2014 ORSC Table R301.2.2.1.1) | $D_1$<br>$0.50g < SD_S < 0.83g$ |

Soil liquefaction is a phenomenon wherein saturated soil deposits temporarily lose strength and behave as a liquid in response to earthquake shaking. Soil liquefaction is generally limited to loose, granular soils located below the water table. Following development, on-site soils will consist predominantly of engineered fill or stiff native soils above the water table, which are not considered susceptible to liquefaction. Therefore, it is our opinion that special design or construction measures are not required to mitigate the effects of liquefaction.

## Excavating Conditions and Utility Trench Backfill

We anticipate that on-site soils can be excavated to depths of at least 10 feet using conventional heavy equipment such as trackhoes. Weathered basalt bedrock was not encountered in any of the test pits, however boulders were encountered in Test Pit TP-6 and weathered basalt bedrock may be encountered at depths below our explorations, particularly along the site’s northern boundary. HGSI’s experience at a neighboring site indicates that if encountered, basalt bedrock will likely consist of soft to medium hard basalt. Soft basalt can typically be excavated using a medium-sized excavator while medium hard basalt typically requires

chipping with a hydraulic hammer or mass excavation. Rock hardness may increase with depth and rock hardness beyond the depth of our exploratory test pits is unknown.

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. Flatter temporary excavation slopes will be needed if groundwater is present, or if significant thicknesses of sandy soils are present in excavation sidewalls.

Perched groundwater conditions often occur over fine-grained native deposits such as those beneath the site, particularly during the wet season. If 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. 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.

Utility trench backfill should consist of ¾”-0 crushed rock, compacted to at least 90% of the maximum dry density obtained by Modified Proctor (ASTM D1557) or equivalent. Initial backfill lift thick nesses 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, one density test is taken for every 4 vertical feet of backfill on each 200-lineal-foot section of trench.

**Typical Pavement Sections**

For on-site local residential streets, we recommended the following minimum pavement section for dry weather construction conditions.

**Table 3. Recommended Minimum Dry Weather Pavement Sections**

| Material Layer | Minimum Thickness (inches) Streets | Compaction Standard |
|---|---|---|
| Asphaltic Concrete (AC) | 3 | 92% of Rice Density (top lift)<br>91% of Rice Density (lower lifts)<br>AASHTO T-209 |
| Crushed Aggregate Base ¾”-0 (leveling course) | 2 | 95% of Modified Proctor<br>ASTM D1557 |
| Crushed Aggregate Base 1½”-0 | 8 | 95% of Modified Proctor<br>ASTM D1557 |
| Recommended Subgrade | 12 | 95% of Standard Proctor<br>or approved native |

In new pavement areas, the native soil subgrade should be ripped or tilled to a minimum depth of 12 inches, moisture conditioned, and recompacted in-place to at least 95 percent of ASTM D698 (Standard Proctor) or equivalent. In order to verify subgrade strength, we recommend proof-rolling directly on subgrade with a loaded dump truck during dry weather and on top of base course in wet weather. Soft areas that pump, rut, or weave should be stabilized prior to paving. If pavement areas are to be constructed during wet weather, HGSI should review subgrade at the time of construction so that condition specific recommendations can be provided. Wet weather pavement construction is likely to require soil amendment or geotextile fabric and an increase in base course thickness.

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

**Erosion Control Considerations**

Fine grained soils on steep slopes are susceptible to erosion. Erosion during construction can be minimized by implementing the project erosion control plan, which should include judicious use of bio-bags, silt fences, or other appropriate technology. Where used, 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.

## UNCERTAINTIES AND LIMITATIONS

We have prepared this report for the owner and his/her 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.**

REGISTERED PROFESSIONAL ENGINEER
64,422PE
OREGON
SEPTEMBER 26, 2000
SCOTT L. HARDMAN

6-1-16

**EXPIRES: 06-30-2017**

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

Attachments: References
Figure 1 – Vicinity Map
Figure 2 – Site Plan
Figure 3 – Fill Slope Detail
Logs of Test Pits TP-1 through TP-9

## REFERENCES

Beeson, M.H., Tolan, T.L., and Madin, I.P., 1991, Geologic map of the Lake Oswego Quadrangle, Multnomah, and Washington Counties, Oregon: Oregon Department of Geology and Mineral Industries Geological Map Series GMS-75, scale 1:24,000.

Gannett, M.W. and Caldwell, R.R., 1998, Geologic framework of the Willamette Lowland aquifer system, Oregon and Washington: U.S. Geological Survey Professional Paper 1424-A, 32 pages text, 8 plates.

Madin, I.P., 1990, Earthquake hazard geology maps of the Portland metropolitan area, Oregon: Oregon Department of Geology and Mineral Industries Open-File Report 0-90-2, scale 1:24,000, 22 p.

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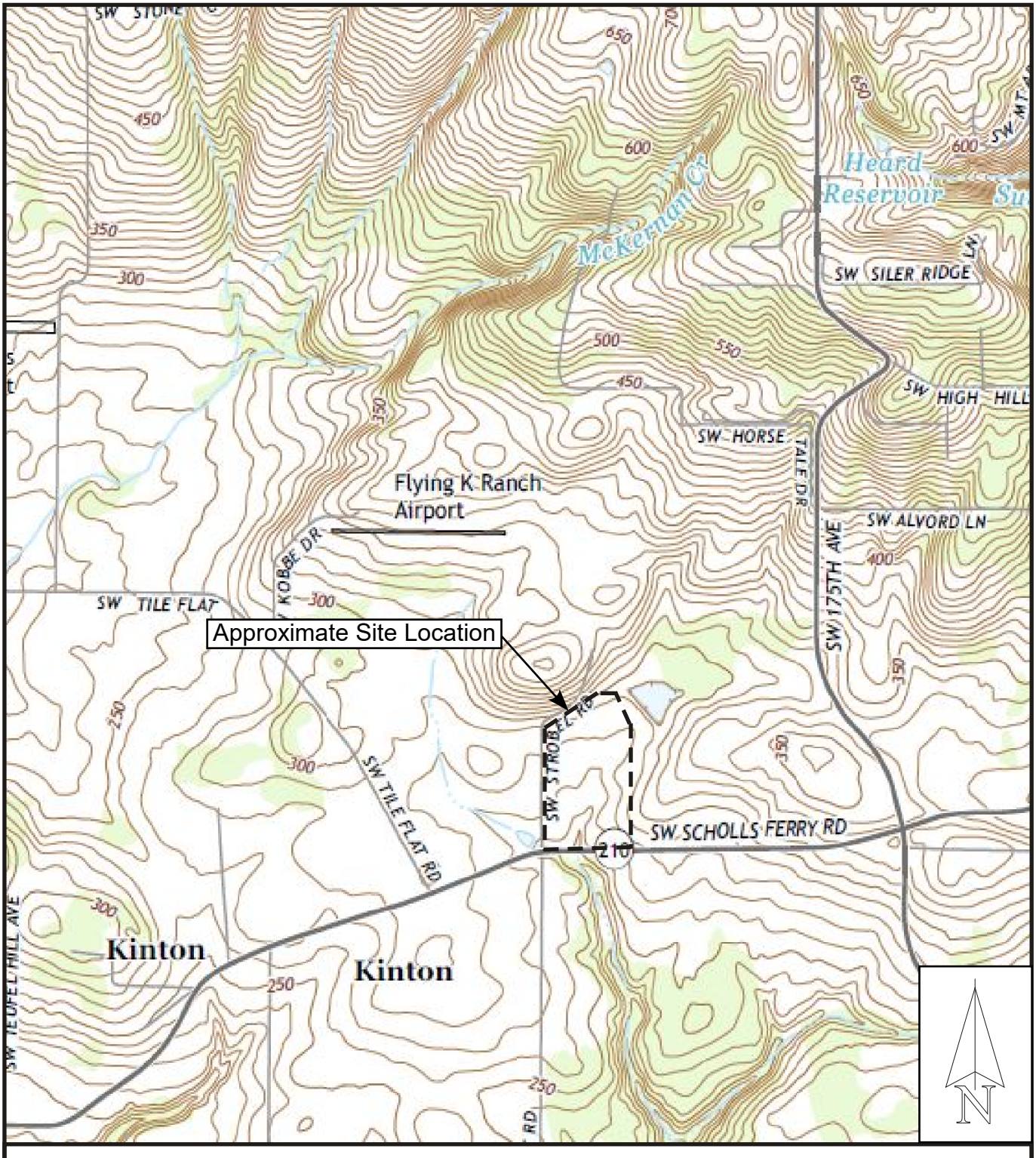
# VICINITY MAP




Base Map: USGS Scholls and Beaverton Quad, from US Topo, 2015

Not to Scale

| | | |
|---|---|---|
| Project: 18185 & 18407 SW Scholls Ferry Road<br>Beaverton, Oregon | Project No. 16-2028 | FIGURE 1 |

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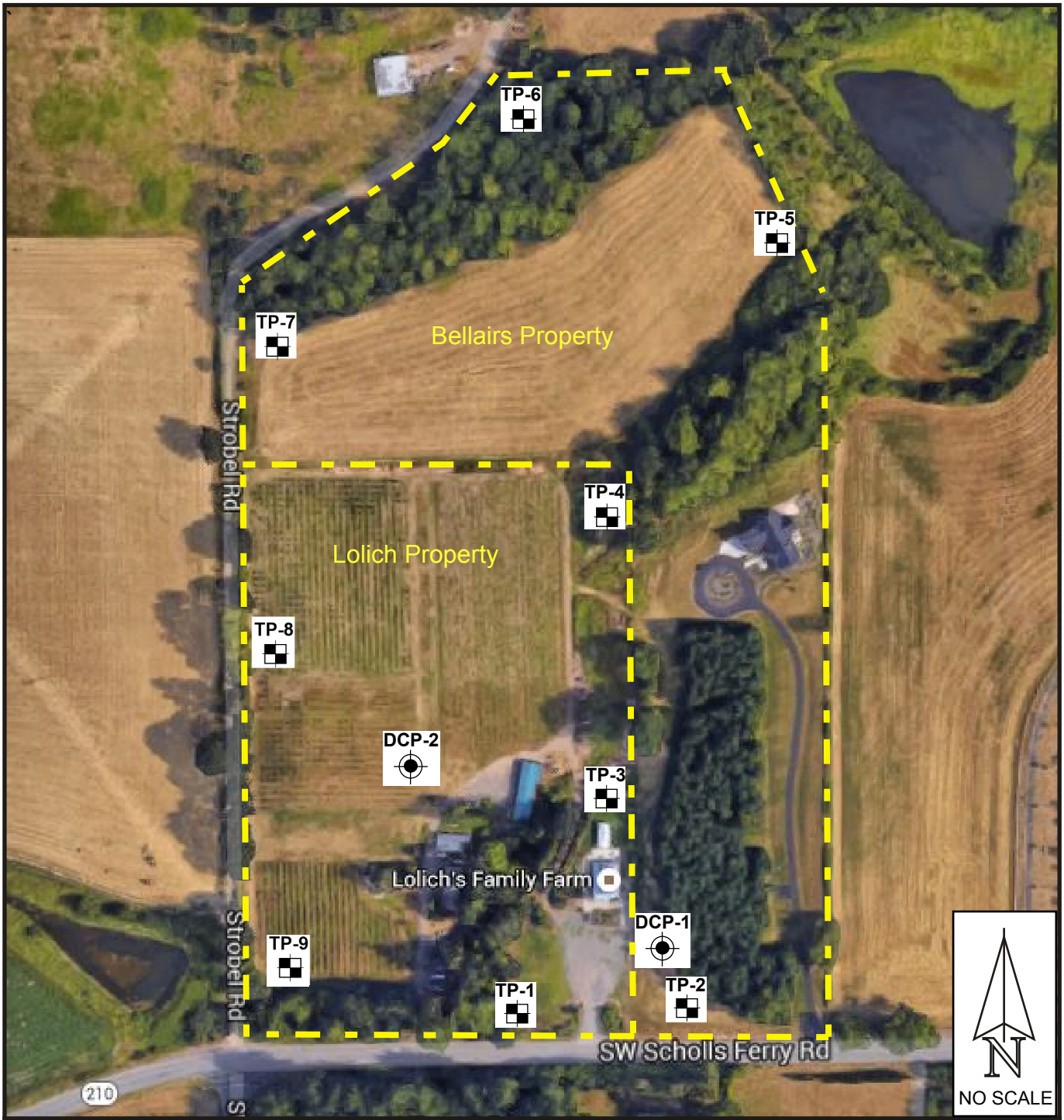
# SITE PLAN




Legend

TP-9 Test Pit, Approximate Location

DCP-2 DCP Test, Approximate Location

Approximate Site Boundary

Base map from Google Maps

| Project: 18185 & 18407 SW Scholls Ferry Road Beaverton, Oregon | Project No. 16-2028 | FIGURE 2 |
|---|---|---|

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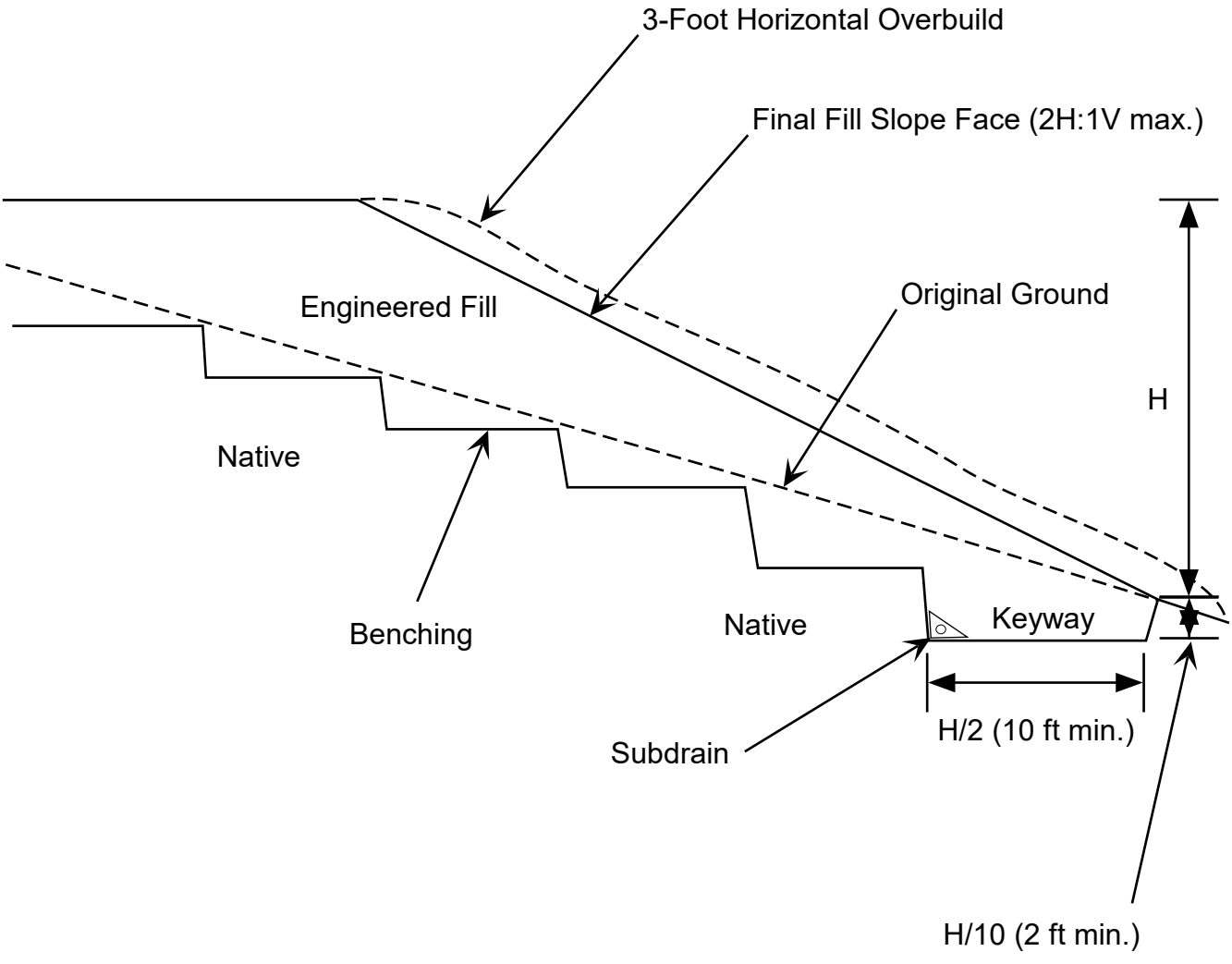
# FILL SLOPE DETAIL

## TYPICAL KEYWAY, BENCHING & FILL SLOPE DESIGN




Recommended subdrain is minimum 3-inch-diameter ADS Heavy Duty grade (or equivalent), perforated plastic pipe enveloped in a minimum of 3 cubic feet per lineal foot of 2" to 1/2" open-graded gravel drain rock wrapped with geotextile filter fabric (Mirafi 140N or equivalent).

| | | |
|---|---|---|
| Project: 18185 & 18407 Scholls Ferry<br>Beaverton, Oregon | Project No. 16-2028 | FIGURE 3 |

# TEST PIT LOG

| Project: 18185 & 18407 SW Scholls Ferry<br>Beaverton, Oregon | Project No. 16-2028 | Test Pit No. TP-1 |
|---|---|---|

| Depth (ft) | Sample Interval | Sample Designation | Pocket Penetrometer (tons/ft²) | Moisture Content (%) | Groundwater | Material Description |
|---|---|---|---|---|---|---|
| | | | | | | 3" Soft, highly organic SILT with angular gravel (OL), dark brown, moist (Fill) |
| | | | | | | Medium stiff, organic SILT with abundant brick and concrete debris up to 18 inches in diameter (OL), dark brown, moist (Fill) |
| 2 | | | | | | |
| 4 | | | | | | |
| 6 | | | | | | Medium stiff, highly organic SILT (OL), dark brown, moist (Buried topsoil) |
| | | | | | | Stiff, SILT (ML), micaceous, brown with gray mottling, moist (Residual soil) |
| 8 | | | | | | |
| 10 | | | | | | Test pit terminated at 10 feet<br>No groundwater or seepage encountered |
| 12 | | | | | | |
| 14 | | | | | | |
| 16 | | | | | | |

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LEGEND

S-1 — Soil Sample Depth Interval and Designation

Water Level at Time of Drilling

Date Excavated: 5/26/16

Logged By: PBR

# TEST PIT LOG

| Project: 18185 & 18407 SW Scholls Ferry<br>Beaverton, Oregon | Project No. 16-2028 | Test Pit No. TP-2 |
|---|---|---|

| Depth (ft) | Sample Interval | Sample Designation | Pocket Penetrometer (tons/ft²) | Moisture Content (%) | Groundwater | Material Description |
|---|---|---|---|---|---|---|
| | | | | | | Soft, highly organic SILT (OL), dark brown, moist (Topsoil) |
| | | | | | | Stiff to very stiff, SILT (ML), micaceous, brown with gray and orange mottling, moist (Residual soil) |
| 2 | | | | | | |
| 4 | | | | | | |
| 6 | | | | | | |
| | | | | | | Trace fine sand below 7.5 feet |
| 8 | | | | | | |
| | | | | | | Increased fine sand content with depth |
| 10 | | | | | | Test pit terminated at 10 feet<br>No groundwater or seepage encountered |
| 12 | | | | | | |
| 14 | | | | | | |
| 16 | | | | | | |

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Date Excavated: 5/26/16

Logged By: PBR

# TEST PIT LOG

| Project: 18185 & 18407 SW Scholls Ferry<br>Beaverton, Oregon | Project No. 16-2028 | Test Pit No. TP-3 |
|---|---|---|

| Depth (ft) | Sample Interval | Sample Designation | Pocket Penetrometer (tons/ft²) | Moisture Content (%) | Groundwater | Material Description |
|---|---|---|---|---|---|---|
| | | | | | | 4" Soft, highly organic SILT (OL), dark brown, moist (Fill) |
| | | | | | | Medium stiff, sparsely organic SILT (ML-OL), brown to dark brown, moist (Fill) |
| 2 | | | | | | |
| 4 | | | | | | |
| | | | | | | Stiff to very stiff, SILT (ML), micaceous, brown with gray mottling, moist (Residual soil) |
| 6 | | | | | | |
| | | | | | | Trace fine sand below 7 feet |
| 8 | | | | | | |
| | | | | | | Test pit terminated at 9 feet<br>No groundwater or seepage encountered |
| 10 | | | | | | |
| 12 | | | | | | |
| 14 | | | | | | |
| 16 | | | | | | |

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# TEST PIT LOG

| Project: 18185 & 18407 SW Scholls Ferry Beaverton, Oregon | Project No. 16-2028 | Test Pit No. TP- 4 |
|---|---|---|

| Depth (ft) | Sample Interval | Sample Designation | Pocket Penetrometer (tons/ft$^2$) | Moisture Content (%) | Groundwater | Material Description |
|---|---|---|---|---|---|---|
| | | | | | | 3" Soft, highly organic SILT (OL), dark brown, moist (Topsoil) |
| | | | | | | Stiff to very stiff, SILT (ML), micaceous, mottled gray and brown, moist (Residual soil) |
| 2 | | | | | | |
| 4 | | | | | | |
| 6 | | | | | | |
| | | | | | | Color change to brown with gray mottling |
| 8 | | | | | | |
| 10 | | | | | | Test pit terminated at 9.5 feet<br>No groundwater or seepage encountered |
| 12 | | | | | | |
| 14 | | | | | | |
| 16 | | | | | | |

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Water Level at Time of Drilling

Date Excavated: 5/26/16

Logged By: PBR

# TEST PIT LOG

| Project: 18185 & 18407 SW Scholls Ferry Beaverton, Oregon | Project No. 16-2028 | Test Pit No. TP- 5 |
|---|---|---|

| Depth (ft) | Sample Interval | Sample Designation | Pocket Penetrometer (tons/ft²) | Moisture Content (%) | Groundwater | Material Description |
|---|---|---|---|---|---|---|
| | | | | | | Soft, highly organic SILT (OL), dark brown, moist (Topsoil) |
| 2 | | | | | | Stiff to very stiff, SILT (ML), micaceous, mottled gray and brown, moist (Residual soil) |
| | | | | | | Color change to brown with gray mottling |
| 4 | | | | | | |
| 6 | | | | | | |
| 8 | | | | | | Trace fine sand below 8 feet |
| 10 | | | | | | Test pit terminated at 10 feet<br>No groundwater or seepage encountered |
| 12 | | | | | | |
| 14 | | | | | | |
| 16 | | | | | | |

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# TEST PIT LOG

| Project: 18185 & 18407 SW Scholls Ferry Beaverton, Oregon | Project No. 16-2028 | Test Pit No. TP- 6 |
|---|---|---|

| Depth (ft) | Sample Interval | Sample Designation | Pocket Penetrometer (tons/ft²) | Moisture Content (%) | Groundwater | Material Description |
|---|---|---|---|---|---|---|
| | | | | | | Soft, highly organic SILT (OL), dark brown, moist (Topsoil) |
| | | | | | | Stiff, SILT (ML), micaceous, brown with gray mottling, moist (Residual soil) |
| 2 | | | | | | |
| 4 | | | | | | |
| | | | | | | Rock encountered at 5.5 feet. Possible boulder, but unknown because it was too large to excavate from narrow test pit |
| 6 | | | | | | Two foot diameter boulder encountered at 6.5 feet |
| | | | | | | Very stiff, SILT with abundant gravel sized highly weathered basalt clasts (ML), micaceous, reddish brown, moist (Residual soil) |
| 8 | | | | | | |
| 10 | | | | | | Test pit terminated at 10 feet<br>No groundwater or seepage encountered |
| 12 | | | | | | |
| 14 | | | | | | |
| 16 | | | | | | |

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Water Level at Time of Drilling

Date Excavated: 5/26/16

Logged By: PBR

# TEST PIT LOG

| Project: 18185 & 18407 SW Scholls Ferry<br>Beaverton, Oregon | Project No. 16-2028 | Test Pit No. TP- 7 |
|---|---|---|

| Depth (ft) | Sample Interval | Sample Designation | Pocket Penetrometer (tons/ft$^2$) | Moisture Content (%) | Groundwater | Material Description |
|---|---|---|---|---|---|---|
| | | | | | | Soft to medium stiff, SILT with sparse angular gravel (ML), brown, moist (Fill) |
| 2 | | | | | | Soft, highly organic SILT (OL), dark brown, moist (Buried topsoil) |
| | | | | | | Stiff to very stiff, SILT (ML), micaceous, brown with gray and orange mottling, moist (Residual soil) |
| 4 | | | | | | |
| 6 | | | | | | Trace fine sand below 7 feet |
| 8 | | | | | | |
| 10 | | | | | | Test pit terminated at 9.5 feet<br>No groundwater or seepage encountered |
| 12 | | | | | | |
| 14 | | | | | | |
| 16 | | | | | | |

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S-1 — Soil Sample Depth Interval and Designation

Water Level at Time of Drilling

Date Excavated: 5/26/16

Logged By: PBR

# TEST PIT LOG

| Project: 18185 & 18407 SW Scholls Ferry<br>Beaverton, Oregon | Project No. 16-2028 | Test Pit No. TP- 8 |
|---|---|---|

| Depth (ft) | Sample Interval | Sample Designation | Pocket Penetrometer (tons/ft²) | Moisture Content (%) | Groundwater | Material Description |
|---|---|---|---|---|---|---|
| | | | | | | Soft, highly organic SILT (OL), dark brown, moist (Topsoil) |
| 2 | | | | | | Stiff to very stiff, SILT (ML), micaceous, brown with gray and orange mottling, moist (Residual soil) |
| 4 | | | | | | |
| 6 | | | | | | |
| 8 | | | | | | Trace fine sand below 8 feet |
| 10 | | | | | | Test pit terminated at 10 feet<br>No groundwater or seepage encountered |
| 12 | | | | | | |
| 14 | | | | | | |
| 16 | | | | | | |

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# TEST PIT LOG

| Project: 18185 & 18407 SW Scholls Ferry<br>Beaverton, Oregon | Project No. 16-2028 | Test Pit No. TP- 9 |
|---|---|---|

| Depth (ft) | Sample Interval | Sample Designation | Pocket Penetrometer (tons/ft$^2$) | Moisture Content (%) | Groundwater | Material Description |
|---|---|---|---|---|---|---|
| | | | | | | Soft, highly organic SILT (OL), dark brown, moist (Topsoil) |
| | | | | | | Stiff to very stiff, SILT (ML), micaceous, brown with gray and orange mottling, moist (Residual soil) |
| 2 | | | | | | |
| 4 | | | | | | |
| 6 | | | | | | |
| 8 | | | | | | Trace fine sand below 8.5 feet |
| 10 | | | | | | Test pit terminated at 10 feet<br>No groundwater or seepage encountered |
| 12 | | | | | | |
| 14 | | | | | | |
| 16 | | | | | | |

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