Real-World Geotechnical Solutions
Investigation • Design • Construction Support

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**RE: GEOTECHNICAL INVESTIGATION & LANDSLIDE HAZARD STUDY
13221 NW MCNAMEE ROAD HOMESITE
MULTNOMAH COUNTY, OREGON**

Reference: *Geotechnical Investigation and Landslide Hazard Study, Luethe Partition Lot 1, Section 32 2N 1W Tax Lot 800, Multnomah County, Oregon,* GeoPacific Engineering, Inc. report dated March 17, 2010.

This report presents the results of a geotechnical investigation and landslide hazard study for the proposed new single-family home on NW McNamee Road in Multnomah County, Oregon. The primary purpose of this study was to evaluate geological hazards, soil conditions and land suitability criteria specified by Multnomah County Code MCC 33.7890 with respect to residential homesite construction. The scope of our investigation included review of published geologic maps and Lidar based high resolution digital elevation maps, field reconnaissance, exploratory test pits, and preparation of the report. It is our understanding that a landslide hazard study is required because the property is within a mapped landslide inventory area. GeoPacific Engineering has previously issued the results of a Geotechnical Investigation and Landslide Hazard Study for the site (known as Luethe Partition Lot 1) dated March 17, 2010. Since issuing the report in 2010, site grading for the driveway has been performed.

SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The subject site is approximately 2 acres in size located off the south side of NW McNamee Road in Multnomah County, Oregon. The proposed homesite is situated on a gently-sloping ridgeline adjacent to an incised drainage gully to McCarthy Creek with moderate side slopes inclining to the east with grades up to approximately 50 percent (Figures 1 & 2). Topography in the vicinity of the proposed home is on the order of 8 to 10 percent grade (Figure 3). Up to 25 feet of engineered fill has been placed to the east of the existing driveway during the mid-1990's and 2014 to 2016. The site is currently unimproved with the exception of the existing driveway.

It is our understanding that the proposed improvements will consist of the construction of one single family home, driveway, septic field, and associated underground utilities. The home will be two stories in height. The grading plan indicates cuts and fills up to 2 feet are planned.

REGIONAL GEOLOGY

The subject site is underlain by Quaternary age (last 1.6 million years) loess, a windblown silt deposit that mantles older deposits, basalt bedrock, and elevated areas in the Portland region (Beeson et al., 1989; Madin, 1990). The loess generally consists of massive silt deposited following repeated catastrophic flooding events in the Willamette Valley, the last of which occurred about 10,000 years ago. In localized areas, the loess includes buried paleosols that developed between depositional events. Regionally, the total thickness of loess ranges from 5 feet to greater than 100 feet.

The loess is underlain by the Columbia River Basalt Formation (Phillips, 1987). The Miocene aged (about 14.5 to 16.5 million years ago) Columbia River Basalts are a thick sequence of lava flows which form the crystalline basement of the Tualatin Valley (Madin, 1990). The basalts are composed of dense, finely crystalline rock that is commonly fractured along blocky and columnar vertical joints. Individual basalt flow units typically range from 25 to 125 feet thick and interflow zones are typically vesicular, scoriaceous, brecciated, and sometimes include sedimentary rocks.

REGIONAL SEISMIC SETTING

At least three potential source zones capable of generating damaging earthquakes are thought to exist in the region. These include the Portland Hills Fault Zone, Gales Creek-Newberg-Mt. Angel Structural Zone, and the Cascadia Subduction Zone, as discussed below.

Portland Hills Fault Zone

The Portland Hills Fault Zone is a series of NW-trending faults that include the central Portland Hills Fault, the western Oatfield Fault, and the eastern East Bank Fault. These faults occur in a northwest-trending zone that varies in width between 3.5 and 5.0 miles. The combined three faults vertically displace the Columbia River Basalt by 1,130 feet and appear to control thickness changes in late Pleistocene (approx. 780,000 years) sediment (Madin, 1990). The Portland Hills Fault occurs along the Willamette River at the base of the Portland Hills, and is approximately 1.1 miles northeast of the site. The East Bank Fault occurs along the eastern margin of the Willamette River, and is located approximately 2.2 miles east of the site. The Oatfield Fault occurs along the western side of the Portland Hills, and is approximately 2.9 miles south of the site. The accuracy of the fault mapping is stated to be within 500 meters (Wong, et al., 2000). No historical seismicity is correlated with the mapped portion of the Portland Hills Fault Zone, but in 1991 a M3.5 earthquake occurred on a NW-trending shear plane located 1.3 miles east of the fault (Yelin, 1992). Although there is no definitive evidence of recent activity, the Portland Hills Fault Zone is assumed to be potentially active (Geomatrix Consultants, 1995).

Gales Creek-Newberg-Mt. Angel Structural Zone

The Gales Creek-Newberg-Mt. Angel Structural Zone is a 50-mile-long zone of discontinuous, NW-trending faults that lies approximately 16.7 miles southwest of the subject site. These faults are recognized in the subsurface by vertical separation of the Columbia River Basalt and offset seismic reflectors in the overlying basin sediment (Yeats et al., 1996; Werner et al., 1992). A geologic reconnaissance and photogeologic analysis study conducted for the Scoggins Dam site in the Tualatin Basin revealed no evidence of deformed geomorphic surfaces along the structural zone (Unruh et al., 1994). No seismicity has been recorded on the Gales Creek Fault (the fault closest to the subject site); however, these faults are considered to be potentially active because they may connect with the seismically active Mount Angel Fault and the rupture plane of the 1993 M5.6 Scotts Mills earthquake (Werner et al. 1992; Geomatrix Consultants, 1995).

Cascadia Subduction Zone

The Cascadia Subduction Zone is a 680-mile-long zone of active tectonic convergence where oceanic crust of the Juan de Fuca Plate is subducting beneath the North American continent at a rate of 4 cm per year (Goldfinger et al., 1996). A growing body of geologic evidence suggests that prehistoric subduction zone earthquakes have occurred (Atwater, 1992; Carver, 1992; Peterson et al., 1993; Geomatrix Consultants, 1995). This evidence includes: (1) buried tidal marshes recording episodic, sudden subsidence along the coast of northern California, Oregon, and Washington, (2) burial of subsided tidal marshes by tsunami wave deposits, (3) paleoliquefaction features, and (4) geodetic uplift patterns on the Oregon coast. Radiocarbon dates on buried tidal marshes indicate a recurrence interval for major subduction zone earthquakes of 250 to 650 years with the last event occurring 300 years ago (Atwater, 1992; Carver, 1992; Peterson et al., 1993; Geomatrix Consultants, 1995). The inferred seismogenic portion of the plate interface lies approximately along the Oregon Coast at depths of between 20 and 40 kilometers below the surface.

SUBSURFACE CONDITIONS

Our site-specific exploration for this report was conducted on February 10th, 2010. Two exploratory test pits were excavated with a medium sized trackhoe to depths ranging between 8 and 8.5 feet at the approximate locations shown on Figure 3. A reconnaissance was performed on November 6, 2018. It should be noted that exploration locations were located in the field by pacing or taping distances from apparent property corners and other site features shown on the plans provided. As such, the locations of the explorations should be considered approximate.

A GeoPacific geologist continuously monitored the field exploration program and logged the test pits. Soils observed in the explorations were classified in general accordance with the Unified Soil Classification System (USCS). During exploration, our geologist also noted geotechnical conditions such as soil consistency, moisture and groundwater conditions. Logs of test pits are attached to this report. The following report sections are based on the exploration program and summarize subsurface conditions encountered at the site.

Undocumented Fill: Undocumented fill was not encountered in our explorations conducted for this study; however, areas of fill may be present outside our exploration locations. Engineered fill was placed along the driveway alignment to the northeast and east of the proposed home.

Topsoil Horizon: Directly underlying the ground surface in test pits was a topsoil horizon consisting of organic SILT (OL) with a 3-inch- to 4-inch thick root mat for low grasses. The topsoil was brown to dark brown in color and had a mixed structure, presumably due to land clearing activities and/or agricultural tilling. The total thickness of the topsoil horizon is approximately 8 to 10 inches.

Native Soil Horizon: Underlying the topsoil horizon was a native soil weathering horizon consisting of clayey SILT (ML). The soil color varies from light brown to mixed brown, light brown, orange and gray. Field pocket penetrometer measurements indicate approximate unconfined compressive strengths of 0.5 to 4.0 tons/ft^2 under damp to moist conditions. These measurements are considered consistent with a medium-stiff to very-stiff consistency. In test pits TP-1 and TP-2, the native soil horizon is approximately 1 to 3 feet thick.

Quaternary Loess Deposit: Underlying the native soil horizon was a deposit of Quaternary windblown loess that consists of clayey SILT (ML) to silty CLAY (CL). The loess was generally uniform in texture, had a low to moderate plasticity, and a very-stiff consistency. Field pocket penetrometer measurements indicate an approximate unconfined compressive strength of 3.5 to 4.0 tons/ft^2. In test pits, the loess deposit was greater than 6 feet in thickness and extended below the maximum depth explored (8 feet below the ground surface).

Soil Moisture and Groundwater

On February 10, 2010, soil moisture conditions observed in test pits were generally damp to moist. Neither perched groundwater nor seepage was encountered in test pits excavated to a maximum depth of 8 feet below the ground surface. Experience has shown that temporary storm related perched groundwater within the near surface soils often occur over fine-grained native deposits such as those beneath the site 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.

INFILTRATION TESTING

On February 10, 2010, three open-hole, falling-head infiltration tests were performed at the home site in general accordance with City of Portland Stormwater Management Manual guidelines. The tests were conducted in 8-inch diameter holes excavated into native soils at an approximate depth of 4 feet below the ground surface. The test holes were pre-saturated for 4 hours prior to performing the final test measurements. During the tests, water levels were measured over 15 minute intervals with approximate head pressures ranging between 8 and 10 inches until three successive measurements showing a consistent infiltration rate were achieved. Soils encountered in the pits consisted of clayey silt with a field determined Unified Soil Classification System designation of ML. The approximate test location is shown in Figure 3.

The results of our infiltration test measurements indicate that the average infiltration rate is 1.37 inches per hour. These results indicate that infiltration rates at the site are low, such that water tends to flow laterally in the upper few feet of soil rather than vertically. Due to the potential to adversely affect slope stability, infiltration of stormwater should be avoided.

SLOPE STABILITY

For the purpose of evaluating slope stability, we reviewed regional site topography (Figure 1), reviewed published geologic mapping and Lidar based high resolution digital elevation maps (Figure 2), performed a field reconnaissance on November 20, 2020, and explored subsurface conditions at the proposed homesite with two exploratory test pits, the locations of which are indicated on Figure 3.

Our review 1:24,000 scale topographic mapping by the U.S. Geological Survey indicate that the vicinity topography is smooth and uniform, consistent with relatively stable slope conditions. The Lidar based high resolution digital elevation maps depict topography following early site grading that occurred in the mid 1990’s and prior to grading that was completed in 2016 (DOGAMI, 2021). The statewide landslide database and available landslide inventory mapping indicate no mapped landslides are present in the vicinity of the homesite; however, the headscarp of a prehistoric (>150 years old) debris flow is mapped within the incised drainage gully located approximately 60 feet to the north of the proposed homesite (Burns et al, 2011; SLIDO, 2021) as shown on Figure 2. The head of this debris flow is located in an area that has since been filled with engineered fill. Several other historic and prehistoric earthflow type landslides are mapped along the McCarthy Creek drainage located east of the site (SLIDO, 2021). Seismic slope instability hazard mapping identifies the site vicinity as being in a low relative slope instability hazard zone based primarily on slope steepness (Mabey et al., 1996).

No geomorphic evidence of prior, deep seated slope instability (such as hummocky topography, benches, or old scarps) was observed during our reconnaissance. No seeps or springs were observed. Some areas of shallow slope instability were observed along the fillslope to the northeast of the proposed home. These features originated approximately 15 feet downslope of the existing driveway grade and were less than 3 feet in depth.

Topographic mapping indicates that the proposed building site is situated on a gently sloping ridge with grades of approximately 8 to 17 percent (Figure 3) at elevations of approximately 900 to 930 feet above mean sea level (Figure 1). The homesite is adjacent to an engineered fill slope inclining at a gradient of 2H:1V.

Exploratory test pit data of the homesite vicinity indicates that the slope is underlain by loess soils characterized by a very stiff consistency. No weak zones such as volcanic ash layers were observed in explorations and contacts between the layers appeared to be gradational. Our explorations indicate that native soils underlying the slope are characterized by moderate to high shear strength and a moderate to high resistance to slope instability on gentle to moderately steep slopes.

GeoPacific should review the final grading and building plans to verify compliance with the geotechnical recommendations and to make additional recommendations, if necessary. Septic drain fields should be located on topography sloping less than 25 percent grade and located at least 50 feet from foundations. We recommend that surface runoff be collected and water discharged the proposed stormwater facility that is to be located to the southwest and east of the proposed home. Due to the potential to adversely affect slope stability, the stormwater facility should be lined with an impermeable barrier and infiltration of stormwater should be avoided. The flow dispersal trench should overflow to existing drainage channels with culverts and/or enclosed pipes. 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 test pits performed, and review of available geologic literature.

CONCLUSIONS AND RECOMMENDATIONS

Our investigation indicates that the subject site is suitable from a geologic standpoint for support of conventional spread foundations provided that the following recommendations are incorporated into the design and construction phases of the project. In our opinion, the potential for damage to the single-family home due to slope instability is low provided that the project is designed and constructed in accordance with our recommendations. In order to minimize the potential for undermining of the foundation due to shallow slope instability, we recommend a 3 to 5 foot embedment of the footings into native soils. A footing to slope setback of 20 feet should be maintained. Due to the potential to adversely affect slope stability, the stormwater facility should be lined with an impermeable barrier and infiltration of stormwater should be avoided. GeoPacific

Engineering should review the foundation excavation during construction to verify embedment, subgrade bearing strength, and footing-to-slope setbacks.

Maintenance of Hillside Homesites

Homes on hillside lots require additional maintenance measures because they are subject to natural slope processes such as runoff, erosion, shallow soil sloughing, soil creep, perched groundwater, etc. An abbreviated checklist of common Do's and Don'ts recommended for maintaining hillside homesites is attached. This checklist should be provided to any future homeowners, who are responsible to maintain the property adequately.

No additional fill should be placed on this site without careful review by a qualified geotechnical engineer. It is likely that placement of additional fills, construction of new retaining walls or other features would require significant additional stabilization measures.

Stormwater Disposal and Proposed Septic Field

For maintaining slope stability immediately below the homesite, we recommend that stormwater facilities be lined with an impermeable barrier and not be discharged directly to slopes. For control of stormwater, we suggest that runoff from the home be routed in a controlled manner to the proposed stormwater facilities and that overflow be discharged to existing drainages or discharged well down slope of the structure. The proposed stormwater disposal system includes flow dispersal trenches to the southwest and east of the proposed home. Infiltration of stormwater on slopes immediately adjacent to the home should be avoided. Footing and retaining wall subdrains may outlet to slopes since discharge rates from these subdrains are expected to be minimal.

Septic systems should be carefully sited to limit potential adverse effects on slope stability. The proposed primary septic field area is located to the south and southeast of the proposed home in an area where grades are on the order of 10 to 20 percent. Septic fields can be installed on slopes up to 35 percent grade without adverse slope stability impacts.

Slope Stability

In our opinion, the potential for damage to the proposed building due to deep-seated slope instability is low provided that the project is designed and constructed in accordance with our recommendations. Exploratory test pits indicate that the homesite is underlain by competent, very-stiff, clayey silt to silty clay characterized by a moderate to high shear strength and moderate to high resistance to slope instability on gently sloping topography. We recommend a minimum footing to slope setback distance from the gulley side slope of 20 feet horizontal for structures intended for human occupancy. Further geotechnical review would be required for construction proposed closer to the slope than the recommended setback.

In order to minimize the potential for undermining of the foundation due to shallow slope instability of the adjacent engineered fill slope, we recommend a minimum of 3 to 5 feet embedment of foundation footings such that footings are founded on stiff to very stiff loess soils. GeoPacific should review the foundation excavation to verify exposed soil conditions prior to pouring footings. As with all hillside homesites, we recommend that the owner maintain this property in a manner appropriate to hillside development as outlined in the attached "Maintenance of Hillside Homesites."

Site Preparation

Areas of proposed construction and areas to receive fill should be cleared of vegetation and any organic and inorganic debris. Inorganic debris and organic materials from clearing should be stockpiled away from the homesite. Organic-rich root zones should then be stripped from construction areas of the site or where engineered fill is to be placed. The estimated average necessary depth of removal in undisturbed areas for moderately organic soils is about 12 inches. The final depth of soil removal will be determined on the basis of a site inspection after the stripping/excavation has been performed. Stripped topsoil should preferably be stockpiled away from the building site. Any remaining topsoil should be stockpiled only in designated areas and stripping operations should be observed and documented by the geotechnical engineer or his representative.

If encountered, undocumented fill in structural areas should be removed and the excavations backfilled with engineered fill as recommended in the *Grading and Erosion Contro*l section in this report.

Grading and Erosion Control

In general, we anticipate that soils from utility trench excavations will be suitable for reuse as nonstructural trench backfill provided they are adequately moisture conditioned prior to compacting. Fill placement for foundation backfilling should be kept to a minimum, particularly on sloping topography. Imported fill material should be reviewed by GeoPacific 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. For landscape, stockpiles, or other nonstructural fill areas, 85 percent of modified Proctor maximum dry density is recommended and limited to 3 feet in thickness.

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 generally 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 cubic yards, 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.

Permanent cut slopes should be constructed no steeper than 2H:1V (50% grade). Due to the presence of moderately to steeply sloping gradients, we consider the potential for adverse erosion during construction to be moderate. Erosion at the site during construction can be minimized by implementing the project erosion control plan, which should include judicious use of straw wattles and silt fences. If used, these erosion control devices should be in place and remain in place throughout site preparation and construction. Due to the fine-grained nature of on-site soils, once particles become suspended by disturbance in ponded water they will precipitate slowly.

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. Cut and fill slopes should be seeded or planted as soon as possible after construction, so that vegetation has time to become established before the onset of the next wet-weather season.

Spread Foundations

The subject lot is suitable for spread foundations bearing on competent, native soil, and/or engineered fill. As previously discussed, footings should be founded on stiff to very stiff loess soils with a minimum embedment of 3 to 5 feet. GeoPacific should review the foundation excavation to verify exposed soil conditions prior to pouring footings. Foundation design, construction, and setback requirements should conform to applicable building codes at the time of permitting. For protection against frost heave, spread footings should be embedded at a minimum depth of 18 inches below exterior grade. The recommended minimum width for continuous footings supporting wood-framed walls without masonry is 15 inches for a two-story and 18 inches for a three-story building. Minimum foundation reinforcement should consist of one No. 4 bar at the top of stem wall, and one No. 4 bar at the bottom of the footing. Concrete slab-on-grade reinforcement should consist of No. 4 bars placed on 24-inch centers in a grid pattern. Actual footing widths, sizing, and reinforcement should be determined by the house designer, architect- or engineer-of-record.

The recommended allowable soil bearing pressure is 1,500 lbs/ft^2 for footings on stiff, native soil and engineered fill. A maximum chimney and column load of 40 kips is recommended for the site. For heavier loads, GeoPacific should be specifically consulted. The coefficient of friction between on-site soil and poured-in-place concrete may be taken as 0.42 (value does not include any factor of safety adjustment). The maximum anticipated total and differential footing movements (generally from soil expansion and/or settlement) are 1 inch and ¾ inch over a span of 20 feet, respectively. Excavations near structural footings should not extend within a 1H:1V plane projected downward from the bottom edge of footings.

Footing excavations should penetrate through surficial fill, topsoil and any loose soil to competent subgrade that is suitable for bearing support with a minimum embedment of 3 to 5 feet. All footing excavations should be trimmed neat, and all loose or softened soil should be removed from the excavation bottom prior to placing reinforcing steel bars. Foundations constructed during the wet weather season may require localized overexcavation of footings and backfill with compacted, crushed aggregate to retard softening of subgrade soils by surface water.

Below Grade 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 wall, an at-rest equivalent fluid pressure of 55 pcf should be used in design, again assuming level backfill against the wall. These values assume that the recommended drainage provisions are incorporated, hydrostatic pressures are not allowed to develop against the wall, and walls are backfilled with engineered fill.

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

We assume relatively level ground surface below the base of the walls. As such, we recommend passive earth pressure of 320 pcf for use in design, assuming wall footings are cast against competent native soils or engineered fill. If the ground surface slopes down and away from the base of any of the walls, a lower passive earth pressure should be used and GeoPacific should be contacted for additional recommendations.

A coefficient of friction of 0.45 may be assumed along the interface between the base of the wall footing and subgrade soils. The recommended coefficient of friction and passive earth pressure values do not include a safety factor, and an appropriate safety factor should be included in design. The upper 12 inches of soil should be neglected in passive pressure computations unless it is protected by pavement or slabs on grade.

The above recommendations for lateral earth pressures assume that the backfill behind the subsurface walls will consist of properly compacted structural fill, and no adjacent surcharge loading. If the walls will be subjected to the influence of surcharge loading within a horizontal distance equal to or less than the height of the wall, the walls should be designed for the additional horizontal pressure. For uniform surcharge pressures, a uniformly distributed lateral pressure of 0.3 times the surcharge pressure should be added. Traffic surcharges may be estimated using an additional vertical load of 250 psf (2 feet of additional fill), in accordance with local practice.

The recommended equivalent fluid densities assume a free-draining condition behind the walls so that hydrostatic pressures do not build-up. This can be accomplished by placing a 12- to 18-inch wide zone of sand and gravel containing less than 5 percent 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 suitable discharge point to remove water in this zone of sand and gravel. The drain pipe should be wrapped in filter fabric (Mirafi 140N or other as approved by the geotechnical engineer) to minimize clogging.

GeoPacific should be contacted during construction to verify subgrade strength in wall keyway excavations, to verify that backslope soils are in accordance with our assumptions, and to take density tests on the wall backfill materials.

Structures should be located a horizontal distance of at least 1.5H away from the back of the retaining wall, where H is the total height of the wall. GeoPacific should be contacted for additional foundation recommendations where structures are located closer than 1.5H to the top of any wall.

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 150 kcf (87 pci) should be assumed for the medium stiff native silt 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 Table 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. Under-slab aggregate should be compacted to at least 95% of its maximum dry density as determined by ASTM D698 (Standard Proctor) or equivalent.

In areas where moisture will be detrimental to floor coverings or equipment inside the proposed structure, appropriate vapor barrier and damp-proofing measures should be implemented. A commonly applied vapor barrier system consists of a 10-mil polyethylene vapor barrier placed directly over the capillary break material. 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 GeoPacific's area of expertise.

Seismic Design

The Oregon Department of Geology and Mineral Industries (DOGAMI), Oregon HazVu: 2021 Statewide GeoHazards Viewer indicates that the site is in an area where *severe* ground shaking is anticipated during an earthquake. Structures should be designed to resist earthquake loading in accordance with the methodology described in the 2015 International Building Code (IBC) with applicable Oregon Structural Specialty Code (OSSC) revisions (current 2014). We recommend Site Class C be used for design per the OSSC, Table 1613.5.2 and as defined in ASCE 7, Chapter 20, Table 20.3-1. Design values determined for the site using the Applied Technology Council (ATC) 2020 Hazards by Location Online Tool are summarized in Table 1, and are based upon existing soil conditions.

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

Parameter	Value
Location (Lat, Long), degrees	45.617, -122.842
Mapped Spectral Acceleration Values (MCE):	
Peak Ground Acceleration PGA_M	0.492
Short Period, S_s	0.913 g
1.0 Sec Period, S_1	0.423 g
Soil Factors for Site Class D:	
F_a	1.135
F_v	*1.877
Residential Site Value $SD_s = 2/3 \times F_a \times S_s$	0.691 g
$SD_1 = 2/3 \times F_v \times S_1$	*0.529 g
Residential Seismic Design Category	D

* F_v value reported in the above table is a straight-line interpolation of mapped spectral response acceleration at 1-second period, S_1 per Table 1613.2.3(2) of OSSC 2019 with the assumption that Exception 2 of ASCE 7-16 Chapter 11.4.8 is met per the Structural Engineer. If Exception 2 is not met, and the long-period site coefficient (F_v) is required for design, GeoPacific Engineering can be consulted to provide a site-specific procedure as per ASCE 7-16, Chapter 21.

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. According to the Oregon HazVu: Statewide Geohazards Viewer, the subject site is regionally characterized as not having a risk of soil liquefaction (DOGAMI:HazVu, 2021).

Footing and Roof Drains

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 expose ground in the crawlspace, and crawlspace ventilation (foundation vents). 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 consulting regarding crawlspace ventilation, building material selection and mold prevention issues, which are outside GeoPacific's area of expertise.

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

If the proposed structure will have a raised floor, and no concrete slab-on-grade floors in living spaces are used, perimeter footing drains would not be required based on soil conditions encountered at the site and experience with standard local construction practices. Where it is desired to reduce the potential for moist crawl spaces, footing drains may be installed. If concrete slab-on-grade floors are used, perimeter footing drains should be installed as recommended below.

Where necessary, perimeter footing drains should consist of 3 or 4-inch diameter, perforated plastic pipe embedded in a minimum of 1 ft^3 per lineal foot of clean, free-draining drain rock. The drain pipe and surrounding drain rock should be wrapped in non-woven geotextile (Mirafi 140N, or approved equivalent) to minimize the potential for clogging and/or ground loss due to piping. A minimum 0.5 percent fall should be maintained throughout the drain and non-perforated pipe outlet. In our opinion, footing drains may outlet at the curb, or on the back sides of lots where sufficient fall is not available to allow drainage to meet the street.

Additional recommendations regarding maintenance of site drainage conditions on hillside homesites is outlined in the attached “Maintenance of Hillside Homesites”.

UNCERTAINTY AND LIMITATIONS

We have prepared this report for the client, for use on this project only. The report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, the conclusions and interpretations presented in this report should not be construed as a warranty of the subsurface conditions. Experience has shown that soil and groundwater conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations that may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, GeoPacific should be notified for review of the recommendations of this report, and revision of such if necessary.

Within the limitations of scope, schedule and budget, GeoPacific attempted to execute these services in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology at the time the report was prepared. No warranty, express or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or groundwater at this site.

We appreciate this opportunity to be of service.

Sincerely,

GEOPACIFIC ENGINEERING, INC.

CERTIFIED OREGON ELIZABETH K. RAPP No. E2190 ENGINEERING GEOLOGIST
exp 2/1/22

REGISTERED PROFESSIONAL ENGINEER 14743PE OREGON JAN. 23, 1990 JAMES D. IMBRIE
EXPIRES: 06/30/2021

Beth K. Rapp, C.E.G.
Senior Engineering Geologist

Reviewed by: James D. Imbrie, G.E., C.E.G.
Principal Geotechnical Engineer

Attachments: Figure 1 – Vicinity Map
Figure 2 – Lidar Based Vicinity Map-With Mapped Landslides
Figure 3 – Site Grading Plan with Exploration Locations
Test Pit Logs (TP-1 through TP-2)
Maintenance of Hillside Homesites

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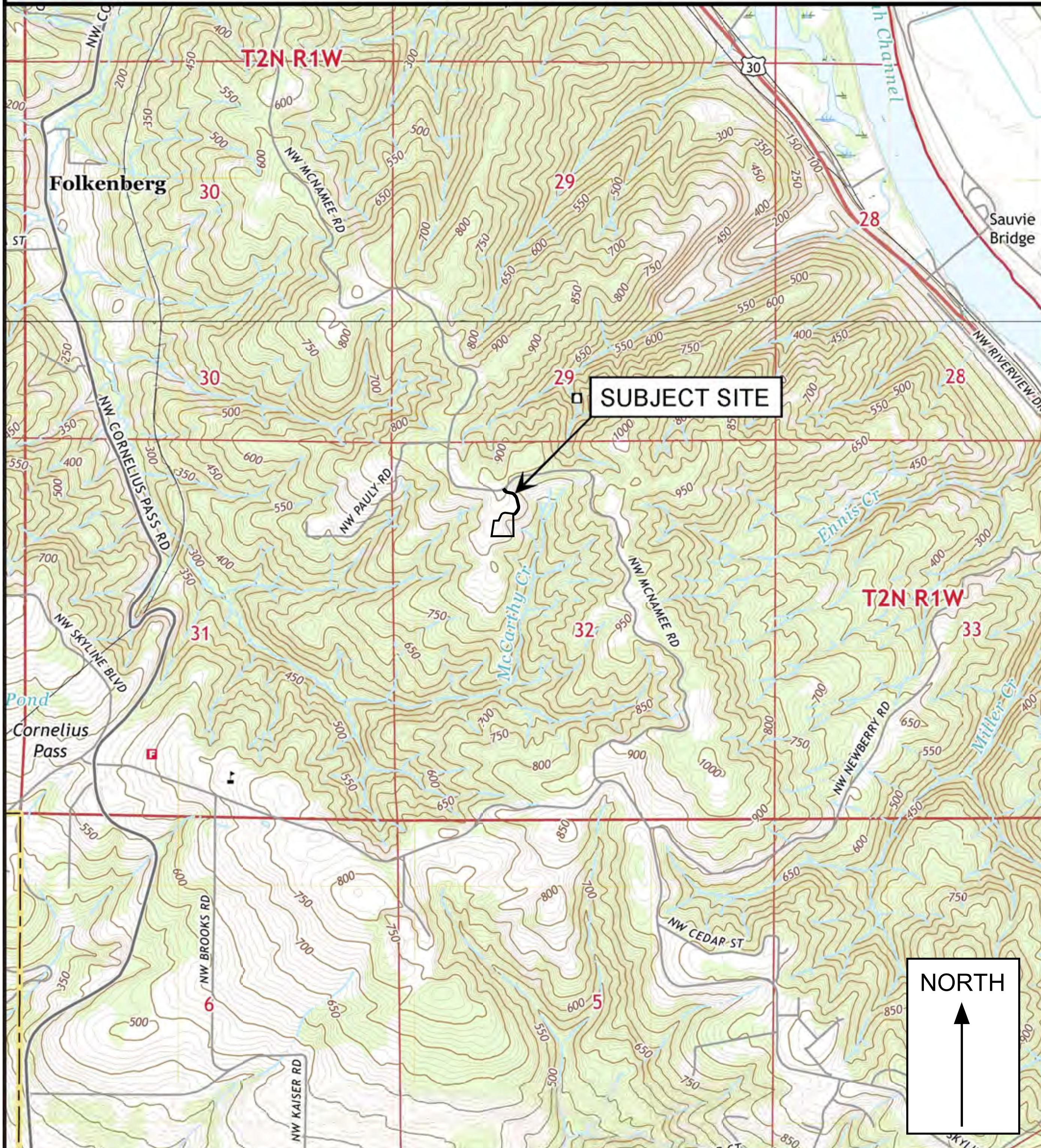
VICINITY MAP



Legend

Approximate Scale 1 in = 2,000 feet

Date: 4/5/2021
Drawn by: EKR

Base map: U.S. Geological Survey 7.5 minute Topographic Map Series, Linnton, Oregon Quadrangle, 2020 and Sauvie Island, Oregon Quadrangle, 2020.

Project: 13221 NW McNamee Road Homesite Multnomah County, Oregon	Project No. 20-5645	FIGURE 1

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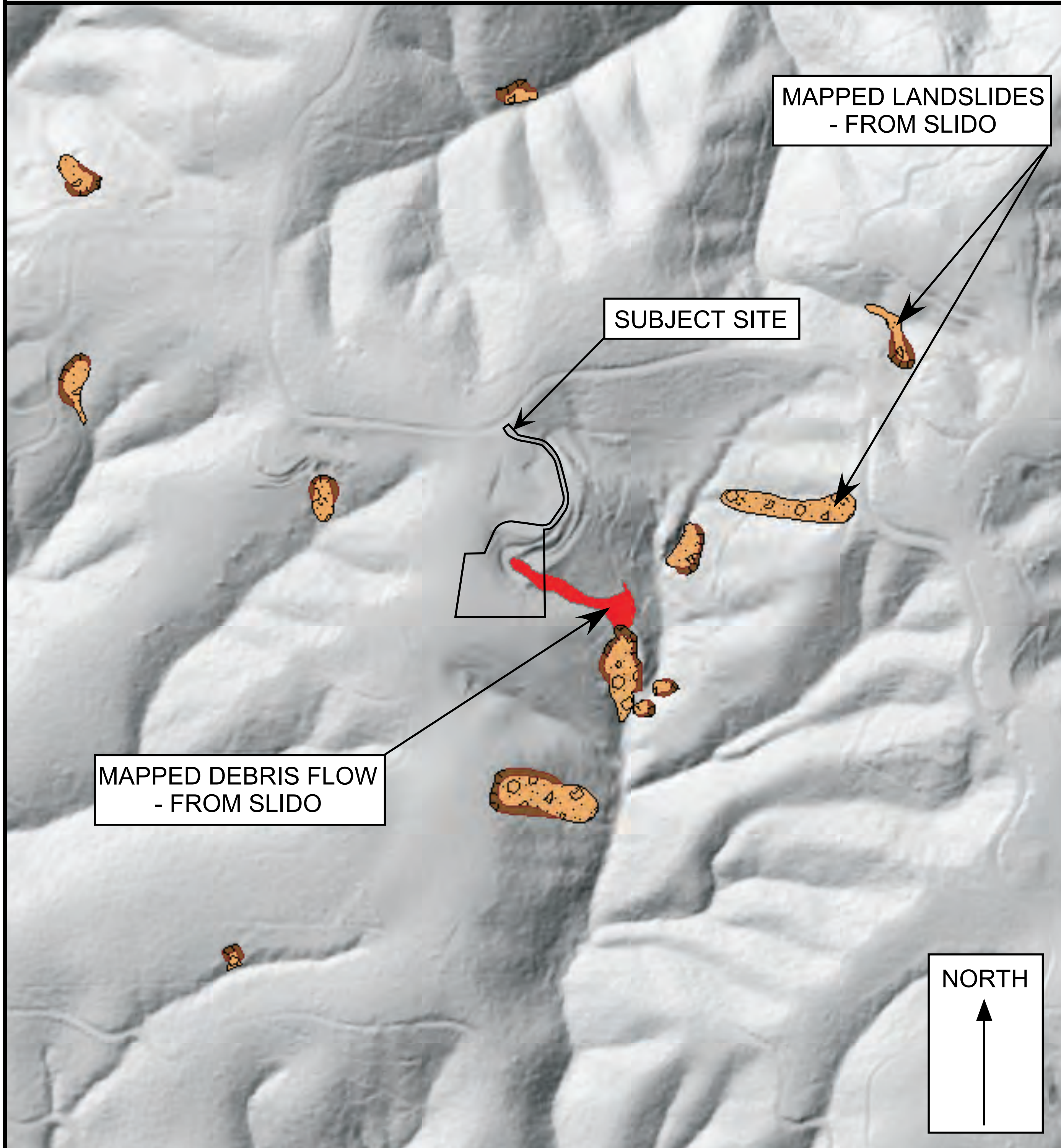
LIDAR BASED VICINITY MAP - WITH MAPPED LANDSLIDES



Legend

Approximate Scale 1 in = 500 ft

Date: 4/5/2021
Drawn by: EKR

Base map: Oregon Department of Geology and Mineral Industries, 2021, Statewide Landslide Information Database for Oregon (SLIDO): https://gis.dogami.oregon.gov/maps/slido/

Project: 13221 NW McNamee Road Homesite, Multnomah County, Oregon	Project No. 20-5645	FIGURE 2

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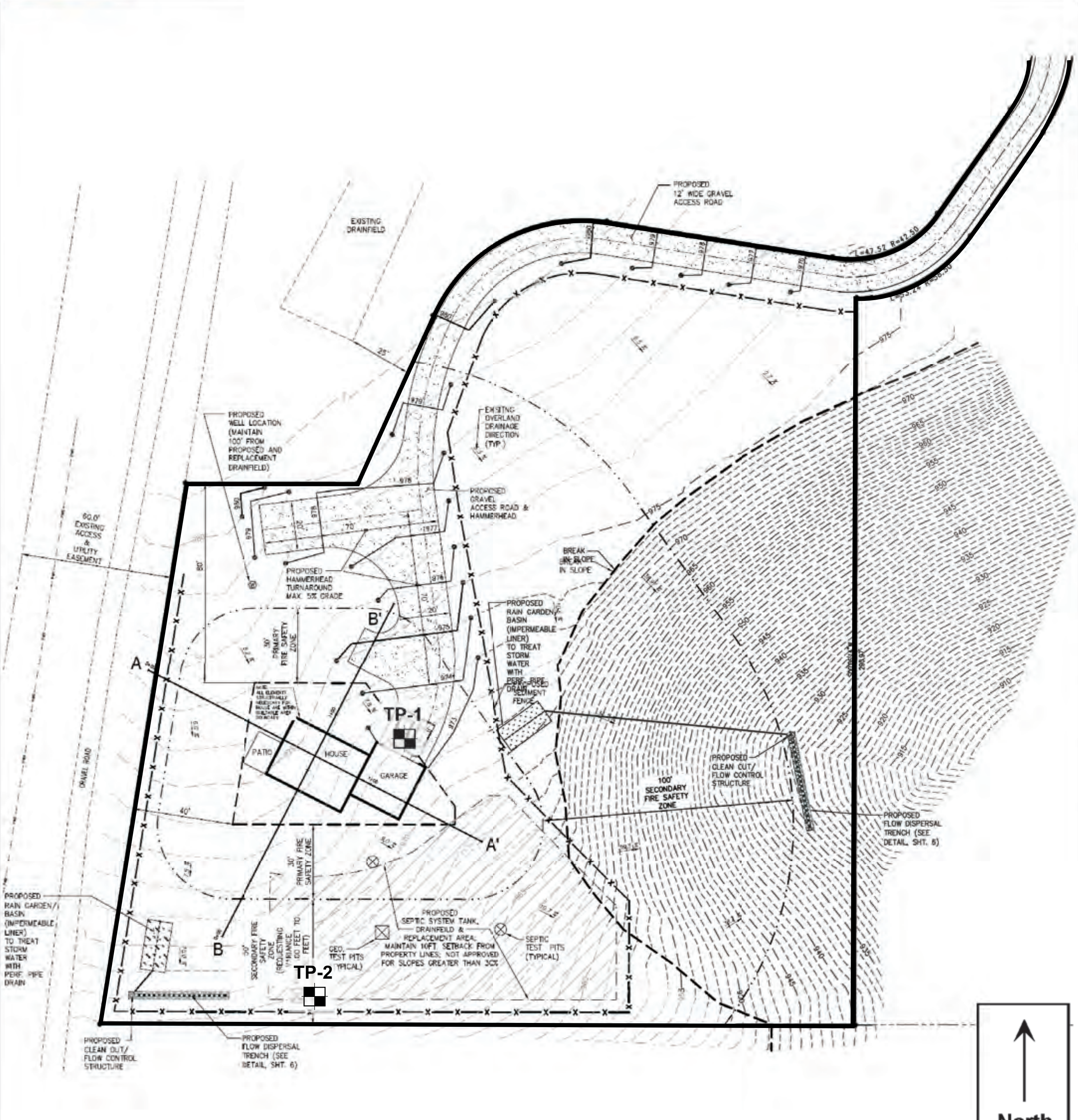
SITE PLAN AND EXPLORATION LOCATIONS



Base map provided by NW Engineers dated 3/4/2021

Legend

TP-1 Test Pit Designation and Approximate Location

0 60'

APPROXIMATE SCALE 1"=60'

Date: 4/5/2021
Drawn by: EKR

Project: 13221 NW McNamee Road Homesite, Multnomah County, Oregon	Project No. 20-5645	FIGURE 3

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TEST PIT LOG

Project: 13221 NW McNamee Road, Multnomah County, Oregon	Project No. 20-5645	Test Pit No. **TP-1**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
0–0.4						Organic SILT (OL), brown, root mat for grass, moist (Topsoil)
1	0.5					Soft to medium-stiff, clayey SILT (ML), mixed brown, light brown, orange and gray, common fine roots, moist (Disturbed Native Soil - Tilled Zone)
2	4.0					
3	4.0					Very-stiff, clayey SILT (ML) to silty CLAY (CL), mottled light brown, orange and gray, moderate plasticity, damp to moist (Quaternary Loess Deposit)
4	3.5					
5						
6						
7						
8						Test Pit Terminated at 8 feet

Note: No seepage or groundwater encountered.

LEGEND

100 to 1,000 g — Bag Sample
5 Gal. Bucket — Bucket Sample
Shelby Tube Sample
Seepage
Water Bearing Zone
Water Level at Abandonment

Date Excavated: 2/10/2010
Logged By: P. Crenna
Surface Elevation:

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TEST PIT LOG

Project: 13221 NW McNamee Road, Multnomah County, Oregon	Project No. 20-5645	Test Pit No. **TP-2**

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Type	In-Situ Dry Density (lb/ft³)	Moisture Content (%)	Water Bearing Zone	Material Description
0–0.8						Organic SILT (OL), dark brown, root mat for grass, moist (Topsoil)
1	2.0					Stiff, clayey SILT (ML), light brown, common fine roots, moist (Native Soil Horizon)
2	2.0					
3	4.0					Very-stiff, clayey SILT (ML) to silty CLAY (CL), mottled light brown, orange and gray, low to moderate plasticity, damp to moist (Quaternary Loess Deposit)
4	4.0					
5	4.0					
6						
7						
8						
9						Test Pit Terminated at 8.5 feet
10						Note: No seepage or groundwater encountered.
11						
12						
13						
14						
15						
16						
17						

LEGEND

100 to 1,000 g — Bag Sample
5 Gal. Bucket — Bucket Sample
Shelby Tube Sample
Seepage
Water Bearing Zone
Water Level at Abandonment

Date Excavated: 2/10/2010
Logged By: P. Crenna
Surface Elevation:

MAINTENANCE OF HILLSIDE HOMESITES

All homes require a certain level of maintenance for general upkeep and to preserve the overall integrity of structures and land. Hillside homesites require some additional maintenance because they are subject to natural slope processes, such as runoff, erosion, shallow soil sloughing, soil creep, perched groundwater, etc. If not properly controlled, these processes could adversely affect your or neighboring properties. Although surface processes are usually only capable of causing minor damage, if left unattended, they could possibly lead to more serious instability problems.

The primary source of problems on hillsides is uncontrolled surface water runoff and blocked groundwater seepage which can erode, saturate and weaken soil. Therefore, it is important that drainage and erosion control features be implemented on the property, and that these features be maintained in operative condition (unless changed on the basis of qualified professional advice). By employing simple precautions, you can help properly maintain your hillside site and avoid most potential problems. The following is an abbreviated list of common Do's and Don'ts recommended for maintaining hillside homesites.

Do List

1. Make sure that roof rain drains are connected to the street, local storm drain system, or transported via enclosed conduits or lined ditches to suitable discharge points away from structures and improvements. In no case, should rain drain water be discharged onto slopes or in an uncontrolled manner. Energy dissipation devices should be employed at discharge points to help prevent erosion.
2. Check your roof drains, gutters and spouts to make sure that they are clear. Roofs are capable of producing a substantial flow of water. Blocked gutters, etc., can cause water to pond or run off in such a way that erosion or adverse oversaturation of soil can occur.
3. Make sure that drainage ditches and/or berms are kept clear throughout the rainy season. If you notice that a neighbor's ditches are blocked such that water is directed onto your property or in an uncontrolled manner, politely inform them of this condition.
4. Locate and check all drain inlets, outlets and weep holes from foundation footings, retaining walls, driveways, etc. on a regular basis. Clean out any of these that have become clogged with debris.
5. Watch for wet spots on the property. These may be caused by natural seepage or indicate a broken or leaking water or sewer line. In either event, professional advice regarding the problem should be obtained followed by corrective action, if necessary.
6. Do maintain the ground surface adjacent to lined ditches so that surface water is collected in the ditch. Water should not be allowed to collect behind or flow under the lining.

Don't List

1. Do not change the grading or drainage ditches on the property without professional advice. You could adversely alter the drainage pattern across the site and cause erosion or soil movement.
2. Do not allow water to pond on the property. Such water will seep into the ground causing unwanted saturation of soil.
3. Do not allow water to flow onto slopes in an uncontrolled manner. Once erosion or oversaturation occurs, damage can result quickly or without warning.
4. Do not let water pond against foundations, retaining walls or basements. Such walls are typically designed for fully-drained conditions.
5. Do not connect roof drainage to subsurface disposal systems unless approved by a geotechnical engineer.
6. Do not irrigate in an unreasonable or excessive manner. Regularly check irrigation systems for leaks. Drip systems are preferred on hillsides.

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