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Updated Geotechnical Report
Mahogany Ridge Subdivision
South of Parcel at 944 East Main Street
John Day, Oregon

CGT Project Number G2005305

Prepared for

Joshua T. Walker
Mahogany Ridge Properties
601 S Canyon Boulevard
John Day, Oregon 97845

July 7, 2020

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John Day, Oregon

CGT Project Number G2005305

Dear Mr. Walker:

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this updated geotechnical report for the proposed Mahogany Ridge Subdivision project. The site is located south of the parcel located at 944 East Main Street in John Day, Oregon. We performed our work in general accordance with CGT Proposal GP8962, dated June 4, 2020. Written authorization for our services was received on June 23, 2020.

We appreciate the opportunity to work with you on this project. Please contact us at 503.601.8250 if you have any questions regarding this report.

Respectfully Submitted,
CARLSON GEOTECHNICAL

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1.0 INTRODUCTION

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this updated geotechnical report for the proposed Mahogany Ridge Subdivision project. The site is located south of the parcel at 944 East Main Street in John Day, Oregon, as shown on the attached Site Location, Figure 1.

CGT previously performed a geotechnical investigation for the project site and nearby land, the results of which were presented in our August 2, 2006, "Report of Geologic Hazard Assessment & Preliminary Geotechnical Investigation, Strawberry View 80-Acre Subdivision, East of John Day" CGT Project Number G0602826. The site has subsequently been renamed as the "Mahogany Ridge Subdivision," which comprises the eastern approximate 18.2 acres of the area addressed by the 2006 report. The relative project areas addressed by the 2006 report and the current report are shown on the Site Location, attached as Figure 1. Logs for our test pits excavated on the site in 2006 are presented in Appendix A, attached at the end of this report.

1.1 Project Information

Based on the most recent development plans for the site, prepared by Sisul Engineering, dated May 2020, we understand the project will include:

- Partitioning the approximate 18.2-acre parcel into 11 residential lots and four "open space" lots.
- Construction of new residential structures on each of the new lots. The residential structures will include stand-alone single-family residences, duplexes, triplexes, and a quad-plex. Although no architectural plans have been provided, we anticipate new residential structures will be up to 3 stories tall, wood-framed, and will incorporate slab-on-grade floors or post and beam floor construction (crawlspaces). We anticipate some of the structures will incorporate daylight basement levels with retaining walls up to about 8 feet in height. For the purposes of this report, we have assumed that building loads will be typical of these types of structures, with continuous perimeter footing loads of less than 3 kips per lineal foot (klf), interior column loads of less than 30 kips, and floor slab loads less than 100 psf.
- Construction of new roadways to provide access to the new lots. We assume new pavements will be surfaced with asphalt concrete (AC) with localized Portland cement concrete (PCC) aprons.
- Installation of underground utilities to serve the new residences.
- The plans indicate stormwater runoff from new impervious areas of the site will be collected and diverted into the roadside ditch along Highway 26 (i.e. on-site stormwater infiltration facilities are not being pursued at this time).
- Although no grading plans have been provided, we understand permanent grade changes at the site will typically include cuts and fills up to about 5 feet relative to existing grades. One area of deeper fill may be required where the roadway crosses the existing drainage at the west central portion of the site. We anticipate this area will require the placement of up to 10 feet of structural fill to reach finished grades.

1.2 Project Approach

Based on review of the plans described above, the project is generally consistent with that understood in the referenced 2006 report, except the proposed development area has been reduced as shown on the Site Plan, attached as Figure 2. We understand the City of John Day requires an updated report be prepared to address current building code [2017 Oregon Residential Specialty Code (ORSC)].

Based on our recent site visit, described in Section 2.2, it was evident that no significant grading has been performed at the site since our subsurface investigation in 2006. Based on review of historical aerial imagery provided by online media, clearing of vegetation on the north end of the site and minor grading for new access roads first appears in 2011, as described in further detail in Section 2.2 below.

Due to the age of the referenced report, this current report is presented as a complete, stand-alone geotechnical investigation report. The recommendations contained in this report supersede those presented in the above referenced 2006 report.

1.3 Scope of Services

Our scope of work included the following:

- Visit the site to confirm site conditions are consistent with those observed during our previous (2006) field investigation.
- Provide a technical narrative describing surface and subsurface deposits, and local geology of the site, based on the results of our explorations and published geologic mapping.
- Provide recommendations for the Seismic Site Class, mapped maximum considered earthquake spectral response accelerations, and site seismic coefficients.
- Provide a qualitative evaluation of seismic hazards at the site, including earthquake-induced liquefaction, landsliding, and surface rupture due to faulting or lateral spread.
- Provide geotechnical recommendations for site preparation and earthwork.
- Provide geotechnical engineering recommendations for use in design and construction of shallow foundations, floor slabs, retaining walls, and AC pavements.
- Provide this written report summarizing the results of our geotechnical investigation and recommendations for the project.

2.0 SITE DESCRIPTION

2.1 Site Geology

The geologic map for the area[1] indicates that the site is underlain by Miocene Rattlesnake Formation sediments and tuffs. The sedimentary rocks typically consist of a semi-consolidated clay, sand, and gravel conglomerate (Tr). The tuff member of the Rattlesnake Formation (Trt) generally consists of rhyolite tuff, which ranges from densely welded near the upper portions of the unit, to poorly welded sections near the base of the unit. The Rattlesnake Formation tuff has a thickness of up to 100 feet in the John Day area, and makes up the “rim rock” along the tops of the cliffs in the area. Some areas of Columbia River Basalt have also been mapped in the immediate vicinity of the site.

The geologic map shows areas of landslide deposits across the northern end of the site. This map unit includes ancient landslides, active landslides, and surficial failures. The report accompanying the geologic map indicates that hillside slopes in landslide terrain should be considered potentially unstable, and may be unsuitable for development in areas. The report indicates that the softer areas of the Rattlesnake Formation sediments are especially vulnerable to failure where overlain by the welded tuff member of the Rattlesnake Formation.

---

[1] Schlicker, Herbert G., and Brooks, Howard. Engineering Geology of the John Day Area, Grant County, Oregon, 1975. Oregon Department of Geology and Mineral Industries.

During preparation of our 2006 report, John Day Land Development, LLC, (our original client) indicated that the Oregon Department of Transportation (ODOT) experienced a landslide on their property located several hundred feet east of the northeast corner of the site. This slide was reportedly activated by excavation at the base of the slope. The depth of the slide plain and the date of the slide is not known by CGT. The offsite slide was reportedly stabilized using a buttress fill, and has not reportedly experienced any additional movement.

2.2 Site Surface Conditions

As shown on the Site Plan, attached as Figure 2, and the Aerial Photograph, attached as Figure 3, the northern property boundary was typically located approximately 150 to 200 feet south of Highway 26. The area north of the northern site boundary and south of Highway 26 was occupied by single-family residences and one church. The property abutted Highway 26 for approximately 75 feet in the extreme northeast corner of the site, which will provide the primary access to the subdivision. The properties located east, west, and south of the site were undeveloped large parcels used primarily as rangeland.

Work conducted prior to 2006 included grading associated with an access road along the northern property boundary, as indicated on the Aerial Photograph, attached as Figure 3. Typical site gradients ranged from approximately 2H:1V to 4H:1V along the majority of the site. Elevations across the site ranged from 3,130 feet above Mean Sea Level (MSL) at the northeast corner of the site along Highway 26 to approximately 3,380 feet MSL near the southwest corner of the site. Site topography is shown on the Site Plan, attached as Figure 2.

The site was located on a north-facing slope dissected by two roughly parallel, north-trending drainages, which may run along ancient fault lines associated with the John Day Fault Zone. The largest of these drainages was located near the center of the site, and had a small stream flowing at the time of the 2006 investigation. A culvert had been installed under the graded roadbed along the northern property boundary.

Localized areas of active surficial instability and potential landslides were noted throughout the site in 2006. No large-scale slides or headscarps were observed were identified during the subsurface exploration in 2006.

CGT reviewed aerial photographs of the site to determine if any significant grading or vegetation removal has occurred since our 2006 investigation. Based on the aerial photographs, we determined part of the northern portion of the site had been cleared of vegetation and new access roads were graded sometime between 2008 and 2011. In addition, it appears one of the access roads lead to a borrow pit in the north-central portion of the site. A possible stockpile can also be seen in some photographs near the north edge of the site. These features are indicated on the Aerial Photograph, attached as Figure 3.

Subsequent to 2011, changes between aerial photographs appear to be limited to vegetation growth and occasional vehicles on the access roads.

CGT geological staff visited the northern portion of the site in June 2020 to observe existing site conditions. Based on our observations, grading associated with the 2011 access roads was limited to less than a few feet of cut and fill. The amount of the material removed from the borrow pit was not readily apparent, as the area was now covered with grasses and small trees (Figure 4, Photograph 4). No signs of erosion or instability were noted on the northern portion of the site during our June 2020 site visit.

2.3 Subsurface Conditions

2.3.1 Subsurface Investigation & Laboratory Testing

CGT witnessed the excavation of seventeen test pits (TP-1 through TP-17) at the site on June 16, 2006, to depths of up to 14 feet below ground surface (bgs) using a Cat 330B excavator provided and operated by Winegar Excavating. Eight of these test pits (TP-1, TP-2, TP-3, TP-7, TP-8, TP-9, TP-10, and TP-17) were located within or reasonably close to the current limits of construction, as shown on the Site Plan, attached as Figure 2. In summary, the test pits within the current limits of construction were excavated to depths ranging from about 6 to 14 feet bgs. Details regarding the subsurface investigation, logs of the explorations, and results of laboratory testing are presented in Appendix A. Subsurface conditions encountered during our investigation are summarized below.

2.3.2 Subsurface Materials

Logs of the explorations are presented in Appendix A. The following describes each of the subsurface materials encountered at the site.

2.3.2.1 Topsoil

Encountered at the surface of all test pits was silt topsoil. This material was typically very soft, damp, tan, and slightly organic. The topsoil extended to depths of 1 to 2 feet bgs in our test pits.

2.3.2.2 Rattlesnake Formation Sediments

Underlying the silt topsoil in the test pits was silt, sand, and gravel consistent with Rattlesnake Formation sediments mapped in the vicinity of the site and described in Section 2.1 above. Overall, the grain size of the Rattlesnake Formation sedimentary deposits varied from clay to boulder size, and varied from virtually unconsolidated alluvium to well consolidated conglomerate. The Rattlesnake Formation sediments were deposited in an alluvial fan environment on the slopes of the ancient John Day River valley. The scope of this report did not include sufficient test pits to map the lateral extent of these layers. The sedimentary layers are anticipated to extend laterally for some distance before transitioning to a different sedimentary facies[2].

The sediments encountered at the site consisted of interbedded silty sand, silt, and silty gravel to clayey gravel. The following sections provide general descriptions of the interbedded soils.

2.3.2.2.1 Silty Sand (SM)

This material was typically loose, damp, and tan.

2.3.2.2.2 Silt (ML)

This material was typically medium stiff to very stiff, damp, tan, and contained a variable amount of sand, gravel, cobbles, and boulders.

2.3.2.2.3 Silty Gravel (GM) to Clayey Gravel (GC)

This material ranged from loose to dense, damp, brown to tan to reddish brown, contained fragments of white tuff and occasional boulders up to 2 feet in diameter.

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[2] **facies** [stratigraphy] - A distinctive group of characteristics that distinguish one group from another within a stratigraphic unit; the sum of all primary lithologic and paleontological characteristics of sediments or sedimentary rock that are used to infer its origin and environment; the general nature of appearance of sediments or sedimentary rock produced under a given set of conditions; e.g.: contrasting river-channel facies and overbank-flood-plain facies in alluvial valley fills.

2.3.3 Groundwater

Groundwater was not encountered during our 2006 investigation of the project site. CGT conducted a review of water well logs published by the Oregon Department of Water Resources[3] for wells located within about 1 mile of the site. We found that the church located near the northeast corner of the site had a well that encountered static groundwater at approximately 100 feet bgs. It should be noted that groundwater levels are relative to the ground surface and, due to local topography, the levels reported on the logs are considered generally indicative of local water levels and may not reflect actual groundwater levels at the site. We anticipate that the water levels in the John Day area are highly variable and are largely controlled by the sediments and rock formations in the area. In addition, we anticipate that groundwater levels will fluctuate due to seasonal and annual variations in precipitation, changes in site utilization, or other factors. The on-site fine-grained sediments are conducive to low infiltration rates and the formation of perched groundwater tables. We anticipate that cuts made into the drainages will encounter localized perched groundwater.

3.0 GEOLOGIC HAZARD UPDATE

3.1 Additional Geologic Hazard Mapping

Since preparation of our 2006 report, the Oregon Department of Geology and Mineral Industries (DOGAMI) issued additional geologic hazard maps covering the site. CGT reviewed the following maps during preparation of this update report:

3.1.1 Landsliding

The Statewide Landslide Information Database for Oregon (SLIDO)[4] show the landslide deposits/landslide topography on the northern portion of the site, which was also described on the 1975 geologic map of the area described in Section 2.1 above. SLIDO does not provide significant detail regarding the landslide. No historic (recent) reactivations of the slide are shown on the mapping. Review of Lidar- (LIght Detection And Ranging) based imagery available on SLIDO shows the landslide topography as well. The Lidar imagery shows the landslide topography as incised by streams and the features have been “softened” through gradual erosion, which is indicative of very old landslides in the area.

DOGAMI developed a statewide landslide susceptibility map[5] using the Lidar data, USGS topography, SLIDO historical landslide information, and the state geologic map. The landslide susceptibility hazard mapping available via the DOGAMI Oregon Statewide Geohazards Viewer[6] (HAZVU) indicates a “moderate” (landsliding possible) to “very high” (existing landslide deposits) for the site and surrounding properties based mainly on their relative slope gradients. The “very high” rating is due to the presence of a mapped, large-scale, prehistoric landslide discussed above. No obvious signs of recent, large-scale slope instability were noted during our field observations in 2006 and 2020. Based on the geology of the site, the results of our 2006 field exploration, and the lack of reactivations of the ancient landslide, it is our opinion that localized, steep portions of the site present a moderate risk of localized landsliding. These slopes are located above

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[3] ORWD, 2020. Water well logs obtained from the Oregon Water Resources Department web site, http://www.wrd.state.or.us/

[4] Oregon Department of Geology and Mineral Industries, 2020. Statewide Landslide Information Database for Oregon (SLIDO), *accessed July 2020*, from DOGAMI web site: https://gis.dogami.oregon.gov/maps/slido/.

[5] Burns, William J, Mickelson, Katherine A., and Madin, Ian P, 2020. Landslide susceptibility overview map of Oregon. Oregon Department of Geology and Mineral Industries, Open-File Report O-16-02. Available on Oregon Statewide Geohazards Viewer, *accessed July 2020*, from DOGAMI web site: https://www.oregongeology.org/hazvu/.

[6] Oregon Department of Geology and Mineral Industries, 2020. Oregon Statewide Geohazards Viewer, *accessed July 2020*, from DOGAMI web site: https://www.oregongeology.org/hazvu/.

the majority of the City of John Day, so the risk of landsliding impacting the site is similar to surrounding sites.

It should be noted that any construction within hillside areas inherently bears greater risk of slope instability. This risk increases in seismically active areas or areas of previous landslide activity. The owner, not CGT, must recognize and accept the risk of potential slope instability from causes beyond their control or as yet unrecognized.

3.1.2 Seismic Hazards

Additional mapping associated with seismic hazards was also reviewed, and is summarized in Section 4.2 of this report.

3.2 Geologic Hazards Discussion

As indicated in Section 2.2 above, the surface conditions at the site in June 2020 were similar to that described in our 2006 report. Minor vegetation removal and grading had been performed sometime prior to 2011. No additional signs of slope instability or erosion were noted during our recent site reconnaissance.

Based on our review of the site plan, recent observation of site surface conditions, and our review of the relatively recent geologic hazard publications, we are of the opinion that the investigation findings presented in our 2006 report remain applicable for the finalized project. We conclude the site is geologically suitable for the proposed development as described in Section 1.1. We anticipate that with proper construction, grading, and stormwater management, the geology and topography of the site and the surrounding area will not adversely affect the proposed project and the project will have a minimum geologic impact on adjacent properties.

4.0 SEISMIC CONSIDERATIONS

4.1 Seismic Design

The 2017 Oregon Residential Specialty Code (2017 ORSC) requires the determination of seismic site class be determined in accordance with Chapter 20 of the American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures (ASCE 7-16). We have assigned the site as Site Class D ("Stiff Soil") based on geologic mapping and subsurface conditions encountered during our 2006 investigation.

Seismic ground motion values were determined in accordance with Section R301.2.2 of the 2017 ORSC using the Seismic Hazards by Location calculator on the ATC website[7]. The Seismic Design Category was determined from Table R301.2.2.1.1 of the 2017 ORSC. The site Latitude 44.412493° North and Longitude 118.932468° West were input as the site location. The following table shows the recommended seismic design parameters for the site.

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[7] Applied Technology Council (ATC), 2020. USGS seismic design parameters determined using "Seismic Hazards by Location," *accessed July 2020*, from the ATC website https://hazards.atcouncil.org/.

Table 1 Seismic Ground Motion Values

Parameter		Value
Mapped Acceleration Parameters	Spectral Acceleration, 0.2 second (S_s)	0.311g
Coefficients (Site Class D)	Site Coefficient, 0.2 second (F_A)	1.551
Adjusted MCE Spectral Response Parameters	MCE Spectral Acceleration, 0.2 second (S_{MS})	0.482g
Design Spectral Response Accelerations	Design Spectral Acceleration, 0.2 second (S_{DS})	0.321g
Seismic Design Category (Risk Category II)		B

4.2 Seismic Hazards

4.2.1 Liquefaction

In general, liquefaction occurs when deposits of loose/soft, saturated, cohesionless soils, generally sands and silts, are subjected to strong earthquake shaking. If these deposits cannot drain quickly enough, pore water pressures can increase, approaching the value of the overburden pressure. The shear strength of a cohesionless soil is directly proportional to the effective stress, which is equal to the difference between the overburden pressure and the pore water pressure. When the pore water pressure increases to the value of the overburden pressure, the shear strength of the soil approaches zero, and the soil can liquefy. The liquefied soils can undergo rapid consolidation or, if unconfined, can flow as a liquid. Structures supported by the liquefied soils can experience rapid, excessive settlement, shearing, or even catastrophic failure.

For fine-grained soils, susceptibility to liquefaction is evaluated based on penetration resistance and plasticity, among other characteristics. Criteria for identifying non-liquefiable, fine-grained soils are constantly evolving. Current practice to identify non-liquefiable, fine-grained soils is based on moisture content and plasticity characteristics of the soils[8,9,10]. The susceptibility of sands, gravels, and sand-gravel mixtures to liquefaction is typically assessed based on penetration resistance, as measured using SPTs, CPTs, or Becker Hammer Penetration tests (BPTs).

The Oregon Department of Geology and Mineral Industries' Oregon Statewide Geohazards Viewer (HazVu)[11] shows a moderate hazard for liquefaction at the site. This is based on the northern portion of the site being mapped as ancient landslide deposits, which are automatically considered by Hazvu to be potentially liquefiable, an inherent limitation with the State's broad mapping system.

Based on the anticipated depth of groundwater below the site (about 100 feet bgs), the relative density/consistency of the materials encountered in our test pits, and the anticipated depth to hard basalt bedrock under a portion of the site, the on-site soils are considered non-liquefiable. Accordingly, the risk of liquefaction occurring at this site is anticipated to be very low.

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[8] Seed, R.B. et al., 2003. Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework. Earthquake Engineering Research Center Report No. EERC 2003-06.

[9] Bray, Jonathan D., Sancio, Rodolfo B., et al., 2006. Liquefaction Susceptibility of Fine-Grained Soils, Journal of Geotechnical and Geoenvironmental Engineering, Volume 132, Issue 9, September 2006.

[10] Idriss, I.M., Boulanger, R.W., 2008. Soil Liquefaction During Earthquakes, Earthquakes Engineering Research Institute Monograph MNO-12.

[11] Oregon Department of Geology and Mineral Industries, 2020. Oregon Statewide Geohazards Viewer, *accessed July 2020*, from DOGAMI web site: https://www.oregongeology.org/hazvu/.

4.2.2 Slope Instability

As discussed in Section 3.1 above, the SLIDO, available at the DOGAMI website[12], shows the northern portion of the site is underlain by ancient landslide deposits. No historic landslides are located at or in the immediate vicinity of the site.

The site has relatively low seismic coefficients, but contains localized steep slopes. Based on the geology of the site, the absence of groundwater in the test pits, and proposed minimal changes in site grades, the risk of localized slope instability *due to seismic forces* at the site is considered moderate. If the property owner wishes to further define the risk of slope instability at the site, a quantitative slope stability analysis could be performed. Such an analysis would require borings using powered drilling equipment, and is outside the scope of this assignment.

4.2.3 Surface Rupture

4.2.3.1 Faulting

Although the site is situated in a region of the country with known active faults and historic seismic activity, no known faults exist on or immediately adjacent to the site. Therefore, the risk of surface rupture at the site due to faulting is considered low.

4.2.3.2 Lateral Spread

Surface rupture due to lateral spread can occur on sites underlain by liquefiable soils that are located on or immediately adjacent to slopes steeper than about 3 degrees (20H:1V), and/or adjacent to a free face, such as a stream bank or the shore of an open body of water. During lateral spread, the materials overlying the liquefied soils are subject to lateral movement downslope or toward the free face. Based on the non-liquefiable nature of the soils at the site, the risk of damage associated with lateral spread is negligible.

5.0 CONCLUSIONS

Based on the results of our field explorations and analyses, the site may be developed as described in Section 1.1, provided the recommendations presented in this report are incorporated into the design and development. Satisfactory subgrade support for planned shallow foundations, floor slabs, and pavements can be achieved by the native, inorganic, medium dense/medium stiff to better sediments (SM, ML, GM, GC), or structural fill that is properly placed and compacted on these materials during construction.

The principal geotechnical concern for this project is the need to limit over-steepening slopes during and after construction. Permanent grade changes in sloping areas of the site should be limited to the extent possible. Addition of water to the site through excessive irrigation, infiltration of stormwater from new impervious areas, or infiltration of sanitary discharge is not recommended, as these activities inherently increase the potential for instability of the slopes. All stormwater runoff and sewage should be collected and diverted to suitable discharge location(s) approved by the local jurisdiction.

We anticipate that up to 10 feet of structural fill may be necessary to bring the roadway at the western edge of the proposed development where it crosses the existing drainage. Water drainage should be maintained through structural fills using culverts and drains built into the embankments in substantial conformance with the approved civil plans. Where fills are required to achieve desired finished site grades, the native slopes

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[12] Oregon Department of Geology and Mineral Industries, 2020. Statewide Landslide Information Database for Oregon (SLIDO), *accessed July 2020*, from DOGAMI web site: https://gis.dogami.oregon.gov/maps/slido/.

should be keyed and benched in accordance with the recommendations presented in Section 6.5 below, and fill materials should be compacted with the recommendations presented in accordance with Section 6.4 below.

6.0 RECOMMENDATIONS

The recommendations presented in this report are based on the information provided to us, results of our 2006 field investigation, 2020 site observations, analyses, laboratory data, and professional judgment. CGT has observed only a small portion of the pertinent subsurface conditions. The recommendations are based on the assumptions that the subsurface conditions do not deviate appreciably from those found during the field investigation. CGT should be consulted for further recommendations if the design of the proposed development changes and/or variations or undesirable geotechnical conditions are encountered during site development.

6.1 Site Preparation

6.1.1 Stripping

Existing vegetation, rooted soils, topsoil (OL), and soft/loose native soils should be removed from within, and for a minimum 5-foot margin around, proposed structural fill, building pad, and pavement areas. Based on the results of our field explorations, topsoil stripping depths are anticipated to be less than 2 feet bgs. Based on the results of our field explorations, localized soft soils may be present to depths up to about 3 feet below existing grades. These materials may be deeper or shallower at locations away from the completed explorations. The geotechnical engineer's representative should provide recommendations for actual stripping depths based on observations during site stripping. Stripped surface vegetation and rooted soils should be transported off-site for disposal, or stockpiled for later use in landscaped areas. Stripped, inorganic fill materials should be transported off-site for disposal, or may be stockpiled for later use as structural fill as described in Section 6.4.1 of this report.

6.1.2 Grubbing

Grubbing of trees should include the removal of the root mass and roots greater than ½-inch in diameter. Grubbed materials should be transported off-site for disposal. Root masses from larger trees may extend greater than 3 feet bgs. Where root masses are removed, the resulting excavation should be properly backfilled with structural fill in conformance with Section 6.4.2 of this report.

6.1.3 Test Pit Backfills

The test pits conducted at the site were loosely backfilled during our 2006 field investigation. Where test pits are located within finalized building, structural fill, or pavement areas, the loose backfill materials should be re-excavated. The resulting excavations should be backfilled with structural fill in conformance with Section 6.4 of this report.

6.1.4 Existing Utilities & Below-Grade Structures

All existing utilities at the site should be identified prior to excavation. Abandoned utility lines beneath the new buildings, pavements, and hardscaping features should be completely removed or grouted full. Soft, loose, or otherwise unsuitable soils encountered in utility trench excavations should be removed and replaced with structural fill in conformance with Section 6.4 this report. Buried structures (i.e. footings, foundation walls, retaining walls, slabs-on-grade, tanks, etc.), if encountered during site development, should be completely removed and replaced with structural fill in conformance with Section 6.4 of this report.

6.1.5 Subgrade Preparation – Pavements & Areas to Receive Structural Fill

After site preparation as recommended above, but prior to placement of structural fill and/or aggregate base, the geotechnical engineer or their representative should observe the exposed subgrade soils in order to identify areas of excessive yielding through either proof rolling or probing. Proof rolling of subgrade soils is typically conducted during dry weather using a fully-loaded, 10- to 12-cubic-yard, tandem-axle, tire-mounted, dump truck or equivalent weighted water truck. Areas of limited access or that appear too soft or wet to support proof rolling equipment should be evaluated by probing. During wet weather, subgrade preparation should be performed in general accordance with the recommendations presented in Section 6.3 of this report. If areas of soft soil or excessive yielding are identified, the affected material should be over-excavated to firm, unyielding subgrade, and replaced with imported granular structural fill in conformance with Section 6.4.2 of this report.

Preparation of subgrade soils during wet weather should be in conformance with Section 6.3 of this report. As indicated therein, increased base rock sections and a geotextile separation fabric may be required in wet conditions in order to support construction traffic and protect the subgrade.

6.1.6 Freezing Weather Considerations

For construction that occurs during extended periods of sub-freezing temperatures, the following special provisions are recommended:

- Structural fill should not be placed over frozen ground.
- Frozen soil should not be placed as structural fill.
- Fine-grained (silty, clayey) soils should not be placed as structural fill during sub-freezing temperatures.

Identification of frozen soils at the site should be in accordance with ASTM D4083-01 “Standard Practice for Description of Frozen Soils” or other approved method. The geotechnical engineer can aid the contractor with supplemental recommendations for earthwork that will take place during extended periods of sub-freezing weather, as required.

6.1.7 Erosion Control

Erosion and sedimentation control measures should be employed in accordance with applicable City, County, and State regulations.

6.2 Temporary Excavations

6.2.1 Overview

Conventional earthmoving equipment in proper working condition should be capable of making necessary excavations for the anticipated site cuts into the sediments described earlier in this report. All excavations should be in accordance with applicable OSHA and state regulations. It is the contractor's responsibility to select the excavation methods, to monitor site excavations for safety, and to provide any shoring required to protect personnel and adjacent improvements. A “competent person”, as defined by OR-OSHA, should be on-site during construction in accordance with regulations presented by OR-OSHA. CGT's current role on the project does <u>not</u> include review or oversight of excavation safety.

6.2.2 <u>OSHA Soil Type</u>

For use in the planning and construction of temporary excavations up to 10 feet in depth, CGT recommends an OSHA soil type "C" be used for interbedded fine-grained and granular soils encountered in the test pits. In the event the contractor desires to increase the inclination of temporary cut slopes during construction, the geotechnical engineer should be consulted to provide specific recommendations on a case-by-case basis.

6.2.3 <u>Utility Trenches</u>

Temporary trench cuts should stand near vertical to depths of approximately 4 feet bgs in the native sediments (SM, ML, GM, GC) encountered at the site. If caving of the sidewalls is observed during excavation, the sidewalls should be flattened or shored. Depending on the time of year trench excavations occur, trench dewatering may be required in order to maintain dry working conditions. If groundwater is present at the base of utility excavations, we recommend placing trench stabilization material at the base of the excavations. Trench stabilization material should be in conformance with Section 6.4.3 of this report.

6.2.4 <u>Excavations Near Foundations</u>

Excavations near footings should <u>not</u> extend within a 1½ horizontal to 1 vertical (1½H:1V) plane projected out and down from the outside, bottom edge of the footings. In the event excavation needs to extend below the referenced plane, temporary shoring of the excavation and/or underpinning of the subject footing may be required. The geotechnical engineer should be consulted to review proposed excavation plans for this design case to provide specific recommendations.

6.2.5 <u>Draping of Cut Slopes</u>

In wet weather conditions, we recommend temporary cut slopes in excess of 4 feet in height (created during construction) be draped with minimum 10-mil plastic sheeting (e.g. polyethylene) Draping of cut slopes less than 4 feet in height may also be performed. The draping should extend from the base of the cut slope and back from the top of the cut slope sufficient to limit runoff from flowing under the covering. The plastic sheets should be lapped sufficiently to prevent water from flowing directly onto the slope and should extend at least several feet beyond each side of the cut area. The plastic should be weighted or otherwise anchored so that it remains on the slope during construction. Runoff from the sheeting should <u>not</u> be allowed to pond or infiltrate into the subsurface at the toe of the slope, but should be collected and diverted away from the cut slope to a suitable discharge point.

6.3 Wet Weather Considerations

Notwithstanding the generally arid conditions of the John Day area, soil conditions should be evaluated in the field by the geotechnical engineer's representative at the initial stage of site preparation to determine whether the recommendations within this section should be incorporated into construction.

6.3.1 <u>Overview</u>

Due to their fines content, the near-surface native sediments (SM, ML, GM, GC) are moisture sensitive and susceptible to disturbance during wet weather. Trafficability of these soils may be difficult, and significant damage to subgrade soils could occur, if earthwork is undertaken without proper precautions at times when the exposed soils are more than a few percentage points above optimum moisture content. Site preparation activities may need to be accomplished using track-mounted equipment, loading removed material onto trucks supported on granular haul roads, or other methods to limit soil disturbance. The geotechnical engineer or their representative should evaluate the subgrade during excavation by probing rather than proof rolling. Soils that have been disturbed during site preparation activities, or soft or loose areas identified during probing, should be over-excavated to firm, unyielding subgrade, and replaced with imported granular structural fill in conformance with Section 6.4.2 of this report.

6.3.2 Geotextile Separation Fabric

We recommend a geotextile separation fabric be placed to serve as a barrier between the prepared subgrade and granular fill/base rock in areas of repeated or heavy construction traffic. The geotextile fabric should meet the requirements presented in the current Oregon Department of Transportation Standard Specification for Construction (ODOT SSC), Section 02320.

6.3.3 Granular Working Surfaces (Haul Roads & Staging Areas)

Haul roads subjected to repeated heavy, tire-mounted, construction traffic (e.g. dump trucks, concrete trucks, etc.) will require a minimum of 18 inches of imported granular material. For light staging areas, 12 inches of imported granular material is typically sufficient. Additional granular material or geo-grid reinforcement may be recommended based on site conditions and/or loading at the time of construction. The imported granular material should be in conformance with Section 6.4.2 and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. The prepared subgrade should be covered with geotextile fabric (Section 6.3.2) prior to placement of the imported granular material. The imported granular material should be placed in a single lift (up to 24 inches deep) and compacted using a smooth-drum, non-vibratory roller until well-keyed.

6.3.4 Footing Subgrade Protection

A minimum of 3 inches of imported granular material is recommended to protect fine-grained footing excavation subgrades from foot traffic during inclement weather. The imported granular material should be in conformance with Section 6.4.2. The maximum particle size should be limited to 1 inch. The imported granular material should be placed in one lift over the prepared, undisturbed subgrade, and compacted using non-vibratory equipment until well keyed.

6.4 Structural Fill

The geotechnical engineer should be provided the opportunity to review all materials considered for use as structural fill (prior to placement). Samples of the proposed fill materials should be submitted to the geotechnical engineer a minimum of 5 business days prior their use on site[13]. The geotechnical engineer or their representative should be contacted to evaluate compaction of structural fill as the material is being placed. Evaluation of compaction may take the form of in-place density tests and/or proof roll tests with suitable equipment. Structural fill should be evaluated at intervals not exceeding every 2 vertical feet as the fill is being placed.

6.4.1 On-Site Soils – General Use

6.4.1.1 Silty Sand (SM), Silt (ML), Silty Gravel (GM) to Clayey Gravel (GC)

Re-use of these soils as structural fill may be difficult because, due to their fines content, these soils are sensitive to small changes in moisture content and are difficult, if not impossible, to adequately compact

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[13] Laboratory testing for moisture density relationship (Proctor) is required. Tests for gradation may be required.

during wet weather. We anticipate the moisture content of these soils will be higher than the optimum moisture content for satisfactory compaction. Therefore, moisture conditioning (drying) should be expected in order to achieve adequate compaction. If used as structural fill, these soils should be free of organic matter, debris, and particles larger than 4 inches. Processing (removal) of large cobbles and boulders may be required in some areas and should be factored. When used as structural fill, these soils should be placed in lifts with a maximum pre-compaction thickness of about 8 inches at moisture contents within –1 and +3 percent of optimum, and compacted to not less than 92 percent of the material's maximum dry density, as determined in general accordance with ASTM D1557 (Modified Proctor).

If the on-site materials cannot be properly moisture-conditioned and/or processed, we recommend using imported granular material for structural fill.

6.4.2 Imported Granular Structural Fill – General Use

Imported granular structural fill should consist of angular pit or quarry run rock, crushed rock, or crushed gravel that is fairly well graded between coarse and fine particle sizes. The granular fill should contain no organic matter, debris, or particles larger than 4 inches, and have less than 10 percent material passing the U.S. Standard No. 200 Sieve. For fine-grading purposes, the maximum particle size should be limited to 1½ inches. The percentage of fines can be increased to 15 percent of the material passing the U.S. Standard No. 200 Sieve if placed during dry weather, and provided the fill material is moisture-conditioned, as necessary, for proper compaction. Imported granular fill material should be placed in lifts with a maximum thickness of about 12 inches, and compacted to not less than 95 percent of the material's maximum dry density, as determined in general accordance with ASTM D1557 (Modified Proctor). Proper moisture conditioning and the use of vibratory equipment will facilitate compaction of these materials.

Granular fill materials with high percentages of particle sizes in excess of 1½ inches are considered non-moisture-density testable materials. As an alternative to conventional density testing, compaction of these materials should be evaluated by proof roll test observation (deflection tests), where accepted by the geotechnical engineer.

6.4.3 Trench Base Stabilization Material

If groundwater is present at the base of utility excavations, trench base stabilization material should be placed. Trench base stabilization material should consist of a minimum of 1 foot of well-graded granular material with a maximum particle size of 4 inches and less than 5 percent material passing the U.S. Standard No. 4 Sieve. The material should be free of organic matter and other deleterious material, placed in one lift, and compacted until well-keyed.

6.4.4 Trench Backfill Material

Trench backfill for the utility pipe base and pipe zone should consist of granular material as recommended by the utility pipe manufacturer. Trench backfill above the pipe zone should consist of well-graded granular material containing no organic matter or debris, have a maximum particle size of ¾ inch, and have less than 8 percent material passing the U.S. Standard No. 200 Sieve. As a guideline, trench backfill should be placed in maximum 12-inch-thick lifts. The earthwork contractor may elect to use alternative lift thicknesses based on their experience with specific equipment and fill material conditions during construction in order to achieve the required compaction. The following table presents recommended relative compaction percentages for utility trench backfill.

Table 2 Utility Trench Backfill Compaction Recommendations

Backfill Zone	Recommended Minimum Relative Compaction	
	Structural Areas[1,2]	Landscaping Areas
Pipe Base and Within Pipe Zone	90% ASTM D1557 or pipe manufacturer's recommendation	88% ASTM D1557 or pipe manufacturer's recommendation
Above Pipe Zone	92% ASTM D1557	90% ASTM D1557
Within 3 Feet of Design Subgrade	95% ASTM D1557	90% ASTM D1557

[1] Includes proposed building, pavement areas, structural fill areas, exterior hardscaping, etc.
[2] Or as specified by the local jurisdiction where located in the public right of way.

6.4.5 Controlled Low-Strength Material (CLSM)

CLSM is a self-compacting, cementitious material that is typically considered when backfilling localized areas. CLSM is sometimes referred to as "controlled density fill" or CDF. Due to its flowable characteristics, CLSM typically can be placed in restricted-access excavations where placing and compacting fill is difficult. If chosen for use at this site, we recommend the CLSM be in conformance with Section 00442 of the most recent, ODOT SSC. The geotechnical engineer's representative should observe placement of the CLSM and obtain samples for compression testing in accordance with ASTM D4832. As a guideline, for each day's placement, two compressive strength specimens from the same CLSM sample should be tested. The results of the two individual compressive strength tests should be averaged to obtain the reported 28-day compressive strength. If CLSM is considered for use on this site, please contact the geotechnical engineer for site-specific and application-specific recommendations.

6.5 Permanent Slopes

6.5.1 Overview

Permanent cut or fill slopes constructed at the site, if any, should be graded at 2H:1V or flatter. Constructed slopes should be overbuilt by a few feet depending on their size and gradient so that they can be properly compacted prior to being cut to final grade. The surface of all slopes should be protected from erosion by seeding, sodding, or other acceptable means. Adjacent on-site and off-site structures should be located at least 5 feet from the top of slopes.

6.5.2 Placement of Fill on Slopes

New fill should be placed and compacted against horizontal surfaces. Where slopes exceed 5H:1V, the slopes should be keyed and benched prior to structural fill placement in general accordance with the attached Fill Slope Detail, Figure 5. If subdrains are needed on benches, subject to the review of the CGT geotechnical representative, they should be placed as shown on the attached Fill Slope Detail. In order to achieve well-compacted slope faces, slopes should be overbuilt by a few feet and then trimmed back to proposed final grades. A representative from CGT should observe the benches, keyways, and associated subdrains, if needed, prior to placement of structural fill.

6.6 Shallow Foundations

6.6.1 Subgrade Preparation

Satisfactory subgrade support for shallow foundations can be obtained from the native, medium stiff/medium dense to better sediments (SM, ML, GM, GC), or on structural fill which is properly placed and compacted on these materials during construction. The geotechnical engineer or their representative should be contacted to observe subgrade conditions prior to placement of forms, reinforcement steel, or granular backfill (if required). If soft, loose, or otherwise unsuitable soils are encountered, they should be over-excavated as recommended by the geotechnical representative at the time of construction. Boulders (i.e. particles in excess of 12 inches in diameter) encountered at design foundation subgrade elevations should be removed. The resulting over-excavation(s) should be brought back to grade with imported granular structural fill in conformance with Section 6.4.2. The maximum particle size of over-excavation backfill should be limited to 1½ inches. All granular pads for footings should be constructed a minimum of 6 inches wider on each side of the footing for every vertical foot of over-excavation.

6.6.2 Minimum Footing Width & Embedment

Minimum footing widths should be in conformance with the most recent Oregon Residential Structural Code (ORSC). As a guideline, CGT recommends individual spread footings have a minimum width of 24 inches. For one-story, light-framed structures, we recommend continuous wall footings have a minimum width of 12 inches. Similarly, for two- and three-story, light-framed structures, we recommend continuous wall footings have a minimum width of 15 inches and 18 inches, respectively. All footings should be founded at least 24 inches below the lowest, permanent adjacent grade for frost protection.

6.6.3 Bearing Pressure & Settlement

Footings founded as recommended above should be proportioned for a maximum allowable soil bearing pressure of 2,500 pounds per square foot (psf). This bearing pressure is a net bearing pressure, applies to the total of dead and long-term live loads, and may be increased by one-third when considering seismic or wind loads. For foundations founded as recommended above, total settlement of foundations is anticipated to be less than 1 inch. Differential settlements between adjacent columns and/or bearing walls should not exceed ½-inch. If an increased allowable soil bearing pressure is desired, the geotechnical engineer should be consulted.

6.6.4 Lateral Capacity

A maximum passive (equivalent fluid) earth pressure of 250 pounds per cubic foot (pcf) is recommended for design of footings cast neat into excavations in suitable native soil or confined by imported granular structural fill that is properly placed and compacted during construction. The recommended earth pressure was computed using a factor of safety of 1½, which is appropriate due to the amount of movement required to develop full passive resistance. In order to develop the above capacity, the following should be understood:

1. Concrete must be poured neat in excavations or the foundations must be backfilled with imported granular structural fill,
2. The adjacent grade must be level,
3. The static ground water level must remain below the base of the footings throughout the year.
4. Adjacent floor slabs, pavements, or the upper 12-inch-depth of adjacent, unpaved areas should not be considered when calculating passive resistance.

An ultimate coefficient of friction equal to 0.35 may be used when calculating resistance to sliding for footings founded on the native soils described above. An ultimate coefficient of friction equal to 0.45 may be used when calculating resistance to sliding for footings founded on a minimum of 6 inches of imported granular structural fill (crushed rock) that is properly placed and compacted during construction.

6.6.5 Subsurface Drainage

Recognizing the predominantly granular soils with significant fine-grained portion encountered at this site, we recommend placing foundation drains at the exterior, base elevations of perimeter continuous wall footings. Foundation drains should consist of a minimum 4-inch diameter, perforated, PVC drainpipe wrapped with a non-woven geotextile filter fabric. The drains should be backfilled with a minimum of 2 cubic feet of open graded drain rock per lineal foot of pipe. The drain rock should also be encased in a geotextile fabric in order to provide separation from the surrounding fine-grained soils. Foundation drains should be positively sloped and should outlet to a suitable discharge point. The geotechnical engineer or their representative should observe the drains prior to backfilling. Roof drains should not be tied into foundation drains.

6.6.6 Foundation Setback from Descending Slope

Section R403.1.9.2 of the 2017 ORSC requires that foundations be a sufficient depth to provide horizontal setback from a descending slope with gradients in excess of 3H:1V. The required setback is ⅓ the height of the slope and a maximum of 40 feet measured horizontally from the base of the foundation to the slope face. CGT is in agreement with the code-specified minimum setback for use in general planning for this project. In the event this setback is desired to be reduced, the geotechnical engineer should be consulted to review the proposed construction.

6.6.7 Toe of Slope Clearance

Section R403.1.9.1 of the 2017 ORSC requires a setback between the toe of an ascending slope with a gradient in excess of 3H:1V and the nearest wall of the proposed structure. The purpose of the setback is to help provide protection from surficial failures, erosion of the slope, and slope drainage. The toe of slope clearance should be ½ the slope height or a maximum of 15 feet. For retained slopes, the height of the slope should be measured considering the top of the retaining wall as the toe of the slope. CGT is in agreement with the code-specified minimum clearance for this project. In the event this clearance is desired to be reduced, the geotechnical engineer should be consulted to review the proposed construction.

6.7 Rigid Retaining Walls

6.7.1 Footings

Retaining wall footings should be designed and constructed in conformance with the recommendations presented in Section 6.6, as applicable.

6.7.2 Wall Drains

We recommend placing retaining wall drains at the base elevation of the heel of retaining wall footings. Retaining wall drains should consist of a minimum 4-inch-diameter, perforated, HDPE (High Density Polyethylene) drainpipe wrapped with a non-woven geotextile filter fabric. The drains should be backfilled with a minimum of 2 cubic feet of open graded drain rock per lineal foot of pipe. The drain rock should be encased in a geotextile fabric in order to provide separation from the surrounding soils. Retaining wall drains should be positively sloped and should outlet to a suitable discharge point. The geotechnical engineer or their representative should be contacted to observe the drains prior to backfilling. Roof or area drains should not be tied into retaining wall drains.

6.7.3 Wall Backfill

Retaining walls should be backfilled with imported granular structural fill in conformance with Section 6.4.2 and contain less than 5 percent passing the U.S. Standard No. 200 Sieve. The backfill should be compacted to a minimum of 90 percent of the material's maximum dry density as determined in general accordance with ASTM D1557 (Modified Proctor). When placing fill behind walls, care must be taken to minimize undue lateral loads on the walls. Heavy compaction equipment should be kept at least "H" feet from the back of the walls, where "H" is the height of the wall. Light mechanical or hand tamping equipment should be used for compaction of backfill materials within "H" feet of the back of the walls.

6.7.4 Design Parameters & Limitations

For rigid retaining walls founded, backfilled, and drained as recommended above, the following table presents parameters recommended for design.

Table 3 Design Parameters for Rigid Retaining Walls

Retaining Wall Condition	Modeled Backfill Condition	Static Equivalent Fluid Pressure (S_A)[1]	Seismic Equivalent Fluid Pressure (S_{AE})[1,2]	Surcharge from Uniform Load, q, Acting on Backfill Behind Retaining Wall
Not Restrained from Rotation	Level (i = 0)	29 pcf	34 pcf	0.22*q
Restrained from Rotation	Level (i = 0)	52 pcf	52 pcf	0.38*q

[1] Refer to the attached Figure 6 for a graphical representation of static and seismic loading conditions. Seismic resultant force acts at 0.6H above the base of the wall.

[2] Seismic (dynamic) lateral loads were computed using the Mononobe-Okabe Equation as presented in the 1997 Federal Highway Administration (FHWA) design manual. Static and seismic equivalent fluid pressures are not additive.

The above design recommendations are based on the assumptions that:

- The walls consist of concrete cantilevered retaining walls (β = 0 and δ = 24 degrees, see Figure 6).
- The walls are 10 feet or less in height.
- The backfill is drained and consists of imported granular structural fill (ϕ = 38 degrees).
- No line load or point load surcharges are imposed behind the walls.
- The grade behind the wall is level, or sloping down and away from the wall, for a distance of 10 feet or more from the wall.
- The grade in front of the walls is level or ascending for a distance of at least 5 feet from the wall.

Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project vary from these assumptions.

6.8 Floor Slabs

6.8.1 Subgrade Preparation

Satisfactory subgrade support for slabs constructed on grade, supporting up to 150 psf area loading, can be obtained from the native, native medium stiff/medium dense to better sediments (SM, ML, GM, GC), or new structural fill that is properly placed and compacted on these materials during construction. The geotechnical engineer or their representative should observe floor slab subgrade soils to evaluate surface consistencies. If soft, loose, or otherwise unsuitable soils are encountered, they should be over-excavated as recommended by the CGT geotechnical representative at the time of construction. The resulting over-excavation should be brought back to grade with imported granular structural fill as described in Section 6.4.2.

6.8.2 Crushed Rock Base

Concrete floor slabs should be supported on a minimum 6-inch-thick layer of crushed rock (base rock).

6.8.2.1 Conventional Base Rock

Floor slab base rock should consist of well-graded granular material (crushed rock) containing no organic matter or debris, have a maximum particle size of ¾ inch, and have less than 10 percent material passing the U.S. Standard No. 200 Sieve. Floor slab base rock should be placed in one lift and compacted to not less than 95 percent of the material's maximum dry density as determined in general accordance with ASTM D1557 (Modified Proctor). We recommend "choking" the surface of the base rock with sand just prior to concrete placement. Choking means the voids between the largest aggregate particles are filled with sand, but does not provide a layer of sand above the base rock. Choking the base rock surface reduces the lateral restraint on the bottom of the concrete during curing. Choking the base rock also reduces punctures in vapor retarding membranes due to foot traffic where such membranes are used.

6.8.2.2 Gas Permeable Base Rock

Floor slab base rock in areas where radon gas mitigation is desired should consist of open-graded crushed rock containing no organic matter or debris, with all material passing through a 2-inch sieve and retained on the ¼-inch sieve, in accordance with 2017 ORSC Appendix F, Section AF103.2, Bullet 1.

CGT recommends that a minimum 10-mil polyethylene sheeting or equivalent material with equal or greater tensile strength, resistance to puncture, resistance to deterioration, and resistance to water-vapor transmission be placed on top of the gas-permeable base rock to act as a soil-gas-retarder. Placement and installation of this sheeting should be in conformance with that indicated in 2017 ORSC Appendix F, Section AF103.3.

The geotechnical engineer or their representative should be contacted to observe gas-permeable base rock conditions prior to placement of the soil-gas-retarder.

6.8.3 Design Considerations

For floor slabs constructed with a 6-inch thick base rock layer as recommended, an effective modulus of subgrade reaction of 150 pounds per cubic inch (pci) is recommended for the design of the floor slab. A higher effective modulus of subgrade reaction can be obtained by increasing the base rock thickness. Please contact the geotechnical engineer for additional recommendations if a higher modulus is desired. Floor slabs constructed as recommended will likely settle less than ½ inch. For general floor slab construction, slabs should be jointed around columns and walls to permit slabs and foundations to settle differentially.

6.8.4 Subgrade Moisture Considerations

Liquid moisture and moisture vapor should be expected at the subgrade surface. The recommended crushed rock base is anticipated to provide protection against liquid moisture. Where moisture vapor emission through the slab must be minimized, e.g. impervious floor coverings, storage of moisture sensitive materials directly on the slab surface, etc., a vapor retarding membrane or vapor barrier below the slab should be considered. Factors such as cost, special considerations for construction, floor coverings, and end use suggest that the decision regarding a vapor retarding membrane or vapor barrier be made by the architect and owner.

If a vapor retarder or vapor barrier is placed below the slab, its location should be based on current American Concrete Institute (ACI) guidelines, ACI 302 Guide for Concrete Floor and Slab Construction. In some cases, this indicates placement of concrete directly on the vapor retarder or barrier. Please note that the placement of concrete directly on impervious membranes increases the risk of plastic shrinkage cracking and slab curling in the concrete. Construction practices to reduce or eliminate such risk, as described in ACI 302, should be employed during concrete placement.

6.9 Pavements

6.9.1 Subgrade Preparation

Pavement subgrade preparation should be in conformance with Section 6.1.5 of this report. Pavement subgrade surfaces should be crowned (or sloped) for proper drainage in accordance with specifications provided by the project civil engineer.

6.9.2 Traffic Classifications

Recognizing that traffic data has not been provided, CGT has considered two levels of traffic demand for review and design of pavement sections. We modeled the following two design cases (traffic levels) developed from the Asphalt Pavement Association of Oregon (APAO):

- *APAO Level I (Very Light):* This design case considers typical average daily truck traffic (ADTT) of 1 per day over 20 years. Among others, examples under this loading consist of passenger car parking stalls, residential driveways, and seasonal recreational roads.
- *APAO Level II (Light):* This design case considers typical ADTT of 2 to 7 per day over 20 years. Examples under this loading consist of residential streets and parking lots of less than 500 stalls.

We recommend the owner and design team review the traffic levels presented above and select those that most accurately represent anticipated daily truck traffic for select new pavements.

6.9.3 Input Parameters

Design of the asphalt concrete (AC) pavement sections presented below were based on the parameters presented in the following table, the American Association of State Highway and Transportation Officials (AASHTO) 1993 “Design of Pavement Structures” manual, and pavement design manuals presented by APAO and ODOT[14]. If any of the items listed need revision, please contact us and we will reassess the provided design sections.

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[14] Oregon Department of Transportation (ODOT) Pavement Design Guide, August 2011.

Table 4 Input Parameters Used in AC Pavement Design

Input Parameter	Design Value[1]
Pavement Design Life	20 years
Annual Percent Growth	0 percent
Initial Serviceability[2]	4.2
Terminal Serviceability[2]	2.5
Reliability[2]	75 percent
Standard Deviation[2]	0.49
Drainage Factor[3]	1.0

Input Parameter		Design Value[1]
Resilient Modulus	Subgrade (Native Sediments)[4]	5,000 psi
	Crushed Aggregate Base[2]	20,000 psi
Structural Coefficient	Crushed Aggregate Base[2]	0.10
	Asphalt[2]	0.42
Vehicle Traffic[4] (range in ESAL[5])	APAO Level I (Very Light)	Less than 10,000
	APAO Level II (Light)	Less than 50,000

[1] If any of the above parameters are incorrect, please contact us so that we may revise our recommendations, if warranted.
[2] Value based on guidelines presented in the ODOT Pavement Design Guide.
[3] Assumes good drainage away from pavement, base, and subgrade is achieved by proper crowning of subgrades.
[4] Values based on experience with similar soils in the region.
[5] ESAL = Total 18-Kip equivalent single axle load. Traffic levels taken from Table 3.1 of APAO manual. If actual traffic levels will be above those identified above, the geotechnical engineer should be consulted.

6.9.4 Recommended Minimum Sections

The following table presents the minimum AC pavement sections for various traffic loads indicated in the preceding table, based on the referenced AASHTO procedures.

Table 5 Recommended Minimum AC Pavement Sections

Material	APAO Traffic Loading	
	Level I (Passenger Car Traffic Only)	Level II (Entrance & Service Drive Lanes)
Asphalt Pavement (inches)	3	3½
Crushed Aggregate Base (inches)[1]	6	8
Subgrade Soils	Prepared in conformance with Section 6.9.1 of this report.	

[1] Thickness shown assumes dry weather construction. A granular sub-base section and/or a geotextile separation fabric may be required in wet conditions in order to support construction traffic and protect the subgrade. Refer to Section 6.3 for additional discussion.

6.9.5 AC Pavement Materials

We recommend pavement aggregate base consist of dense-graded aggregate in conformance with Section 02630.10 of the most recent ODOT SSC, with the following additional considerations. We recommend the material consist of crushed rock or gravel, have a maximum particle size of 1½ inches, and have less than 10 percent material passing the U.S. Standard No. 200 Sieve. Aggregate base should be compacted to not less than 95 percent of the material's maximum dry density as determined in general accordance with ASTM D1557 (Modified Proctor).

We recommend asphalt pavement consist of Level 2, ½-inch, dense-graded AC in conformance with the most recent ODOT SSC. Asphalt pavement should be compacted to at least 91 percent of the material's theoretical maximum density as determined in general accordance with ASTM D2041 (Rice Specific Gravity), or as specified by the local jurisdiction.

6.10 Additional Considerations

6.10.1 Drainage

Subsurface drains should be connected to the nearest storm drain or other suitable discharge point. Paved surfaces and grading near or adjacent to the buildings should be sloped to drain away from the buildings. Surface water from paved surfaces and open spaces should be collected and routed to a suitable discharge point. Surface water should not be directed into foundation drains, retaining wall drains, or onto site slopes.

6.10.2 Expansive Potential

The near surface native soils consist of generally low to medium plasticity fine-grained soils and granular soils with low plasticity fines. Based on experience with similar soils in the region, these soils are not considered susceptible to appreciable movements from changes in moisture content. Accordingly, no special considerations are required to mitigate expansive potential of the near surface soils at the site.

7.0 RECOMMENDED ADDITIONAL SERVICES

7.1 Design Review

Geotechnical design review is of paramount importance. We recommend the geotechnical design review take place prior to releasing bid packets to contractors.

7.2 Observation of Construction

Satisfactory earthwork, foundation, floor slab, and pavement performance depends to a large degree on the quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during subsurface explorations, and recognition of changed conditions often requires experience. We recommend that qualified personnel visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those observed to date and anticipated in this report. We recommend the geotechnical engineer or their representative attend a pre-construction meeting coordinated by the contractor and/or developer. The project geotechnical engineer or their representative should provide observations and/or testing of at least the following earthwork elements during construction:

- Site Stripping
- Subgrade Preparation for Shallow Foundations, Retaining Walls, Structural Fills, Floor Slabs, and Pavements
- Placement of Perimeter Foundation Drains & Retaining Wall Drains
- Compaction of Structural Fill, Retaining Wall Backfill, and Utility Trench Backfill
- Compaction of Base Rock for Floor Slabs and Pavements
- Compaction of Asphalt Concrete for Pavements

It is imperative that the owner and/or contractor request earthwork observations and testing at a frequency sufficient to allow the geotechnical engineer to provide a final letter of compliance for the earthwork activities.

8.0 LIMITATIONS

We have prepared this report for use by the owner/developer and other members of the design and construction team for the proposed development. The opinions and recommendations contained within this report are forwarded to assist in the planning and design process and are not intended to be, nor should they be construed as, a warranty of subsurface conditions.

We have made observations based on our explorations that indicate the soil conditions at only those specific locations and only to the depths penetrated. These observations do not necessarily reflect soil types, strata thickness, or water level variations that may exist between or away from our explorations. If subsurface conditions vary from those encountered in our site explorations, CGT should be alerted to the change in conditions so that we may provide additional geotechnical recommendations, if necessary. Observation by experienced geotechnical personnel should be considered an integral part of the construction process.

The owner/developer is responsible for ensuring that the project designers and contractors implement our recommendations. When the design has been finalized, prior to releasing bid packets to contractors, we recommend that the design drawings and specifications be reviewed by our firm to see that our recommendations have been interpreted and implemented as intended. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification. Design review and construction phase testing and observation services are beyond the scope of our current assignment, but will be provided for an additional fee.

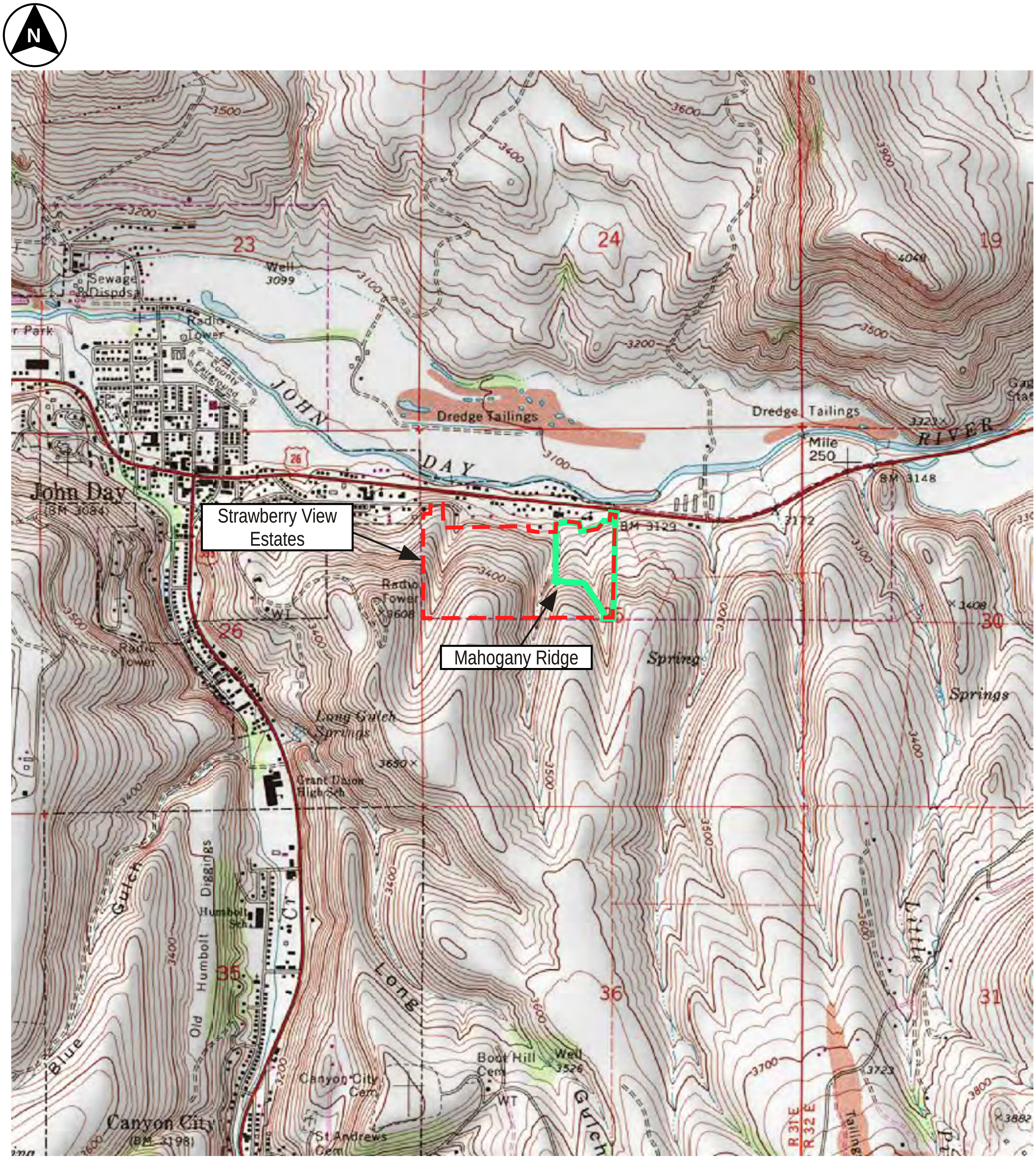
The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor’s methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design.

Geotechnical engineering and the geologic sciences are characterized by a degree of uncertainty. Professional judgments presented in this report are based on our understanding of the proposed construction, familiarity with similar projects in the area, and on general experience. Within the limitations of scope, schedule, and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared; no warranty, expressed or implied, is made. This report is subject to review and should not be relied upon after a period of three years.

MAHOGANY RIDGE SUBDIVISION - JOHN DAY, OREGON
Project Number G2005305

FIGURE 1

Site Location



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Drafted by: RTH

Map created with TOPO!™, © 2006 National Geographic Holdings
USGS 7.5 Minute Topographic Map Series, John Day, Oregon Quadrangle.

Township 13 South, Range 31 East, Section 25 Willamette Meridian

Latitude: 44.412493° North
Longitude: 118.932451° West

1 Inch = 2,000 feet

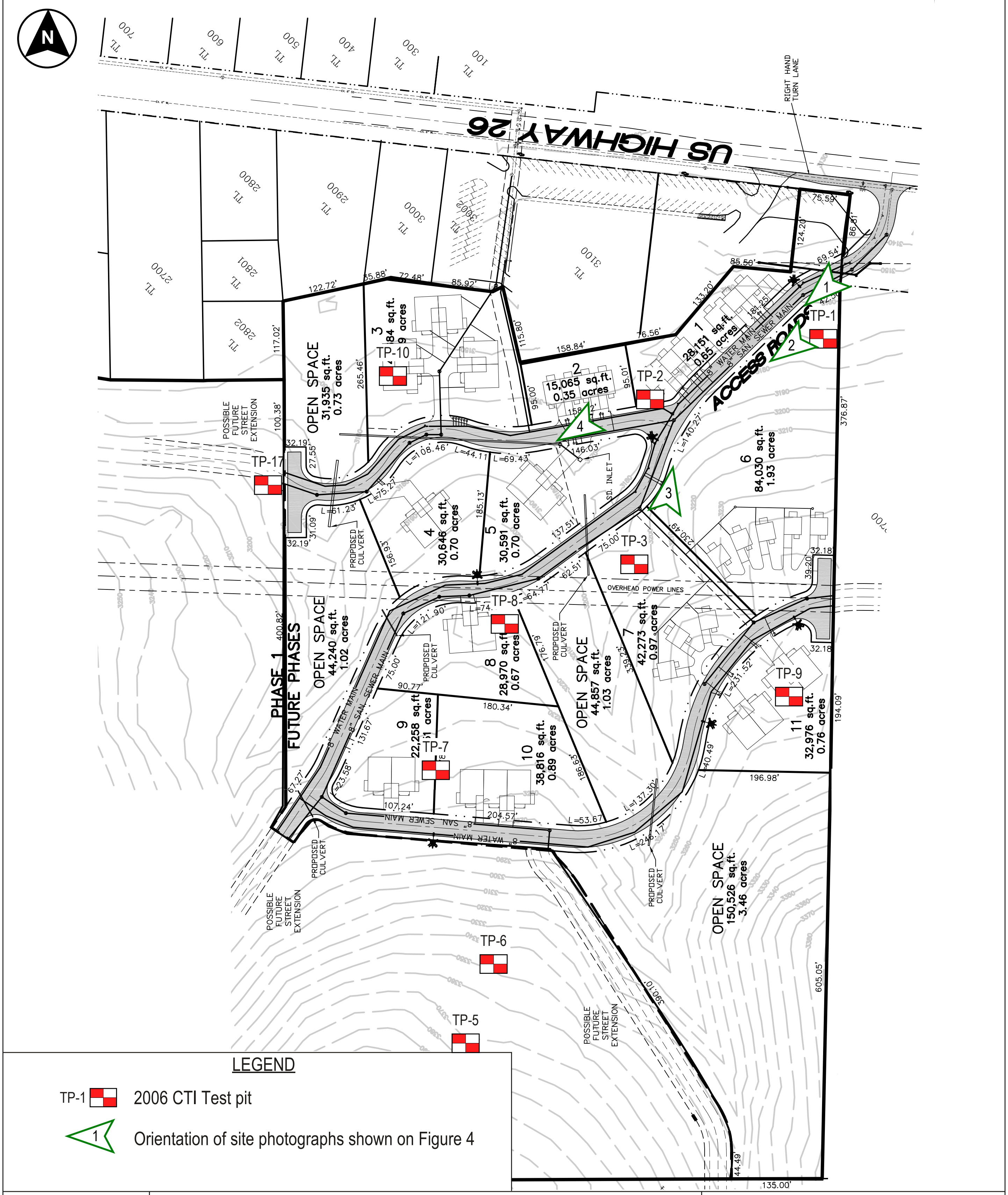
0 2000 4000

MAHOGANY RIDGE SUBDIVISION - JOHN DAY, OREGON

Project Number G2005305

FIGURE 2

Site Plan



LEGEND

TP-1 🟥 2006 CTI Test pit

1 Orientation of site photographs shown on Figure 4

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NOTES: Drawing based on "Mahogany Ridge Street Improvments," prepared by Sisul Engineering, dated May 2020. 2006 test pit locations from CGT report G0602826 dated August 2, 2006.

1 Inch = 200 Feet

0 200 400

MAHOGANY RIDGE SUBDIVISION - JOHN DAY, OREGON
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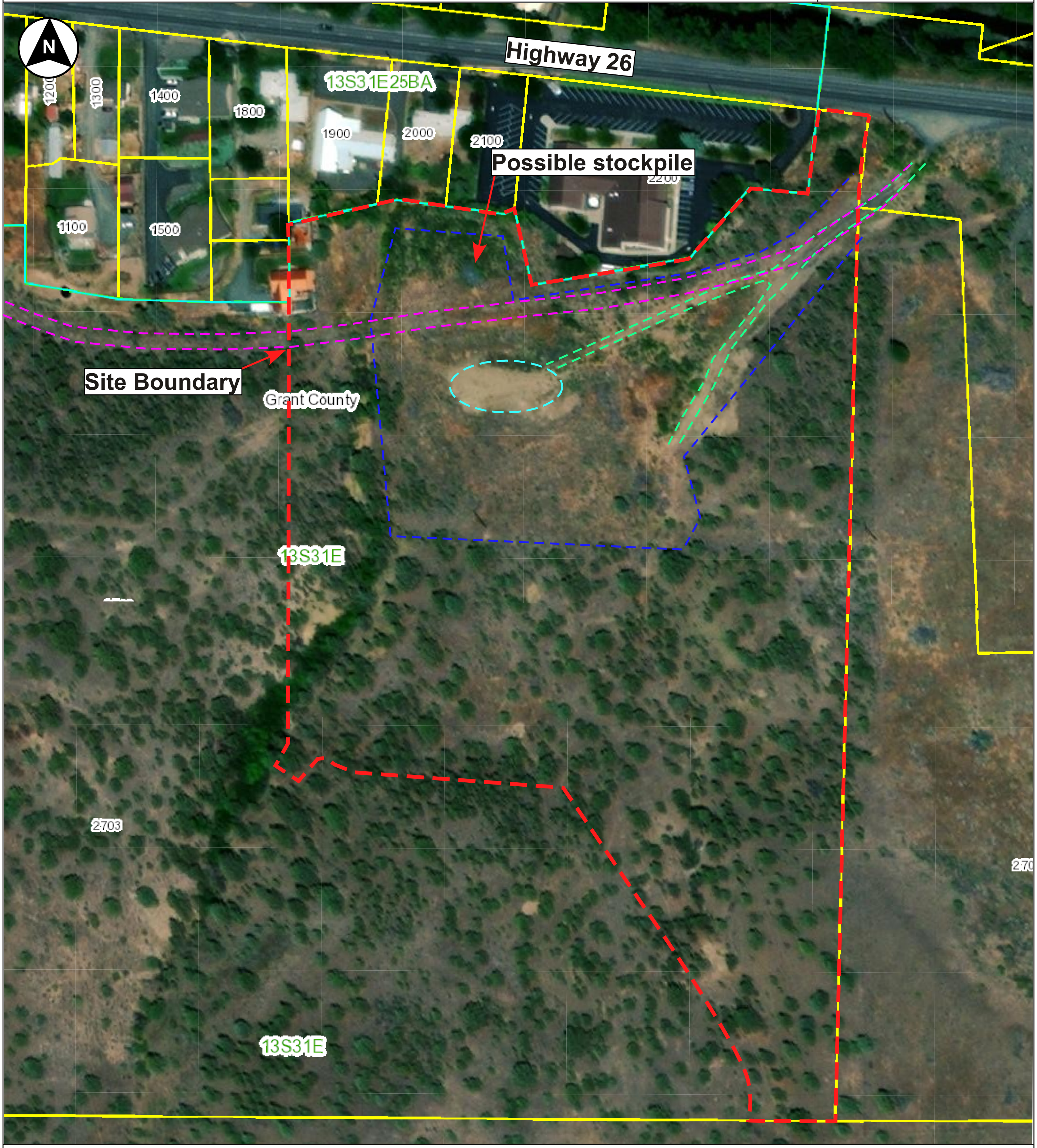
FIGURE 3

2017 Aerial Photograph



LEGEND

▬ ▬	Dirt access road observed in 2006	▬ ▬	Approximate limits of 2011 clearing
▬ ▬	Gravel access road observed in 2020	▬ ▬	Possible borrow pit

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NOTES: 2017 aerial photograph from ORmap.org. All locations are approximate.

1 Inch = 200 Feet

0 200 400

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Project Number G2005305

FIGURE 4

Site Photographs


Photograph 1


Photograph 2


Photograph 3


Photograph 4

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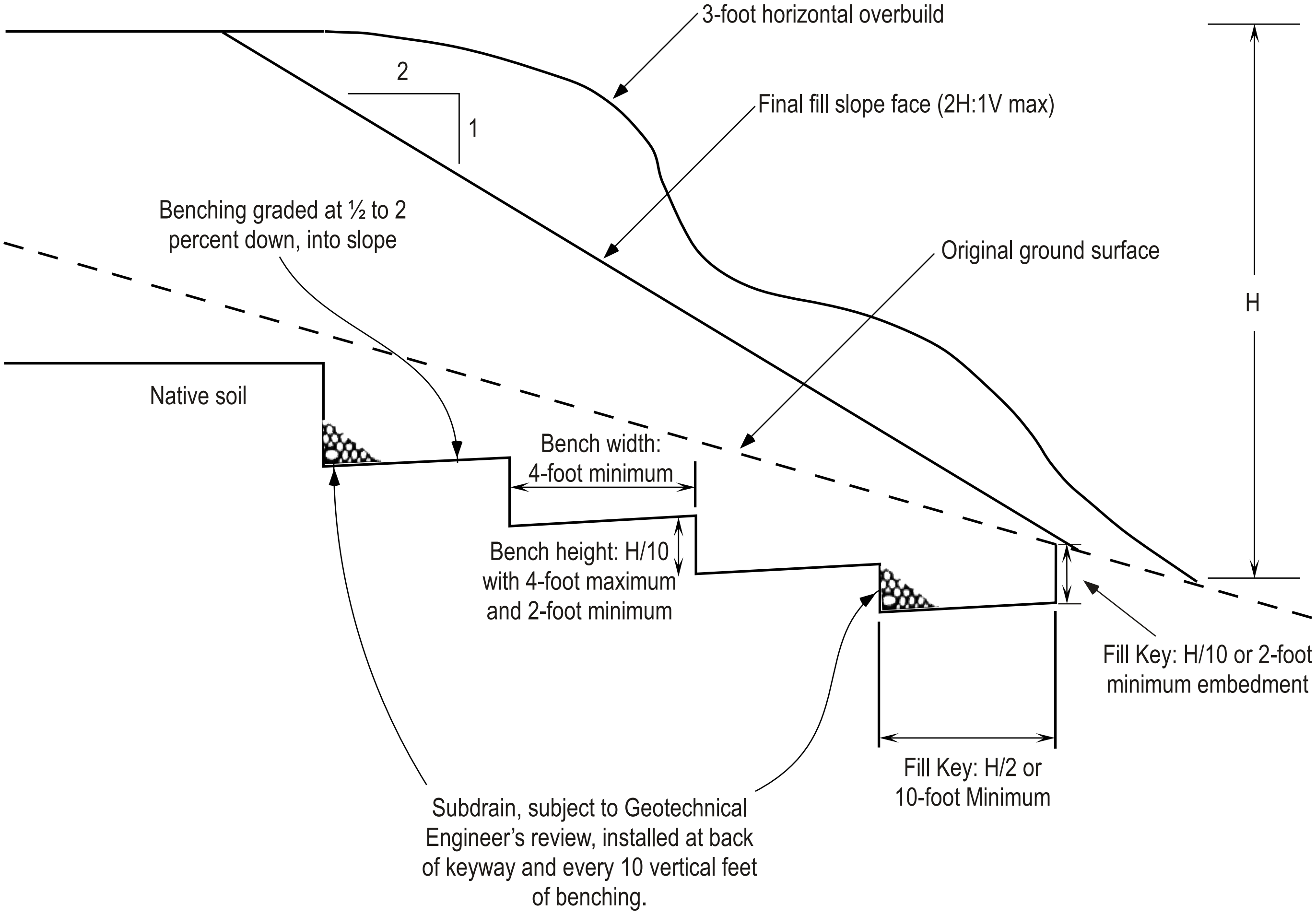
Drafted by: RTH

See Figure 2 for approximate photograph locations and directions. Photographs were taken at the time of our fieldwork.

FIGURE 5

Fill Slope Detail



NOTE: Surfaces to receive fill with slopes steeper than 5H:1V (horizontal:vertical) should be benched and keyed as shown.

MAHOGANY RIDGE SUBDIVISION - JOHN DAY, OREGON
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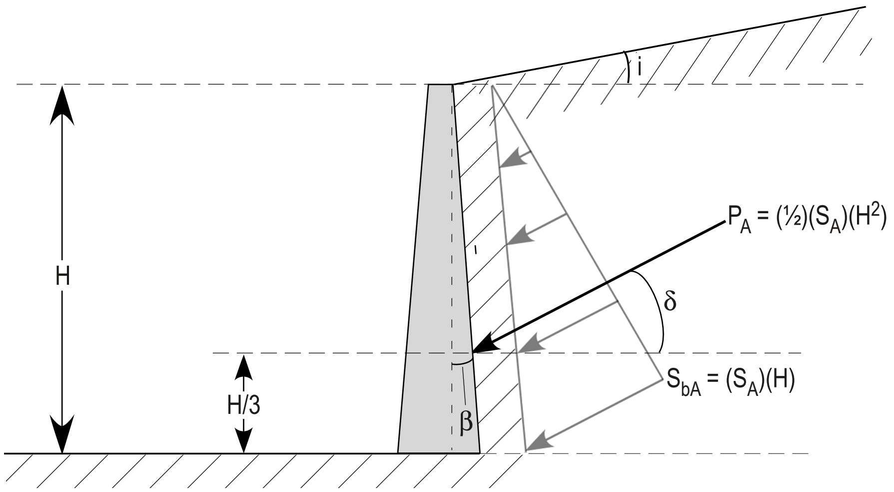
FIGURE 6

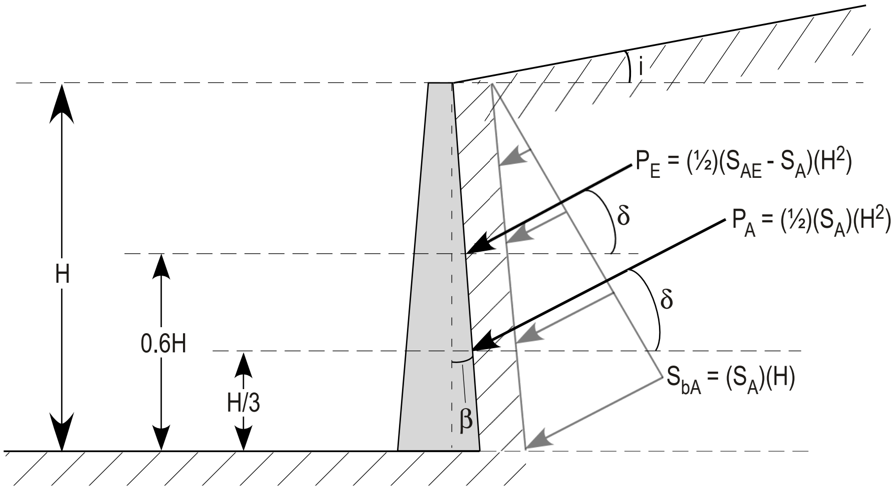
Retaining Walls

ACTIVE LATERAL PRESSURE DISTRIBUTION

STATIC LOADING CONDITIONS



SEISMIC LOADING CONDITIONS



LEGEND

S_A = Active lateral equivalent fluid pressure (lb/ft^3)*

S_{bA} = Active lateral earth pressure (static) at the bottom of wall (lb/ft^3)

S_{AE} = Active total (static + seismic) equivalent fluid pressure (lb/ft^3)*

i = Slope of backfill, relative to horizontal (degrees)**

β = Slope of back of wall, relative to vertical (degrees)**

P_A = Static active thrust force acting at H/3 from bottom of retaining wall (lb/ft)

P_E = Dynamic active thrust force acting at 0.6H from bottom of retaining wall (lb/ft)

δ = Angle from normal of back of wall (degrees). Based on friction developing between wall and backfill**

*Refer to report text for calculated values **Refer to report text for modeled/assumed values

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Notes

1. Uniform pressure distribution of seismic loading is based on empirical evaluations [Sherif et al, 1982 and Whitman, 1990].
2. Placement of seismic resultant force at 0.6H is based on wall behavior and model test results [Whitman, 1990].

Carlson Geotechnical
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Appendix A: Subsurface Investigation and Laboratory Testing

Mahogany Ridge Subdivision
East of John Day, South of Hwy 26
John Day, Oregon

CGT Project Number G2005305

July 7, 2020

Prepared For:

Joshua T. Walker
Mahogany Ridge Properties
601 S Canyon Boulevard
John Day, Oregon 97845

Prepared by
Carlson Geotechnical

Exploration Key...Figure A1
Soil Classification...Figure A2
Exploration Logs...Figures A3 – A10

A.1.0 SUBSURFACE INVESTIGATION

Our field investigation consisted of seventeen test pits completed in June 2006. The exploration locations are shown on the Site Plan, attached to the geotechnical report as Figure 2. As shown on the Site Plan, eight of the test pits (TP-1, TP-2, TP-3, TP-7, TP-8, TP-9, TP-10, and TP-17) were excavated within or very close to the current development area. The exploration locations shown on the Site Plan were originally determined in 2006 based on measurements from existing site features (property corners, etc.) and have been approximated onto the recently provided site layout plan. Surface elevations indicated on the logs were estimated based on the topographic contours shown on the referenced Site Plan and are approximate. The attached figures detail the exploration methods (Figure A1), soil classification criteria (Figure A2), and present detailed logs of the explorations (Figures A3 through A10).

A.1.1 Test Pits

CGT observed the excavation of eight test pits (TP-1, TP-2, TP-3, TP-7, TP-8, TP-9, TP-10, and TP-17) at the site on June 16, 2006, to depths of about 6 to 14 feet bgs. The test pits were excavated using a CAT 330B excavator provided and operated provided by our original excavation subcontractor, Winegar Excavating of John Day, Oregon. The test pits were loosely backfilled with the excavated materials upon completion.

A.1.2 In-Situ Testing - Pocket Penetrometer Tests

Pocket penetrometer readings were generally taken at approximate ½-foot intervals in the upper four feet of each test pit. The pocket penetrometer is a hand-held instrument that provides an approximation of the unconfined compressive strength of cohesive, fine-grained soils. The correlation between pocket penetrometer readings and the consistency of cohesive, fine-grained soils is provided on the attached Figure A2. Since some of the on-site soils were coarse-grained, those pocket penetrometer readings are for informational purposes only, and were not used in our analyses.

A.1.3 Material Classification & Sampling

Representative disturbed (grab) samples of the soils encountered in 2006 were obtained at select intervals within the test pits. A qualified member of CGT's geological staff collected the samples and logged the soils in general accordance with the Visual-Manual Procedure (ASTM D2488). An explanation of this classification system is attached as Figure A2. The grab samples were stored in sealable plastic bags and transported to our soils laboratory for further examination and testing. Our geotechnical staff visually examined all samples in 2006 in order to refine the initial field classifications.

A.1.4 Subsurface Conditions

Subsurface conditions are summarized in Section 2.3 of the geotechnical report. Detailed logs of the explorations are presented on the attached exploration logs, Figures A3 through A10.

A.2.0 LABORATORY TESTING

Laboratory testing was performed on samples collected in the field to refine our initial field classifications and determine in-situ parameters. Laboratory testing conducted on samples collected from test pits within the current development area included the following nine moisture content determinations (ASTM D2216) and two percentage passing the U.S. Standard No. 200 Sieve tests (ASTM D1140). Results of the laboratory tests are shown on the exploration logs.

MAHOGANY RIDGE SUBDIVISION - JOHN DAY, OREGON
Project Number G2005305

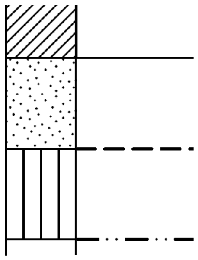
FIGURE A1
Exploration Key

PL —●— LL MC	Atterberg limits (plasticity) test results (ASTM D4318): PL = Plastic Limit, LL = Liquid Limit, and MC= Moisture Content (ASTM D2216)
☐ FINES CONTENT (%)	Percentage passing the U.S. Standard No. 200 Sieve (ASTM D1140)

SAMPLING

GRAB	Grab sample
BULK	Bulk sample
☒ SPT	**Standard Penetration Test** (SPT) consists of driving a 2-inch, outside-diameter, split-spoon sampler into the undisturbed formation with repeated blows of a 140-pound, hammer falling a vertical distance of 30 inches (ASTM D1586). The number of blows (N-value) required to drive the sampler the last 12 inches of an 18-inch sample interval is used to characterize the soil consistency or relative density. The drill rig was equipped with an cat-head or automatic hammer to conduct the SPTs. The observed N-values, hammer efficiency, and N_{60} are noted on the boring logs.
⧗ MC	**Modified California** sampling consists of 3-inch, outside-diameter, split-spoon sampler (ASTM G3550) driven similarly to the SPT sampling method described above. A sampler diameter correction factor of 0.44 is applied to calculate the equivalent SPT N_{60} value per Lacroix and Horn, 1973.
▯ CORE	**Rock Coring** interval
■ SH	**Shelby Tube** is a 3-inch, inner-diameter, thin-walled, steel tube push sampler (ASTM D1587) used to collect relatively undisturbed samples of fine-grained soils.
WDCP	**Wildcat Dynamic Cone Penetrometer** (WDCP) test consists of driving 1.1-inch diameter, steel rods with a 1.4-inch diameter, cone tip into the ground using a 35-pound drop hammer with a 15-inch free-fall height. The number of blows required to drive the steel rods is recorded for each 10 centimeters (3.94 inches) of penetration. The blow count for each interval is then converted to the corresponding SPT N_{60} values.
DCP	**Dynamic Cone Penetrometer** (DCP) test consists of driving a 20-millimeter diameter, hardened steel cone on 16-millimeter diameter steel rods into the ground using a 10-kilogram drop hammer with a 460-millimeter free-fall height. The depth of penetration in millimeters is recorded for each drop of the hammer.
POCKET PEN. (tsf)	**Pocket Penetrometer** test is a hand-held instrument that provides an approximation of the unconfined compressive strength in tons per square foot (tsf) of cohesive, fine-grained soils.

CONTACTS



Observed (measured) contact between soil or rock units.

Inferred (approximate) contact between soil or rock units.

Transitional (gradational) contact between soil or rock units.

ADDITIONAL NOTATIONS

Italics	Notes drilling action or digging effort
{ Braces }	Interpretation of material origin/geologic formation (e.g. { Base Rock } or { Columbia River Basalt })

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All measurements are approximate.

MAHOGANY RIDGE SUBDIVISION - JOHN DAY, OREGON
Project Number G2005305

FIGURE A2
Soil Classification

Classification of Terms and Content

NAME: Group Name and Symbol
Relative Density or Consistency
Color
Moisture Content
Plasticity
Other Constituents
Other: Grain Shape, Approximate Gradation
Organics, Cement, Structure, Odor, etc.
Geologic Name or Formation

Grain Size

		U.S. Standard Sieve
Fines		<#200 (0.075 mm)
Sand	Fine	#200 - #40 (0.425 mm)
	Medium	#40 - #10 (2 mm)
	Coarse	#10 - #4 (4.75 mm)
Gravel	Fine	#4 - 0.75 inch
	Coarse	0.75 inch - 3 inches
Cobbles		3 to 12 inches
Boulders		> 12 inches

Coarse-Grained (Granular) Soils

Relative Density

SPT N_{60}-Value	Density
0 - 4	Very Loose
4 - 10	Loose
10 - 30	Medium Dense
30 - 50	Dense
>50	Very Dense

Minor Constituents

Percent by Volume	Descriptor	Example
0 - 5%	"Trace" as part of soil description	"trace silt"
5 - 15%	"With" as part of group name	"**POORLY GRADED SAND WITH SILT**"
15 - 49%	Modifier to group name	"**SILTY SAND**"

Fine-Grained (Cohesive) Soils

SPT N_{60}-Value	Torvane tsf Shear Strength	Pocket Pen tsf Unconfined	Consistency	Manual Penetration Test
<2	<0.13	<0.25	Very Soft	Thumb penetrates more than 1 inch
2 - 4	0.13 - 0.25	0.25 - 0.50	Soft	Thumb penetrates about 1 inch
4 - 8	0.25 - 0.50	0.50 - 1.00	Medium Stiff	Thumb penetrates about ¼ inch
8 - 15	0.50 - 1.00	1.00 - 2.00	Stiff	Thumb penetrates less than ¼ inch
15 - 30	1.00 - 2.00	2.00 - 4.00	Very Stiff	Readily indented by thumbnail
>30	>2.00	>4.00	Hard	Difficult to indent by thumbnail

Minor Constituents

Percent by Volume	Descriptor	Example
0 - 5%	"Trace" as part of soil description	"trace fine-grained sand"
5 - 15%	"Some" as part of soil description	"some fine-grained sand"
15 - 30%	"With" as part of group name	"**SILT WITH SAND**"
30 - 49%	Modifier to group name	"**SANDY SILT**"

Moisture Content

Dry: Absence of moisture, dusty, dry to the touch

Moist: Leaves moisture on hand

Wet: Visible free water, likely from below water table

	Plasticity	Dry Strength	Dilatancy	Toughness
ML	Non to Low	Non to Low	Slow to Rapid	Low, can't roll
CL	Low to Medium	Medium to High	None to Slow	Medium
MH	Medium to High	Low to Medium	None to Slow	Low to Medium
CH	Medium to High	High to Very High	None	High

Structure

Stratified: Alternating layers of material or color >6 mm thick

Laminated: Alternating layers < 6 mm thick

Fissured: Breaks along definite fracture planes

Slickensided: Striated, polished, or glossy fracture planes

Blocky: Cohesive soil that can be broken down into small angular lumps which resist further breakdown

Lenses: Has small pockets of different soils, note thickness

Homogeneous: Same color and appearance throughout

Visual-Manual Classification

Major Divisions			Group Symbols	Typical Names
Coarse Grained Soils: More than 50% retained on No. 200 sieve	Gravels: 50% or more *retained* on the No. 4 sieve	Clean Gravels	GW	Well-graded gravels and gravel/sand mixtures, little or no fines
			GP	Poorly-graded gravels and gravel/sand mixtures, little or no fines
		Gravels with Fines	GM	Silty gravels, gravel/sand/silt mixtures
			GC	Clayey gravels, gravel/sand/clay mixtures
	Sands: More than 50% *passing* the No. 4 sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly-graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand/silt mixtures
			SC	Clayey sands, sand/clay mixtures
Fine-Grained Soils: 50% or more Passes No. 200 Sieve	Silt and Clays Low Plasticity Fines		ML	Inorganic silts, rock flour, clayey silts
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
			OL	Organic soil of low plasticity
	Silt and Clays High Plasticity Fines		MH	Inorganic silts, clayey silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic soil of medium to high plasticity
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

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References:

ASTM D2487 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)
ASTM D2488 Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)
Terzaghi, K., and Peck, R.B., 1948, Soil Mechanics in Engineering Practice, John Wiley & Sons.

MAHOGANY RIDGE SUBDIVISION

(FORMERLY STRAWBERRY VIEW ESTATES SUBDIVISION)

JOHN DAY, OREGON

Logged by: Ryan Houser

Date Excavated: 06-16-06

Location: See Figure 2

Surface Elevation: 3,169 ft

Depth (ft)	Pocket Penetrometer (tsf)	Sample Number	Sample Type	Moisture Content	Groundwater	United Soil Classification	**Material Description**
	0					OL	Very soft, damp, tan, slightly organic, **SILT TOPSOIL**
1	0						
	0					SM	Loose, damp, tan, **SILTY SAND** Percent passing the U.S. Standard No. 200 Sieve: 47%
2	0.25	S1	100g to 1000g	10			
	0.75						
3	2						
	4					GM	Dense, damp, tan to reddish brown, **SILTY GRAVEL** contained fragments of white tuff. Gravel content and diameter increased with depth.
4	4	S2	100g to 1000g				
5							
6							
7							Minor caving observed at 7 feet bgs
8							
9							
10							
		S3	100g to 1000g				
11							Excavation terminated at 11 feet bgs. Groundwater not encountered. Test pit excavated by Winegar Excavating using a CAT 330B excavator. Test pit loosely backfilled with cuttings upon completion. Ground surface elevation based on topographic map provided by client.
12							
13							
14							
15							
16							
17							

Job No. G2005305 Previously G0602816	Log of Test Pit 1	Figure: A3

MAHOGANY RIDGE SUBDIVISION

(FORMERLY STRAWBERRY VIEW ESTATES SUBDIVISION)

JOHN DAY, OREGON

Logged by: Ryan Houser

Date Excavated: 06-16-06

Location: See Figure 2

Surface Elevation: 3,175 ft

Depth (ft)	Pocket Penetrometer (tsf)	Sample Number	Sample Type	Moisture Content	Groundwater	United Soil Classification	**Material Description**
						OL	Very soft, damp, tan, slightly organic, **SILT TOPSOIL**
1							
2						GM	Dense, damp, tan to reddish brown, **SILTY GRAVEL with cobbles** contained fragments of white tuff and boulders to 2 feet in diameter.
3		S1	100g to 1000g	7			Gravel content and diameter increased with depth.
4							
5							
6							
7							
8		S2	100g to 1000g	16			
9							
10							
							Excavation terminated at 10 feet bgs. Groundwater not encountered. Test pit excavated by Winegar Excavating using a CAT 330B excavator. Test pit loosely backfilled with cuttings upon completion. Ground surface elevation based on topographic map provided by client.
11							
12							
13							
14							
15							
16							
17							

Job No. G2005305
Previously G0602816

Log of Test Pit 2

Figure: A4

MAHOGANY RIDGE SUBDIVISION
(FORMERLY STRAWBERRY VIEW ESTATES SUBDIVISION)
JOHN DAY, OREGON

Logged by: Ryan Houser

Date Excavated: 06-16-06

Location: See Figure 2

Surface Elevation: 3,193 ft

Depth (ft)	Pocket Penetrometer (tsf)	Sample Number	Sample Type	Moisture Content	Groundwater	United Soil Classification	**Material Description**
						OL	Very soft, damp, dark brown, slightly organic, **SILT TOPSOIL**
1						GC	Soft, damp, brown, **CLAYEY GRAVEL**
2		S1	100g to 1000g	8			
3						GC	Dense, damp, tan to reddish brown, **CLAYEY GRAVEL**
4							
5		S2	100g to 1000g				Became hard at 6 feet bgs.
6							Excavation terminated at 6 feet bgs. Groundwater not encountered. Test pit excavated by Winegar Excavating using a CAT 330B excavator. Test pit loosely backfilled with cuttings upon completion. Ground surface elevation based on topographic map provided by client.
7							
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							

Job No. G2005305 **Previously G0602816**	**Log of Test Pit 3**	**Figure: A5**

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(FORMERLY STRAWBERRY VIEW ESTATES SUBDIVISION)
JOHN DAY, OREGON

Logged by: Ryan Houser

Date Excavated: 06-16-06

Location: See Figure 2

Surface Elevation: 3,248 ft

Depth (ft)	Pocket Penetrometer (tsf)	Sample Number	Sample Type	Moisture Content	Groundwater	United Soil Classification	**Material Description**
	1					OL	Soft, damp, dark brown, slightly organic, **SILT TOPSOIL**
1	1.5						
	1.5					ML	Medium stiff, damp, tan, **SANDY SILT**
2	3						
	1						
3	1						
	1.25					ML	Medium stiff, damp, tan, **GRAVELLY SILT** contained cobbles and boulders of white tuff.
4	1	S1	100g to 1000g	23			
5							
6							
7							Minor caving observed at 7 feet bgs
8							
9							
10		S2	100g to 1000g	13			Percent passing the U.S. Standard No. 200 Sieve: 61%
11							
12							
13							
14							
							Excavation terminated at 14 feet bgs. Groundwater not encountered. Test pit excavated by Winegar Excavating using a CAT 330B excavator. Test pit loosely backfilled with cuttings upon completion. Ground surface elevation based on topographic map provided by client.
15							
16							
17							

Job No. G2005305 **Previously G0602816**	**Log of Test Pit 7**	**Figure: A6**

MAHOGANY RIDGE SUBDIVISION
(FORMERLY STRAWBERRY VIEW ESTATES SUBDIVISION)
JOHN DAY, OREGON

Logged by: Ryan Houser

Date Excavated: 06-16-06

Location: See Figure 2

Surface Elevation: 3,218 ft

Depth (ft)	Pocket Penetrometer (tsf)	Sample Number	Sample Type	Moisture Content	Groundwater	United Soil Classification	Material Description
	0.5					OL	Very soft, damp, dark brown, slightly organic, **SILT TOPSOIL**
1	0.5						
	1						
2	3						
	4+	S1	100g to 1000g			ML	Very stiff, dry, tan, **SILT**
3	4+						
	4+						
4	4+						
5							
6							
						GM	Medium dense, damp, tan to brown, **SILTY GRAVEL**
7							
8							
9		S2	100g to 1000g				
10							
		S3	100g to 1000g				Becomes very dense at 11 feet bgs.
11							
							Excavation terminated at 11 feet bgs.
12							Groundwater not encountered.
							Test pit excavated by Winegar Excavating using a CAT 330B excavator.
13							Test pit loosely backfilled with cuttings upon completion.
							Ground surface elevation based on topographic map provided by client.
14							
15							
16							
17							

Job No. G2005305
Previously G0602816

Log of Test Pit 8

Figure: A7

MAHOGANY RIDGE SUBDIVISION

(FORMERLY STRAWBERRY VIEW ESTATES SUBDIVISION)

JOHN DAY, OREGON

Logged by: Ryan Houser

Date Excavated: 06-16-06

Location: See Figure 2

Surface Elevation: 3,256 ft

Depth (ft)	Pocket Penetrometer (tsf)	Sample Number	Sample Type	Moisture Content	Groundwater	United Soil Classification	Material Description
	0.5					OL	Soft, damp, dark brown, slightly organic, **SILT TOPSOIL**
1	1.5						
	1						
2	1					ML	Medium stiff, brown, tan, **SILT**, with trace gravel
	1						
3	2	S1	100g to 1000g	26		GM	Medium dense, damp, tan to brown, **SILTY GRAVEL,** contains trace boulders, density increases with depth. Becomes dense at 4 feet bgs
	2						
4	3						
5							
6							
7							Becomes very dense at 7 feet bgs
8							
9						ML	Medium stiff, dry, brown, tan, **GRAVELLY SILT**
10							
11							Becomes very stiff at 11 feet bgs.
		S2	100g to 1000g				
12							Excavation terminated at 12 feet bgs. Groundwater not encountered. Test pit excavated by Winegar Excavating using a CAT 330B excavator. Test pit loosely backfilled with cuttings upon completion. Ground surface elevation based on topographic map provided by client.
13							
14							
15							
16							
17							

Job No. G2005305
Previously G0602816

Log of Test Pit 9

Figure: A8

MAHOGANY RIDGE SUBDIVISION

(FORMERLY STRAWBERRY VIEW ESTATES SUBDIVISION)

JOHN DAY, OREGON

Logged by: Ryan Houser

Date Excavated: 06-16-06

Location: See Figure 2

Surface Elevation: 3,154 ft

Depth (ft)	Pocket Penetrometer (tsf)	Sample Number	Sample Type	Moisture Content	Groundwater	United Soil Classification	Material Description
	1					OL	Very soft, damp, dark brown, slightly organic, **SILT TOPSOIL**
1	1						
	1.5					ML	Medium stiff, dry to damp, tan, **SILT**, contains abundant cobbles and trace boulders
2	2						
	2						
3	2	S1	100g to 1000g	7			
	3						Becomes stiff at 3.5 feet bgs
4	4						Becomes very stiff at 4 feet bgs
5							
6							
7							
8							
9							
10							
							Excavation terminated at 10 feet bgs. Groundwater not encountered. Test pit excavated by Winegar Excavating using a CAT 330B excavator. Test pit loosely backfilled with cuttings upon completion. Ground surface elevation based on topographic map provided by client.
11							
12							
13							
14							
15							
16							
17							

Job No. G2005305 **Previously G0602816**	**Log of Test Pit 10**	**Figure: A9**

MAHOGANY RIDGE SUBDIVISION

(FORMERLY STRAWBERRY VIEW ESTATES SUBDIVISION)

JOHN DAY, OREGON

Logged by: Ryan Houser

Date Excavated: 06-16-06

Location: See Figure 2

Surface Elevation: 3,184 ft

Depth (ft)	Pocket Penetrometer (tsf)	Sample Number	Sample Type	Moisture Content	Groundwater	United Soil Classification	Material Description
	1					OL	Soft, damp, dark brown, slightly organic, **SILT TOPSOIL,** contains roots and rootlets
1	1						
	1					GM	Medium dense, reddish brown, dry, **CLAYEY GRAVEL**, with trace cobbles
2	0.5	S1	100g to 1000g	7			
	4						Becomes very dense at 2.5 feet bgs
3	4						
	4						
4	4+						
5							
6							
7							Excavation terminated at 7 feet bgs due to practical refusal. Groundwater not encountered. Test pit excavated by Winegar Excavating using a CAT 330B excavator. Test pit loosely backfilled with cuttings upon completion. Ground surface elevation based on topographic map provided by client.
8							
9							
10							
11							
12							
13							
14							
15							
16							
17							

Job No. G2005305
Previously G0602816

Log of Test Pit 17

Figure: A10