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Report of
Geological Hazard Assessment & Preliminary Geotechnical Investigation
Strawberry View 80-Acre Subdivision
East of John Day, South of Highway 26
John Day, Oregon

CGT Project Number G0602826

Prepared for

Blake Weber
John Day Land Development, LLC
PO Box 1292
Sisters, Oregon 97759

August 2, 2006

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Dear Mr. Weber:

Carlson Geotechnical is pleased to submit the results of our Report of Geological Hazard Assessment and Preliminary Geotechnical Investigation for the Strawberry View 80-Acre Subdivision located east of John Day, on the south side of Highway 26, in John Day, Oregon. Two hundred eighty-nine single-family residences are currently planned on the site. CGT performed our work in general accordance with our proposal dated April 5, 2006. You provided written authorization for our services on May 18, 2006.

CGT appreciates the opportunity to work with you on this project. Please call if you have any questions regarding this report.

Sincerely,
Carlson Geotechnical

Ryan T. Houser, CEG
Senior Engineering Geologist

Attachments

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INTRODUCTION

This report presents the results of our geologic hazard assessment and preliminary geotechnical investigation for the Strawberry View Estates Subdivision located on the south side of Highway 26 at the east end of John Day, Oregon. The location of the site is shown on the attached Site Location, Figure 1.

The purpose of our geologic hazard assessment was to identify geologic hazards that may affect the proposed residential development, and to evaluate impacts that the project may have on adjacent properties. The purpose of our geotechnical investigation was to explore subsurface conditions at the site in order to provide preliminary geotechnical recommendations for design and construction of the residences. We understand that the site plan is preliminary, that a grading plan has not been completed, and that the grading plan and site plan revisions are dependant on the findings of this report. Therefore, our geotechnical recommendations should be considered preliminary in nature and are subject to review and revision as plans are developed. Our scope of work included the following:

- Review available literature on geologic hazards for the area. Specific hazards qualitatively addressed by this study include:
    - Landslide potential / slope stability
    - Seismic hazard potential
- Review historical aerial photographs for the site.
- Review available topographic, geologic, and geologic hazard maps for the area.
- Conduct a surface reconnaissance of the project site and surrounding area to identify surface indications of geologic hazards in the area and develop a cross-section of the slope.
- Explore subsurface conditions at the site by observing the excavation of seventeen (17) test pits to depths of up to fourteen (14) feet below the ground surface (bgs) using a Cat 330B excavator provided and operated by Winegar Excavating.
- Classify the materials encountered in the explorations per American Society for Testing and Materials (ASTM) Soil Classification Method D2488. A qualified member of CGT's staff observed the explorations and maintained a detailed log of each test pit.
- Collect representative soil samples from within the test pits in order to perform laboratory testing and to confirm our field classifications.
- Complete sixteen (16) moisture content determinations on select samples from the test pits. The moisture content tests were performed in general accordance with ASTM D2216.

- Complete one (1) Atterberg limits (plasticity) test on a select sample from the test pits. The plasticity test was performed in general accordance with ASTM D4318.
- Complete three (3) percent passing the U.S. Standard No. 200 Sieve analyses on select samples obtained from the test pits. The sieve analyses were performed in general accordance with ASTM C117.
- Provide preliminary recommendations for site preparation, grading and drainage, use of on-site soils for fill, fill type for imported materials, compaction criteria, cut and fill slope criteria, trench excavation and backfill, and wet/dry weather earthwork.
- Provide preliminary geotechnical engineering recommendations for design and construction of shallow spread foundations, including an allowable design bearing pressure and minimum footing depth and width requirements.
- Provide recommendations for subsurface drainage of foundations.
- Provide preliminary geotechnical engineering recommendations for design and construction of concrete floor slabs, including an anticipated value for subgrade modulus, and recommendations for a capillary break and vapor retarder.
- Estimate settlement of footings and floor slabs for the anticipated design loading.
- Provide preliminary design pavement sections, including aggregate base and asphalt concrete thicknesses for driveways and residential streets, based on an assumed California Bearing Ratio (CBR) value for the on-site soils and assumed traffic loading.
- Provide recommendations for the International Building Code (IBC) Site Class, mapped maximum considered earthquake spectral response accelerations, site seismic coefficients, and Seismic Design Category.
- *Qualitatively* evaluate liquefaction potential of the soils encountered within the depths explored.
- Provide a written report summarizing the results of our geologic hazard assessment and preliminary geotechnical investigation.

PROJECT INFORMATION AND SITE DESCRIPTION

We understand that a two- to three-story, wood-framed, single-family residence will be constructed on each of the 289 proposed lots. No detailed structural information has been provided; however, 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.

Based on discussions with you and the plans that you provided, we understand that the development will proceed in stages, beginning with Phase 1 (Figure 3), which will consist of 17 lots in the northeast corner of the site. Phase 2 (Figure 3) would consist of approximately 13 lots immediately west of Phase 1. Phase 3 (Figures 2 and 3) would include the second access road from the northwest corner of site, which would require deep cuts up to 40 feet deep and canyon fills up to 40 feet deep. Later stages of the development are in the preliminary conceptual stages, would likely also include significant cuts and fills, and would proceed up the slope to the southern property boundary ("Future Phases", Figures 2 and 3).

GEOLOGIC HAZARD ASSESSMENT

We reviewed the available geologic hazard literature, and topographic, geologic, and geologic hazard maps of the area. We also performed a field reconnaissance of the site, which consisted of observing site surface and subsurface conditions. Information gathered from the geologic hazard literature, and topographic, geologic, and geologic hazard maps is presented in this report. The results of our field reconnaissance are included in the "Site Conditions" section of this report.

Geologic Hazard Literature Review

We reviewed available literature concerning geologic hazards in the area. We focused our review on hazards associated with earthquakes, including amplification of seismic waves, liquefaction, human induced slope instability, and storm induced landsliding.

Thick sequences of unconsolidated, soft sediments typically amplify the shaking of long period ground motions such as those associated with major earthquakes[1]. Areas underlain by shallow soil profiles are not likely to amplify seismic waves. The amount of amplification is quantified by calculating the expected peak ground acceleration (PGA) of the seismic waves through the soft soil sequence. PGAs are expressed as a fraction of the acceleration of gravity, meaning that a vertical PGA greater than 1.0 g would throw objects into the air. Structures built on a site with anticipated high PGAs are substantially more likely to be damaged during a major earthquake than structures built in areas with low PGAs. Areas that are underlain by unconsolidated sediments are generally susceptible to amplification of seismic waves.

A wide variety of slope and ground failures can occur in response to intense seismic shaking during large magnitude earthquakes. These failures are usually related to the phenomenon of liquefaction, the process by which water-saturated sediment changes from a solid to a liquid state. Since liquefied sediment may not support the overlying ground, or any structure built thereon, a variety of failures may occur including lateral spreading, landslides, ground settlement and cracking, sand boils, oscillation lurching, etc. The conditions necessary for liquefaction to occur are: (1) the presence of poorly consolidated, cohesionless sediment;

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[1] Hofmeister, R., Madin, I., Wang, Y., and Hasenberg, C., 2003. Earthquake and Landslide Hazards Maps and Future Earthquake Damage Estimates, Clackamas County, Oregon: Oregon Department of Geology and Mineral Industries, Open File Report OFR 0-03-10.

(2) saturation of the sediment by groundwater; and (3) an earthquake that produces intense seismic shaking (generally a Richter Magnitude greater than M5.0). In general, older, more consolidated sediment, clayey or gravelly sediment, and sediment above the water table will not liquefy[2]. Field performance data and laboratory tests indicate that liquefaction occurs predominantly in well-sorted, loose to medium dense sand or silty sand[3]. Ground shaking during seismic events can induce landsliding by creating unusual groundwater conditions that raise pore water pressures in slopes, thus reducing the over all strength of the slopes.

Landsliding is a common hazard in the Pacific Northwest that can be initiated on marginally stable slopes by human disturbances such as grading, and natural processes including, earthquake shaking, volcanism, deforestation, heavy rainfalls and rapid snow melt. Recent studies indicate that the most common causes for slope failures are intense rainfall and human alteration including the placement of building loads on slopes, excavating slopes, and the infiltration or diversion of storm water run off[4]. For example, excavation into the base of marginally stable slopes may reduce forces resisting failure on those slopes thus causing movement. Adding a structure or fill to the top or mid portion of a slope increases the driving forces on a slope and may contribute to failure. Redirecting water into slopes may exploit existing planes of weakness within those slopes, causing failure.

Landslides within the John Day area are commonly initiated due to high winter rainfall or snowmelt saturating planes of weakness or bedding planes within weathered and soft layers, particularly when planes of weakness dip downslope.

Topographic Maps

Site topography is shown on the John Day 7½ minute quadrangle[5], attached as Figure 1. The site is located on the southern sideslope of the John Day River valley, between the floodplain and the dissected benchlands to the south. The John Day River valley is typically ½-mile across in the vicinity of the site. The valley sideslopes rise about 500 feet above the river within approximately ½ mile of the floodplain to the dissected benchlands. Slopes average approximately 3H:1V (Horizontal to Vertical) along the southern valley sideslopes, and 10H:1V in the dissected benchlands south of the site. The topographic map shows small areas of very steep arcuate-shaped slopes typical of landslides in the immediate vicinity of the site.

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[2] Youd, T.L. and Hoose, S.N., 1978. Historic ground failures in Northern California triggered by earthquakes: U.S. Geological Survey Professional Paper 993, p.117

[3] Seed, H.B., and Idriss, I.M., 1971. Simplified procedure for evaluating soil liquefaction potential: Journal of Soil Mechanics and Foundation Division, proceedings of the American Society of Civil Engineers, v. 97, p. 1249-1273.

[4] Hofmeister *et al.*, 2003. *Ibid.*

[5] United States Geological Survey, John Day 7½-minute Quadrangle, USGS Topographic Map.

Hummocky topography is present on the north side of the John Day River, indicating landslide terrain.

Regional Geology

The site is located within the southern portion of the Blue Mountain Geologic Province in John Day, Oregon. The Blue Mountain Geologic Province began with Permian-, Triassic-, and Jurassic-Period terranes accreting onto the North American Plate during the late Mesozoic Era (approximately 90 to 65 million years before present (Ma)). Following the accretion of these exotic terranes, the majority of Oregon was covered with a shallow, tropical ocean. During the Eocene Epoch (approximately 57 to 37 Ma) and Oligocene Epoch (approximately 37 to 25 Ma), extensive volcanic activity dominated the depositional environment. During this time, the Columbia River Basalts, originating from fissures in eastern Oregon, flowed as far west as the Pacific Ocean through the ancestral Columbia River drainages[6]. During the late Miocene Epoch (approximately 11 to 5 Ma), streams and rivers deposited extensive layers of Rattlesnake Formation sediments over 1,000 feet thick that consisted of sands and gravels, and were punctuated by ash-flow tuffs.

Engineering Geology Maps

Site geology is shown on the attached Figure 4[7], which 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 (Figure 4). 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.

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[6] Orr, Elizabeth L., and Orr, William N., 1999, Geology of Oregon, Fifth Edition: Kendall/Hunt Publishing, pp. 167-202.

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

Earthquake Sources

The site is located in a tectonically active area that may be affected by large subduction zone earthquakes, intra-slab earthquakes, or crustal earthquakes. The Cascadia Subduction Zone is a 680-mile-long zone of active tectonic convergence where oceanic crust of the Juan De Fuca Plate is subducting beneath the North American continent at a rate of four cm/year[8]. The inferred seismogenic (earthquake producing) portion of the plate interface is roughly coincident with the Oregon coastline. Intra-slab earthquakes occur within the subducting Juan De Fuca Plate where curvature of the plate due to subduction beneath the North American continent causes normal faulting within the plate, thus producing earthquakes. This seismogenic zone is located roughly 30 miles below the Oregon Coast Range.

Faults mapped in the immediate vicinity of the site include the John Day Fault Zone, which consists of the John Day Fault, the Beech Creek Fault, and numerous unnamed faults.

The John Day Fault Zone is an ancient high-angle south-dipping reverse fault at the base of the Strawberry Mountains. The John Day River valley is located approximately along the John Day Fault. The site is located immediately south of the fault. The John Day Fault Zone experienced significant displacement during the uplift of the Strawberry Mountains. Activity on the John Day Fault over the last 6 million year has been minimal[9]. The John Day Fault was possibly active during the Quaternary Period (1.6 million years)[10], but the probability of activity of the John Day Fault Zone impacting the site is considered very low.

Geomorphological evidence suggests that the orientations of the drainages south of the John Day River in the John Day area and on the project site are controlled by faults related to the John Day Fault Zone. Displacement along these inferred faults appears to be minimal, and the probability of activity on these faults resulting in a damaging earthquake is considered extremely remote.

Site Surface Conditions

The northern property boundary was typically located approximately 200 to 400 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 two churches (Figure 5, Photograph 1). 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

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[8] DeMets, C., Gordon, R.G., Argus, D.F., Stein, S., 1990. Current plate motions: Geophysical Journal International, v. 101, p. 425-478.

[9] Clausen, Matthew P., and Carson, Robert J., 2000. Geomorphic Evolution of the Main Fork of the John Day River, Oregon. Eastern Oregon Science Journal, Volume XVI, Eastern Oregon University.

[10] Weldon, R.J., Fletcher, D.K., Weldon, E.M., Scharer, K.M., and McCrory, P.A., 2003. An Update of Quaternary Faults of Central and Eastern Oregon. U.S. Geological Survey Open-File Report 02-301.

of the site were undeveloped large parcels used primarily as rangeland. The southern property boundary was located on the John Day City limit.

The site itself was vacant at the time of our investigation. Previous work on the site included grading associated with a road along the northern property line (See Figures 2 and 3, and Figure 5, Photograph 3), which reportedly once connected with Hillcrest Avenue. This connection was disrupted by offsite grading (Figure 6, Photograph 6) and the construction of a residence located on Tax Lot TL 2001 (Figure 2).

The site was located on a north-facing slope dissected by three parallel, north-trending drainages, which may run along ancient fault lines associated with the John Day Fault Zone (see Earthquake Sources section above for further discussion). The largest of these drainages was located near the center of the site, and had a small stream flowing at the time of the investigation (Figure 5, Photograph 2). A culvert had been installed under the graded roadbed along the northern property boundary.

Typical site gradients ranged from approximately 2H:1V to 4H:1V along the majority of the site. Nearly vertical cuts associated with Hillcrest Avenue were located in the northwest corner of the site (Figure 6, Photograph 5). These cuts exposed hard Rattlesnake Formation consolidated conglomerate. The conglomerate became less consolidated to the east, and did not maintain the vertical cuts (Figure 6, Photograph 6). Nearly vertical natural cliffs were located along the southern portion of the property where the welded Rattlesnake Formation tuffs make up the "rimrock" (Figure 6, Photograph 7) of the John Day River valley. 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 3,550 feet MSL near the southwest corner of the site. Site topography is shown on the Site Plan, attached as Figures 2 and 3.

Areas of instability and potential landslides were noted throughout the site. These slide areas appeared to be surficial instabilities. No large-scale slides or headscarps were observed during the site work or were identified during the subsurface exploration. However, we understand that the Oregon Department of Transportation (ODOT) experienced a landslide on their property located several hundered feet east of the northeast corner of the site. We understand that this slide was activated by excavation at the base of the slope. The depth of the slide plain is not known by CGT. The offsite slide was stabilized using a buttress fill (Figure 5, Photograph 4).

Vegetation on the site typically consisted of scattered trees and grasses.

Site Subsurface Conditions

Field Exploration

Seventeen test pits (TP-1 through TP-17) were excavated at the site on June 16, 2006, to depths of up to fourteen feet bgs using a Cat 330B excavator provided and operated by Winegar Excavating. The approximate test pit locations are shown on the attached Site Plan, Figures 2 and 3. The test pits were located in the field using approximate measurements from existing site features shown on the attached Site Plan (Figures 2 and 3), Global Positioning System coordinates, and surveyor's notes. A member of CGT's staff logged the soils observed within the test pits in general accordance with the Unified Soil Classification System (USCS), and collected representative samples of the materials encountered. CGT has provided an explanation of the USCS on the attached Soil Classification Criteria and Terminology, Figure 7. Our laboratory staff visually examined all samples returned to our laboratory in order to refine the field classifications. Logs of the test pits are presented on the attached Figures 8 through 24. Elevations shown on the logs were interpreted from the topographic map provided, and should be considered approximate. Results of the laboratory tests are shown on the attached logs.

Pocket penetrometer readings were taken within the upper 4 feet of many of the test pits in order to aid in characterizing the consistency of the soils encountered. The pocket penetrometer is a hand-held instrument that provides an approximation of the unconfined compressive strength of fine-grained soils. The correlation between pocket penetrometer readings and the consistency of fine-grained soils is provided on the attached Figure 7.

Subsurface Materials

The materials encountered within the test pits consisted of highly variable sediments of the Rattlesnake Formation. These sediments typically consisted of gravels with varying amounts of silt and clay. For discussion purposes only, the test pits have been assigned types based on similar material characteristics. These type groupings do not necessarily represent the same depositional layer within the Rattlesnake Formation, and are used for organizational purposes only.

The upper 1 to 2½ feet of material encountered within the test pits consisted of silt to gravelly silt topsoil (OL). The silt to gravelly silt topsoil was typically very soft to medium stiff, tan to brown, slightly organic, dry to damp, and contained rootlets.

Type I Test Pits (TP-1, TP-2, TP-3, TP-4, TP-5, TP-8, TP-9, TP-13, and TP-17)

Underlying the silt to gravelly silt topsoil (OL) in the Type I test pits was silty to clayey gravel (GM - GC). The silty to clayey gravel was typically medium dense to very dense, tan to reddish brown, dry to damp, and contained a variable amount of cobbles and boulders up to 24 inches in diameter. Test pit TP-3 contained a two-foot thick layer of softer clayey gravel (GC) over dense clayey gravel, which was encountered at a depth of 3 feet bgs. The dense clayey to silty gravel in test pits TP-1 and TP-5 was overlain by a 1½- to 2-foot thick layer of loose/soft, damp, tan silt (ML) to silty sand (SM). The medium dense silty gravel within test pit TP-8 was overlain by very stiff, dry, tan silt (ML). The gravelly silt within test pit TP-9 was overlain by approximately 6 feet of silty gravel similar to that described in Type II, and a 1-foot-thick layer of medium, brown, stiff silt (ML) overlay the silty gravel (GM). Within the Type I test pits, the silty to clayey gravel was encountered to the maximum depths explored (2½ to 10 feet bgs). Practical refusal was reached with the excavator in the very dense gravels in test pits TP-4 and TP-17.

Type II Test Pits (TP-7, TP-14, and TP-16)

Underlying the silt to gravelly silt topsoil (OL) in the Type II test pits was gravelly silt (ML) to gravelly clay (CL). The gravelly silt to gravelly clay was typically medium stiff to very stiff, dry, and tan to brown. The gravelly clay encountered in test pit TP-16 contained white caliche deposits (hardpan) at depths from 2½ to 6 feet bgs. The gravelly silt in test pit TP-7 was overlain by a 2-foot-thick layer of medium dense sandy silt (ML). The gravelly silt to gravelly clay was encountered to the total depths explored, 11 to 14 feet, in the Type II test pits.

Type III Test Pits (TP-6, TP-10, TP-11, TP-12, and TP-15)

Underlying the silt to gravelly silt topsoil (OL) in the Type III test pits was silt to sandy silt (ML), which was typically medium stiff to very stiff, dry to damp, tan to reddish brown, and contained gravel and cobbles, and a few boulders in test pit TP-10. The sandy silt graded into silty sand (SM) in test pit TP-11 at a depth of approximately 5 feet, which was dense, damp, and tan. The silt to sandy silt and silty sand were encountered to the total depths explored, 9 to 11 feet, in the Type III test pits.

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 alluvium/conglomerate was 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. However, the sedimentary layers are anticipated to extend laterally for some distance before transitioning to a different sedimentary facies[11]. Discontinuities[12] were observed within the bedrock outcrops on the northwest corner of the site.

In addition to the test pits, we observed two cliff faces on the site. The first was located at the northwest corner of the site along Hillcrest Avenue, and consisted of well consolidated (hard) Rattlesnake Formation conglomerate. The cliff was maintaining a nearly vertical 20 to 25-foot high cut slope (Figure 6, Photograph 5), which had reportedly been cut more than 80 years ago. Houses were built at the base of this cut.

The second observed cliff face was located near the southwest corner of the site (Figure 2, lower left, labeled "Rock Outcrop"), which consisted of the tuffaceous member of the Rattlesnake Formation. The tuff appears to have maintained a natural near vertical slope, which is the result of hard tuff toppling as the weaker underlying sediments are eroded away from the base of the slope. In other words, since the tuff is considerably more resistant to erosion, it will maintain the near vertical slopes, but the face of the cliff will retreat as erosion of the underlying sediments cause the face of the cliff to fall off.

Groundwater

Groundwater was not encountered within the explorations. CGT conducted a review of water well logs published by the Oregon Department of Water Resources[13] 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. We anticipate that groundwater levels will fluctuate due to seasonal and annual variations in precipitation, changes in site utilization, or other factors. In addition, the on-site tuff and fine-grained sediments are conducive

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[11] **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.

[12] **Discontinuity** - Any interruption in sedimentation, whatever its cause or length, usually a manifestation of nondeposition and accompanying erosion; an unconformity.

[13] ORWD, 2006. Water well logs obtained from the Oregon Water Resources Department web site, http://www.wrd.state.or.us/

to low infiltration rates and the formation of perched groundwater tables. We anticipate that cuts made into the drainages will encounter perched groundwater.

GEOLOGIC HAZARD ASSESSMENT FINDINGS

Based on our research, review of available maps, and our field investigation, the site is located in an area prone to landsliding. Landsliding on the site generally consists of surficial failures, and no deep-seated failures were observed on the site. However, larger failures can occur if steep cut slopes are excavated near the toe of the slopes, if slopes are oversteepened, or additional fill or structural loads are applied to marginally stable slopes. Therefore, CGT recommends that grading operations consider all surficial materials as potentially unstable, and grading plans should take into account the degree of welding and/or consolidation, if any, of the Rattlesnake Formation tuff (Trt) and conglomerate (Tr). The sedimentary deposits encountered on the site were semi-consolidated and should be trimmed, as appropriate, to not steeper than 2H:1V (cut slopes).

The primary hazard along the existing cliffs on the site is rockfall. Therefore, CGT recommends, for preliminary planning, that existing slopes in the well consolidated conglomerate (northwest corner of site) be trimmed to no steeper than 1H:1V. We understand from the Site Plan (Figures 2 and 3) that no grading is planned within approximately 400 feet downslope from the rimrock welded tuff, which should offer a degree of protection from rockfall, depending upon the final grading of the slopes below the rimrock. Cut slopes should be provided with benches and appropriate toe-of-slope clearances to reduce the potential for damage from rockfall.

Addition of water to the site through excessive irrigation, infiltration of stormwater or sanitary water is not recommended, as these activities would increase the potential for instability of the slopes. All stormwater runoff and sewage should be conducted offsite for proper disposal.

It should be recognized that some portions of the site were inaccessible at the time of this preliminary investigation. Assuming soil, rock, and groundwater conditions do not vary considerably from those encountered in our explorations, we anticipate that with proper grading and construction control, 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. We conclude that the site is geologically suitable for the proposed development, provided that the recommendations contained within this report are followed.

CONCLUSIONS

Based on the results of our field explorations and analyses, the site can be developed as proposed, provided the following recommendations are incorporated into the design and development.

The silt topsoil (OL), soft silts (ML) and clays (CL), and loose silty sands (SM) should not be relied upon for foundation, floor slab, or pavement support. These developments should be supported by the underlying medium dense to very dense silty sands (SM) and gravels (GM to GC), or on the stiff silts (ML).

The principal geotechnical concern for this project is the need to limit oversteepening slopes. CGT recommends that existing slopes in the well consolidated conglomerate (northwest corner of site) be trimmed to no steeper than 1H:1V. We understand from the Site Plan (Figures 2 and 3) that no grading is planned within approximately 400 feet downslope from the rimrock welded tuff, which should offer a degree of protection from rockfall, depending upon the final grading of the slopes below the rimrock. The remaining sedimentary deposits encountered on the site were semi-consolidated and should be trimmed, as appropriate, to not steeper than 2H:1V (cut slopes). Cut slopes should be provided with benches and appropriate toe-of-slope clearances to reduce the potential for damage from rockfall.

Addition of water to the site through excessive irrigation, infiltration of stormwater or sanitary discharge is not recommended, as these activities would increase the potential for instability of the slopes. All stormwater runoff and sewage should be conducted offsite for proper disposal.

We understand that significant structural fill slopes may be necessary to bring the site to grade, particularly within the deep drainages. Water drainage should be maintained through these structural fills using culverts and drains built into the embankments, as needed. Due to the steep nature of the site, all fills placed on the site should be considered structural fills and should be keyed, benched, and compacted appropriately.

The following paragraphs present preliminary geotechnical recommendations for design and construction of the proposed subdivision. We understand that the grading plan is currently undergoing revision, therefore our recommendations should be considered preliminary in nature.

PRELIMINARY RECOMMENDATIONS

The recommendations presented in this report are based on the information provided to us, results of the field investigation, laboratory data, and professional judgment. CGT has observed only a small portion of the pertinent soil and groundwater conditions. The recommendations are based on the assumptions that the soil conditions do not deviate appreciably from those found during the field investigation. CGT strongly recommends that additional exploration be conducted as construction and access allows, in order to identify potentially problematic areas. If the design or location of the proposed development changes, or if variations or undesirable geotechnical conditions are encountered during site development, CGT should be consulted for further recommendations. CGT should review grading plans for each phase of the development, as grading plans are completed.

Site Preparation

Surface vegetation and organic topsoil should be removed from proposed building, structural fill, and pavement locations, and for a 5-foot-margin around such locations. Based on the results of our field explorations, the depth of surface vegetation and organic topsoil stripping within proposed building and pavement locations will be on the order of approximately 1 to 2½ feet. A geotechnical representative from CGT should provide recommendations for actual stripping depths based on observations during site stripping. Stripped surface vegetation and organic topsoil should be transported off-site for disposal, or stockpiled for later use in landscaped areas. Grubbing of trees should include the removal of the root mass, and roots greater than ½-inch in diameter. Grubbed material should be transported off-site for disposal.

After site preparation as recommended above, and prior to excavation for footings or placement of fill materials, a representative from CGT should observe a proof-roll of the exposed subgrade soils in order to identify areas of excessive yielding. If areas of soft soil or excessive yielding are identified, the affected material should be overexcavated to firm, stable subgrade, and replaced with compacted materials as recommended for structural fill.

Silt fences, hay bales, buffer zones of natural growth, sedimentation ponds, and granular haul roads should be used as required to reduce sediment transport during construction to acceptable levels. Measures to reduce erosion should be implemented in general accordance with State of Oregon Administrative Rules 340-41-006 and 340-41-455, and Grant County regulations regarding erosion control.

Wet Weather Considerations

The on-site, silt to sandy silt (ML), silty sand (SM), gravelly clay to gravelly silt (CL – ML), and silty to clayey gravel (GM – GC) have a high percentage of fines and are highly susceptible to disturbance during wet weather. Trafficability of these soils during wet weather will 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. Care should be taken to minimize disturbance of these soils, which may be disturbed by repeated or heavy construction traffic, or by vibratory compaction.

For construction that occurs during wet weather, the site preparation activities may need to be accomplished using track-mounted equipment, loading removed material into trucks supported on granular haul roads, or other methods to limit soil disturbance. A qualified geotechnical engineer should evaluate the subgrade during excavation by probing rather than proofrolling.

Soils that have been disturbed during site preparation activities, or soft or loose areas identified during probing, should be overexcavated to firm, stable subgrade, and replaced with structural fill.

Haul roads subjected to repeated or heavy construction traffic will require a minimum of 18 inches of imported granular material. Twelve inches of imported granular material should be sufficient for light staging areas. The imported granular material should consist of crushed rock that is well-graded between coarse and fine, contains no organic matter, debris, or particles larger than 4 inches, and has less than 5 percent material by weight passing the U.S. Standard No. 200 Sieve. The imported granular material should be placed in one lift over the prepared, undisturbed subgrade, and compacted using a smooth-drum, non-vibratory roller.

CGT recommends that a geotextile filter fabric be placed as a barrier between the subgrade and imported fill in areas of repeated construction traffic. The geotextile filter fabric should have a minimum Mullen burst strength of 250 pounds per square inch for puncture resistance, and an apparent opening size (AOS) between the U.S. Standard No. 70 and No. 100 Sieves.

Structural Fill

Due to the potential for slope instability on the site, any fill placed on the site should be treated as structural fill. Structural fill placed on slopes steeper than 5H:1V (horizontal:vertical) should be benched and keyed in general accordance with the attached Typical Keyway, Bench, and Fill Slope Detail, Figure 25.

On-Site Materials

The silt topsoil (OL) is not suitable for reuse as structural fill. Excavated silt topsoil should be removed from the site or stockpiled for later use in landscaped areas.

Use of the on-site silt to sandy silt (ML), silty sand (SM), gravelly clay to gravelly silt (CL – ML), and silty to clayey gravel (GM – GC), or fine-grained imported material as structural fill may be difficult because fine-grained soils are sensitive to small changes in moisture content and are difficult, if not impossible, to adequately compact during wet weather. If the on-site soils are reused as fill, they should be free of organic matter, debris, and particles larger than 1½ inches. When used as structural fill, the silt to sandy silt (ML), silty sand (SM), gravelly clay to gravelly silt (CL – ML), and silty to clayey gravel (GM – GC) should be placed in lifts with a maximum thickness of about 8 inches, and compacted to not less than 92 percent of the materials maximum dry density, as determined in general accordance with ASTM D1557. If the on-site soils cannot be properly conditioned, CGT recommends using imported granular material for structural fill.

Imported Granular Material

Imported granular material 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 1½ inches, and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. The percentage of fines can be increased to 12 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. Granular fill material should be placed in lifts with a maximum thickness of 12 inches, and compacted to not less than 95 percent of the materials maximum dry density, as determined in general accordance with ASTM D1557.

Shallow Foundations

Shallow spread footings for residences on the site should bear on the medium stiff to very stiff silt to sandy silt (ML), dense silty sand (SM), medium stiff to very stiff gravelly clay to gravelly silt (CL – ML), medium dense to very dense silty to clayey gravel (GM – GC), or on structural fill that is properly placed and compacted on these materials during construction. The medium stiff to very stiff silt to sandy silt (ML) was encountered at depths ranging from 1 to 6 feet bgs within the test pits. The dense silty sand (SM) was encountered at a depth of 5 feet bgs in the test pits. The medium stiff to very stiff gravelly clay to gravelly silt (CL – ML) was encountered at depths ranging from 2 to 9 feet bgs within the test pits. The medium dense to very dense silty to clayey gravel (GM – GC) was encountered at depths ranging from 1 to 6 feet bgs within the test pits.

If soft or otherwise unsuitable soils are encountered, they should be overexcavated as recommended by the CGT geotechnical engineer or engineering geologist. The resulting overexcavation should be brought back to grade with granular structural fill. 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 overexcavation.

CGT recommends that all individual spread footings have a minimum width of 24 inches, and the base of the footings be founded at least 18 inches below the lowest adjacent grade. Continuous wall footings should have a minimum width of 15 inches for residences up to two stories high. Continuous footing widths should be increased to 18 inches for structures up to three stories high. All footings should be founded a minimum of 18 inches below the lowest adjacent grade. Excavations near footings should not extend within a 1H:1V (horizontal:vertical) plane projected out and down from the outside, bottom edge of the footings.

Bearing Pressure and Settlement

In most cases, the minimum widths presented above will govern footing sizes. Regardless, footings founded as recommended should be proportioned for a maximum allowable soil bearing pressure of 2,500 psf for footings founded directly on the native medium stiff to very stiff silt to sandy silt (ML), dense silty sand (SM), medium stiff to very stiff gravelly clay to gravelly silt (CL – ML), medium dense to very dense silty to clayey gravel (GM – GC), or on structural fill which is properly placed and compacted on these materials during construction. This bearing pressure is a net bearing pressure, and applies to the total of dead and long-term live loads, and may be increased by ⅓ when considering seismic or wind loads. If higher soil bearing pressure values are required, we would be pleased to discuss our excavation and gravel backfill requirements.

For the recommended design bearing pressure, total settlement of footings is anticipated to be less than 1 inch. Differential settlements between adjacent load bearing walls and between adjacent columns should not exceed about ½-inch.

Lateral Capacity

CGT recommends using a passive earth pressure of 250 pounds per cubic foot (pcf) for design for footings confined by the medium stiff to very stiff silt to sandy silt (ML), dense silty sand (SM), medium stiff to very stiff gravelly clay to gravelly silt (CL – ML), medium dense to very dense silty to clayey gravel (GM – GC), or structural fill that is properly placed and compacted during construction. The recommended earth pressure was evaluated 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 these capacities, concrete must be poured neat in excavations or the footing backfill must be heavily compacted, the adjacent grade must be level, and the static ground water level must remain below the base of the footings throughout the year. 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 medium stiff to very stiff silt to sandy silt (ML), dense silty sand (SM), medium stiff to very stiff gravelly clay to gravelly silt (CL – ML), medium dense to very dense silty to clayey gravel (GM – GC), or structural fill that is properly placed and compacted during construction.

Drainage

CGT recommends placing foundation drains at the base elevations of the footings on the outside of the footings. Foundation drains should consist of a 4-inch-diameter, perforated, drainpipe wrapped with a geotextile filter fabric. The drains should be backfilled with a minimum of 2 cubic foot per foot of open graded drain rock, which should be encased in a geotextile filter fabric in order to provide separation from the surrounding fine-grained soils. CGT should be contacted to observe the drain prior to backfilling.

Floor Slabs

Satisfactory subgrade support for floor slabs constructed on grade, supporting up to 100 psf area loading, can be obtained from the medium stiff to very stiff silt to sandy silt (ML), dense silty sand (SM), medium stiff to very stiff gravelly clay to gravelly silt (CL – ML), medium dense to very dense silty to clayey gravel (GM – GC), or on structural fill that is properly placed and compacted during construction. The medium stiff to very stiff silt to sandy silt (ML) was encoutered at depths ranging from 1 to 6 feet bgs within the test pits. The dense silty sand (SM) was encountered at a depth of 5 feet bgs in the test pits. The medium stiff to very stiff gravelly clay to gravelly silt (CL – ML) was encountered at depths ranging from 2 to 9 feet bgs within the test pits. The medium dense to very dense silty to clayey gravel (GM – GC) was encountered at depths ranging from 1 to 6 feet bgs within the test pits.

A minimum 6-inch-thick layer of crushed rock base, compacted to not less than 92 percent of the materials maximum dry density, as determined in general accordance with ASTM D1557, should be placed over the prepared subgrade to provide a more uniform surface for placing concrete, and supporting the slab. With a minimum thickness (6 inches) of crushed rock base, a subgrade modulus of 150 pounds per cubic inch may be used for the design of the floor slab supported as described above. Floor slabs constructed as recommended will likely settle less than ½-inch. CGT recommends that slabs be jointed around columns and walls to permit slabs and foundations to settle differentially.

Base rock material placed directly below floor slabs should be ¾-inch maximum or less. The surface of the base rock should be choked with sand just prior to concrete placement. Choking the base rock surface reduces the lateral restraint on the bottom of the concrete during curing.

Due to the presence of on-site, fine-grained soils, liquid moisture and moisture vapor should be expected at the subgrade surface. A capillary break, consisting of at least 6 inches of crushed rock base having less than 5 percent of the material passing the U.S. Standard No. 200 Sieve, typically provides 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.

Asphalt Pavements

Satisfactory subgrade support for asphalt pavements constructed on grade can be obtained from the native medium stiff to very stiff silt to sandy silt (ML), dense silty sand (SM), medium stiff to very stiff gravelly clay to gravelly silt (CL – ML), medium dense to very dense silty to clayey gravel (GM – GC), or on structural fill that is properly placed and compacted during construction. The medium stiff to very stiff silt to sandy silt (ML) was encountered at depths ranging from 1 to 6 feet bgs within the test pits. The dense silty sand (SM) was encountered at a depth of 5 feet bgs in the test pits. The medium stiff to very stiff gravelly clay to gravelly silt (CL – ML) was encountered at depths ranging from 2 to 9 feet bgs within the test pits. The medium dense to very dense silty to clayey gravel (GM – GC) was encountered at depths ranging from 1 to 6 feet bgs within the test pits. All pavement subgrades should be proofrolled under the observation of the project civil engineer, the project geotechnical engineer, or their representative prior to placement of fill of other roadway materials.

<u>The following pavement sections are provided for preliminary planning purposes.</u> They are generally based on the Asphalt Pavement Association of Oregon (APAO) Asphalt Pavement Design Guide traffic loadings; Level I (Driveways), and Level II (Residential Streets). Additional information regarding traffic loadings or other factors may warrant modifications to these preliminary sections as such information becomes available.

Table 1. Preliminary Recommended Minimum Asphalt Pavement Section

Material	**Residential Street**	**Driveways**
Traffic Loading	Level II: 10K to 50K EAL* 2 to 7 ADTT** over 20 years	Level I: up to 10K EAL* 1 ADTT** over 20 years
Asphalt Pavement Thickness (inches)	3½	2½
Crushed Rock Base Course Thickness (inches)	10***	8***

* EAL = Equivalent Axle Loads

** ADTT = Average Daily Truck Traffic

*** may be reduced by 4 inches if native roadway subgrade consists of gravel (such as GM or GC)

The recommended preliminary asphalt pavement sections also assume good subgrade conditions and that construction will be completed during an extended period of dry weather. Increased base rock sections, and/or a separation geotextile fabric may be required in wet conditions in order to support construction traffic and protect the subgrade. We would be pleased to review and revise these preliminary sections, if needed, once detailed traffic information is available. Such review and revision would be provided for limited additional fee. Alternatively, the project civil engineer may revise these recommendations as part of his service for the project.

Asphalt pavement, and base course material should conform to the most recent State of Oregon Standard Specifications for Highway Construction. Place aggregate base in one lift, and compact to not less than 95 percent of the material's maximum dry density, as determined in general accordance with ASTM D1557 (Modified Proctor). 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).

Drainage Considerations

CGT recommends that subsurface drains be connected to the nearest storm drain or other suitable discharge point. CGT also recommends that paved surfaces and ground near or adjacent to any buildings be sloped to drain away from the building. Surface water from pavements and open spaces should be collected and routed to a suitable discharge point. Runoff from roof and pavement areas should not be directed into the foundation drain system.

Utility Trenches

Utility Trench Excavation

Trench cuts should stand near vertical to depths of approximately 4 feet in the silt to sandy silt (ML), silty sand (SM), gravelly clay to gravelly silt (CL – ML), or silty to clayey gravel (GM – GC), provided no groundwater seepage is observed in the sidewalls. If seepage is encountered that undermines the stability of the trench, or caving of the sidewalls is observed during excavation, the sidewalls should be flattened or shored.

Groundwater was not encountered in the test pits on the site, and is not anticipated to be encountered during site development. However, groundwater may be encountered during grading within the north-trending drainages on the site. Therefore trench dewatering may be required to maintain dry working conditions in utility excavations in these areas. Pumping from sumps located within the trench will likely be effective in removing water resulting from seepage. If perched groundwater is present at the base of utility excavations, CGT recommends placing trench stabilization material at the base of the excavations. Trench stabilization material should consist of 1-foot of well-graded gravel, crushed gravel, or crushed rock 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, and should be placed in one lift and compacted until well-keyed.

While CGT has described certain approaches to the trench excavation, it is the contractor's responsibility to select the excavation and dewatering methods, to monitor the trench excavations for safety, and to provide any shoring required to protect personnel and adjacent improvements. All trench excavations should be developed and monitored in accordance with applicable OSHA and state regulations.

Trench Backfill Material

Trench backfill for the utility pipe base and 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.

Backfill for the pipe base and within the pipe zone should be placed in maximum 12-inch-thick lifts, and compacted to not less than 90 percent of the materials maximum dry density, as determined in general accordance with ASTM D1557, or as recommended by the pipe manufacturer. Backfill above the pipe zone should be placed in maximum 12-inch-thick lifts, and compacted to not less than 92 percent of the materials maximum dry density, as determined in general accordance with ASTM D1557. Trench backfill located within 2 feet of finished subgrade elevation should be placed in maximum 12-inch-thick lifts, and compacted to not less than 95 percent of the materials maximum dry density, as determined in general accordance with ASTM D1557.

Permanent Slopes

For preliminary planning purposes, CGT recommends that existing slopes in the well consolidated conglomerate (northwest corner of site) be trimmed to no steeper than 1H:1V. Steeper slopes may be suitable provided appropriate factors of safety are indicated based on additional geotechnical evaluation, including; deeper explorations, field mapping of jointing and slip surfaces, and slope stability analysis.

The sedimentary deposits encountered on the remainder of the site were semi-consolidated and should be trimmed to not steeper than 2H:1V. Cut slopes should be provided with benches and appropriate toe-of-slope clearances to reduce the potential for damage from rockfall. CGT recommends that permanent fill slopes within the site not exceed 2H:1V. Adjacent on-site and off-site structures should be located with the appropriate toe-of-slope and top-of-slope clearances.

Seismic Design

Based on the results of our subsurface explorations and analyses, the following International Building Code (IBC) design criteria were computed using the 2003 IBC:

Table 2. IBC Design Criteria

IBC Coefficient	Value	IBC Source
Site Class	D	Table 1615.1.1
S_s	0.36	Figure 1615(1)
F_a	1.52	Table 1615.1.2(1)
S_1	0.11	Figure 1615(2)
F_v	2.38	Table 1615.1.2(2)
S_{MS}	0.55	Equation 16-38
S_{M1}	0.26	Equation 16-39
S_{DS}	0.37	Equation 16-40
S_{D1}	0.17	Equation 16-41
Category*	II	Table 1604.5
Seismic Use Group	I	Paragraphs 1616.2.1, 1616.2.2, or 1616.2.3
Seismic Design Category	C	Tables 1616.3(1), and 1616.3(2)

**If this is not correct, please inform us in writing so that changes to our recommendations can be made, if warranted.*

Liquefaction

In general, liquefaction occurs when deposits of loose, saturated soils, generally sands, and sand-silt mixtures, are subjected to strong earthquake shaking. If these deposits cannot drain rapidly, there will be an increase in the pore water pressure. With increasing oscillation, the pore water pressure can increase to 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 reduces to zero, and the soil deposit turns into a liquefied state.

The following parameters are generally used to designate non-liquefiable, fine-grained soils:

1. Fines content (percent passing the U.S. Standard No. 200 Sieve) greater than 80 percent.
2. Clay content (particle size less than 0.005 mm) exceeding 20 percent.
3. Liquid limit greater than 35 percent.
4. Water content less than 90 percent of the liquid limit.

Due to the depth of groundwater below the site, the density/consistency of the materials encountered on the site, and the anticipated depth to hard basalt bedrock under a portion of the site, these on-site soils are considered non-liquefiable.

Slope Instability

The site has relatively low seismic coefficients, but contains significant steep slopes. Therefore, the risk of slope instability due to seismic forces at the site is considered moderate.

Surface Rupture

Although the site is situated in a region of the country known for seismic activity, no known active faults exist on or immediately adjacent to the site. Faults in the immediate vicinity of the site have not been identified as having activity during the Quaternary Period (the last 1.6 million years). Accordingly, the risk of surface rupture due to faulting is considered negligible.

OBSERVATION OF CONSTRUCTION

Satisfactory pavement and earthwork 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. CGT recommends 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.

CGT recommends that site stripping, rough grading, foundation, floor slab, and pavement subgrades, and placement of engineered fill are observed by the project geotechnical engineer or their representative. Because observation is typically performed on an on-call basis, CGT recommends that the earthwork contractor be held contractually responsible for scheduling observation.

LIMITATIONS

Due to the large portions of the site that were not accessible during our field exploration, and since grading plans have not been established for this site, the geotechnical recommendations presented herein must be considered preliminary.

CGT has 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 not intended to be, nor should they be construed as a warranty of subsurface conditions, but are forwarded to assist in the planning and design process.

CGT has 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 explorations. If subsurface conditions vary from those encountered in our site exploration, 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.

This report has been issued with the understanding that it is the responsibility of the owner/developer to ensure that the project designers and contractors implement our recommendations. **When the design has been finalized, CGT recommends that the design and specifications be reviewed by our firm to see that our recommendations have been interpreted and implemented as intended.** If design changes are made, CGT requests that we be retained to review our conclusions and recommendations and to provide a written modification or verification.

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 certain degree of uncertainty. Professional judgments presented in this report are based partly 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, are made. This report is subject to review and should not be relied upon after a period of three (3) years.

◆

CGT appreciates the opportunity to serve as your geotechnical consultant on this project. Please contact us if you have any questions.

Sincerely,
CARLSON GEOTECHNICAL

RYAN T. HOUSER
E1914

15,764
JAN. 21, 1992
WILLIAM M. WEYRAUCH
expires 6-30-08

Ryan T. Houser, CEG
Senior Engineering Geologist

William M. Weyrauch, PE
Senior Geotechnical Engineer

Attachments: Site Location, Figure 1
- Site Plan – West Side, Figure 2
- Site Plan – East Side, Figure 3
- Geologic Map, Figure 4
- Site Photographs, Figures 5 and 6
- Soil Classification Criteria and Terminology, Figure 7
- Test Pit Logs, Figures 8 through 24
- Typical Keyway, Bench, and Fill Slope Detail, Figure 25

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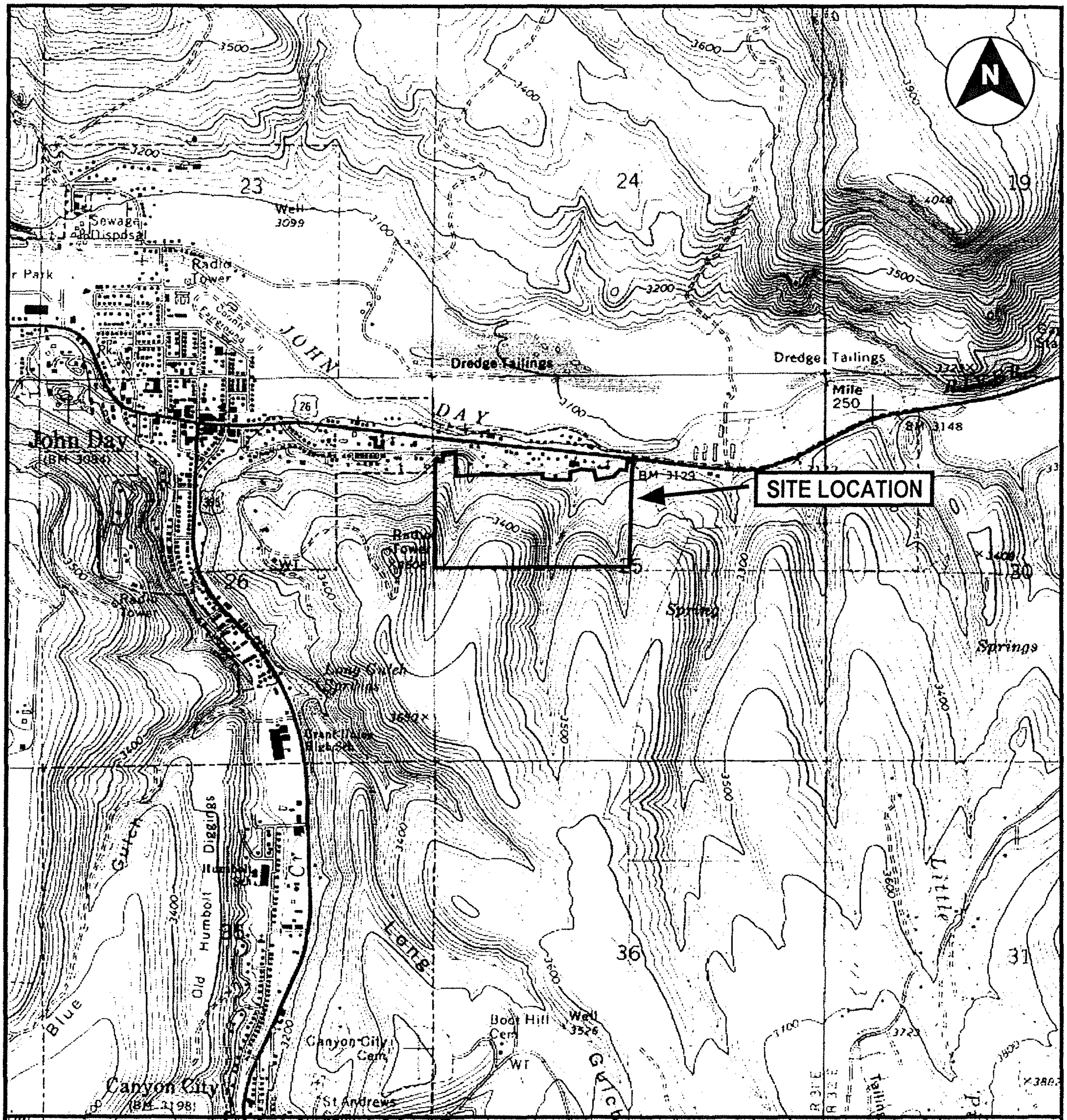
STRAWBERRY VIEW ESTATES, JOHN DAY, OREGON
SITE LOCATION



Map created with TOPO!™ @2000 Wildflower Productions (www.topo.com)
USGS 7.5 Minute Topographic Map Series, John Day, OR Quadrangle.

Scale 1 Inch = 2,000 feet

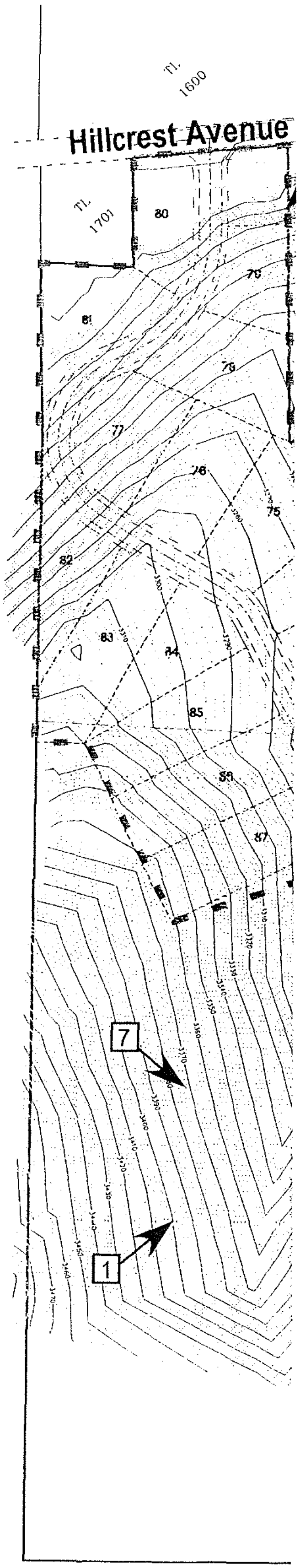
0 2000 4000

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

Carlson Geotechnical
P.O. Box 23814
Tigard, Oregon 97281

CGT Job No. G0602826

FIGURE 1



STRAWBERRY VIEW ESTATES
JOHN DAY, OREGON

SITE PLAN - WEST

JULY 2006 | **FIGURE 2**

CGT JOB NUMBER
G0602826

LEGEND

TP-1 ▚ Approximate location of test pit exploration.

← 1 Number, location, and direction of site photographs shown on Figures 5 & 6

Approximate Scale 1 inch = 200 feet

0 200 400

All locations are approximate.

Site plan provided by the client.
The drawing has been reproduced and modified by CGT staff.

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STRAWBERRY VIEW ESTATES
JOHN DAY, OREGON

SITE PLAN - EAST

JULY 2006	**FIGURE 3**

CGT JOB NUMBER
G0602826

LEGEND

TP-1 ▚	Approximate location of test pit exploration.
←[1]	Number, location, and direction of site photographs shown on Figures 5 & 6

Approximate Scale 1 inch = 200 feet

0 200 400

All locations are approximate.

Site plan provided by the client.
The drawing has been reproduced and modified by CGT staff.

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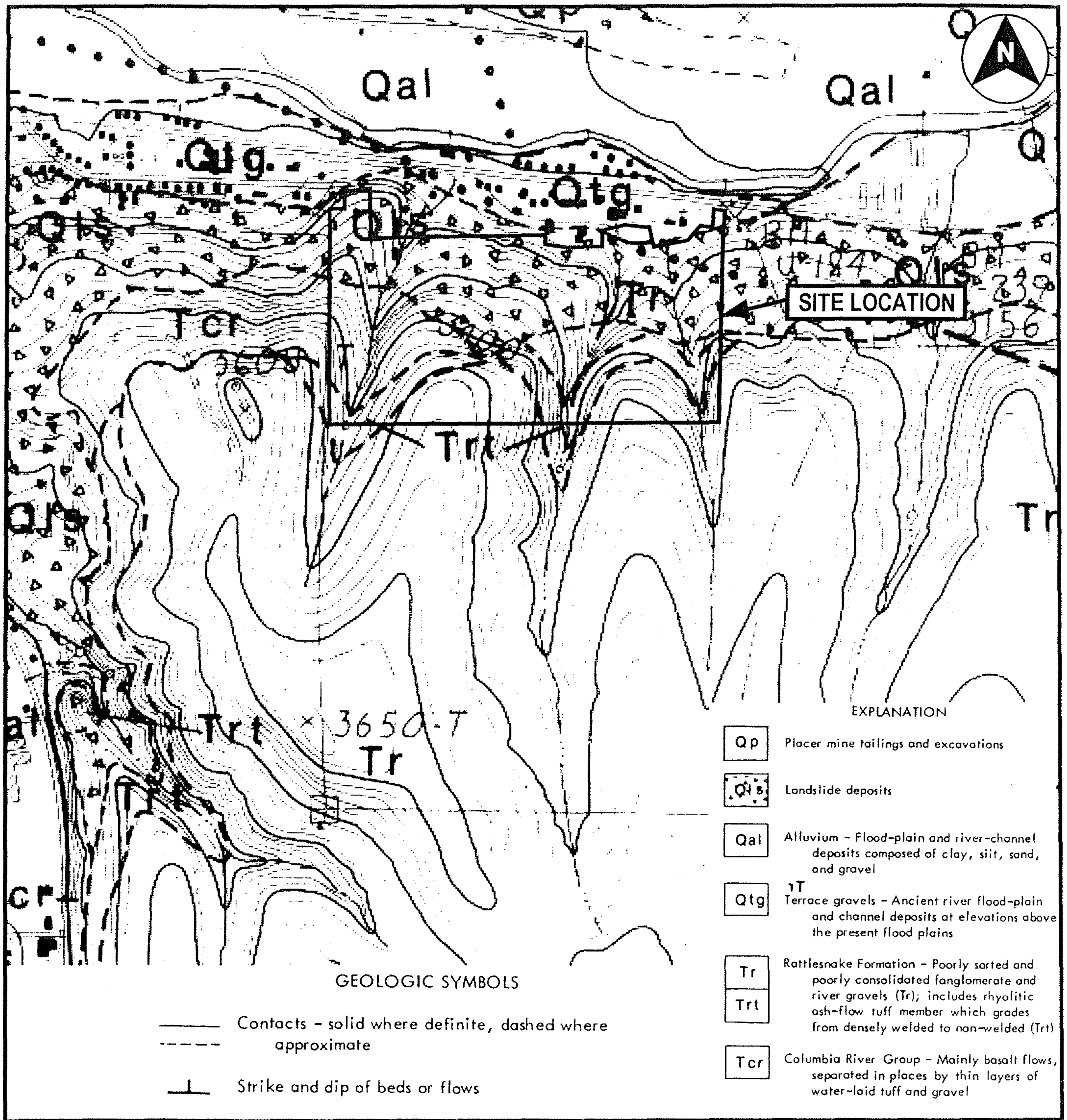
STRAWBERRY VIEW ESTATES, JOHN DAY, OREGON

GEOLOGIC MAP



Map adapted from Schlicker & Brooks, 1975, Engineering Geology of the John Day Area, Grant County, Oregon. State of Oregon Department of Geology and Mineral Industries.

Scale 1 Inch = 1,000 feet

0 1000 2000

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

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FIGURE 4

STRAWBERRY VIEW ESTATES, JOHN DAY, OREGON

SITE PHOTOGRAPHS

Photograph 1: Site with offsite residences in distance. Photo looking northeast.

Photograph 2: Stream located near central portion of site. Photo looking south.

Photograph 3: Road grade along northern property boundary. Photo looking west.

Photograph 4: Buttress fill on offsite ODOT property east of northeast property corner. Photo looking east.

See Figures 2 and 3 for approximate photograph locations and orientations.

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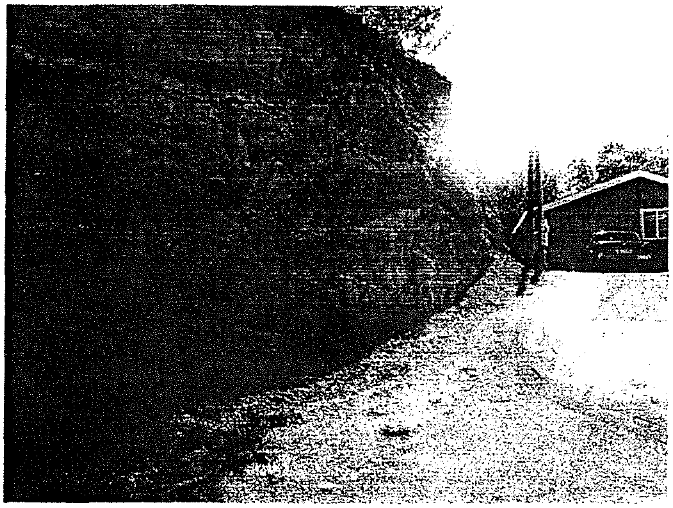
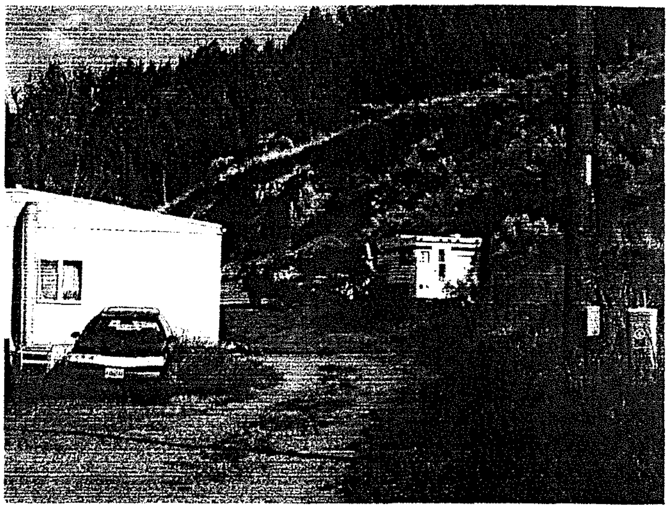
FIGURE 5

STRAWBERRY VIEW ESTATES, JOHN DAY, OREGON

SITE PHOTOGRAPHS



Photograph 5: Northwest corner of site along Hillcrest Avenue. Photo looking west.



Photograph 6: Northwest corner of site along old road gradient. Photo looking southeast.

Photograph 7: Rimrock in southwest corner of site. Photo looking southeast.

See Figures 2 and 3 for approximate photograph locations and orientations.

CARLSON CGT GEOTECHNICAL 503-684-3460	*Carlson Geotechnical* *P.O. Box 23814* *Tigard, Oregon 97281*	CGT Job No. G0602826	FIGURE 6

STRAWBERRY VIEW ESTATES, JOHN DAY, OREGON
SOIL CLASSIFICATION CRITERIA AND TERMINOLOGY

Classification of Terms and Content

NAME : MINOR Constituents (12-50%); MAJOR Constituents (>50%); Slightly (5-12%)
Relative Density or Consistency
Color
Moisture Content
Plasticity
Trace Constituents (0-5%)
Other: Grain Shape, Approximate gradation, Organics, Cement, Structure, Odor....
Geologic Name or Formation: (Fill, Willamette Silt, Till, Alluvium,...)

USCS Grain Size

Fines		<#200 (.075 mm)
Sand	Fine	#200 - #40 (.425 mm)
	Medium	#40 - #10 (2 mm)
	Coarse	#10 - #4 (4.75)
Gravel	Fine	#4 - 0.75 inch
	Coarse	0.75 inch - 3 inches
Cobbles		3 to 12 inches; scattered <15% est., numerous >15% est.
Boulders		> 12 inches

Relative Density or Consistency

Granular Material		Fine-Grained (cohesive) Materials				
SPT N-Value	Density	SPT N-Value	Torvane tst Shear Strength	Pocket Pen tsf Unconfined	Consistency	Manual Penetration Test
		<2	<0.13	>0.25	Very Soft	Easy several inches by fist
0 - 4	VeryLoose	2 - 4	0.13 - 0.25	0.25 - 0.50	Soft	Easy several inches by thumb
4 - 10	Loose	4 - 8	0.25 - 0.50	0.50 - 1.00	Medium Stiff	Moderate several inches by thumb
10 - 30	Medium Dense	8 - 15	0.50 - 1.00	1.00 - 2.00	Stiff	Readily indented by thumb
30 - 50	Dense	15 - 30	1.00 - 2.00	2.00 - 4.00	Very Stiff	Readily indented by thumbnail
>50	Very Dense	>30	>2.00	>4.00	Hard	Difficult by thumbnail

Moisture Content

Dry: Absence of moisture, dusty, dry to the touch
Damp: Some moisture but leaves no moisture on hand
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 Med.	Medium to High	None to Slow	Medium
MH	Med to High	Low to Medium	None to Slow	Low to Medium
CH	Med to High	High to V. High	None	High

Structure

Stratified: Alternating layers of material or color >6 mm thick
Laminated: Alternating layers < 6 mm thick
Fissured: Breaks along definate 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

Unified Soil Classification Chart (Visual-Manual Procedure) (Similar to ASTM Designation D-2488)

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 silt and organic silty clays of low plasticity
	Silt and Clays High Plasticity Fines		MH	Inorganic silts, clayey silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

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CGT Job No. G0602826

Figure 7

STRAWBERRY VIEW 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						
2	0.25	S1	Bag	10		SM	Loose, damp, tan, **SILTY SAND** Percent passing the U.S. Standard No. 200 Sieve: 47%
	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	Bag				
5							
6							
7							Minor caving observed at 7 feet bgs
8							
9							
10							
		S3	Bag				
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. G0602816	Log of Test Pit 1	Figure: 8

STRAWBERRY VIEW 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	■	7			Gravel content and diameter increased with depth.
4							
5							
6							
7							
8		S2	■	16			
9							
10							
11							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.
12							
13							
14							
15							
16							
17							

Job No. G0602816	**Log of Test Pit 2**	**Figure: 9**

STRAWBERRY VIEW 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**
1						OL	Very soft, damp, dark brown, slightly organic, **SILT TOPSOIL**
2 3		S1	▨	8		GC	Soft, damp, brown, **CLAYEY GRAVEL**
4 5 6		S2	▨			GC	Dense, damp, tan to reddish brown, **CLAYEY GRAVEL** Became hard at 6 feet bgs.
7 8 9 10 11 12 13 14 15 16 17							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.

Job No. G0602816	Log of Test Pit 3	Figure: 10

STRAWBERRY VIEW SUBDIVISION
JOHN DAY, OREGON

Logged by: Ryan Houser

Date Excavated: 06-16-06

Location: See Figure 2

Surface Elevation: 3,425 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	Very dense, damp, reddish brown, **SILTY GRAVEL**
3 4 5 6 7 8 9 10 11 12 13 14 15 16 17							Excavation terminated at 2.5 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.

Job No. G0602816	**Log of Test Pit 4**	**Figure: 11**

STRAWBERRY VIEW SUBDIVISION
JOHN DAY, OREGON

Logged by: Ryan Houser

Date Excavated: 06-16-06

Location: See Figure 2

Surface Elevation: 3,375 ft

Depth (ft)	Pocket Penetrometer (tsf)	Sample Number	Sample Type	Moisture Content	Groundwater	United Soil Classification	Material Description
	2					OL	Very soft, damp, dark brown, slightly organic, **SILT TOPSOIL,** contains roots
1	0.25						
	1					ML	Soft, damp, tan, **SILT**
2	2						Percent passing the U.S. Standard No. 200 Sieve: 47%
	2						
3	4	S1	▨	10			
	4+					GM	Dense, damp, tan to reddish brown, **CLAYEY GRAVEL.**
4	4+						
5							
6							Becomes very dense at 6 feet bgs
7							
8		S2	▨				Plastic Limit: 22%; Liquid Limit: 65%; Plasticity Index: 43%
9							
10							Excavation terminated at 9.5 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.
11							
12							
13							
14							
15							
16							
17							

Job No. G0602816	Log of Test Pit 5	Figure: 12

STRAWBERRY VIEW SUBDIVISION
JOHN DAY, OREGON

Logged by: Ryan Houser
Date Excavated: 06-16-06

Location: See Figure 2
Surface Elevation: 3,347 ft

Depth (ft)	Pocket Penetrometer (tsf)	Sample Number	Sample Type	Moisture Content	Groundwater	United Soil Classification	Material Description
	1					OL	Medium stiff, damp, dark brown, slightly organic, **SILT TOPSOIL**
1	1					ML	Medium stiff, damp, tan to reddish brown, **SILT** with trace gravel
	1.5						
2	4						Becomes very stiff at 2 feet bgs.
	4						
3	4						
	4+						
4	4+						
5							
6							
7							
8							
9							
10							
		S1	100g				
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. G0602816 | **Log of Test Pit 6** | **Figure: 13**

STRAWBERRY VIEW 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						
2	3					ML	Medium stiff, damp, tan, **SANDY SILT**
	1						
3	1						
	1.25					ML	Medium stiff, damp, tan, **GRAVELLY SILT** contained cobbles and boulders of white tuff.
4	1	S1	▨	23			
5							
6							
7							Minor caving observed at 7 feet bgs
8							
9							
10		S2	▨	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.
15							
							Test pit loosely backfilled with cuttings upon completion.
16							Ground surface elevation based on topographic map provided by client.
17							

Job No. G0602816	Log of Test Pit 7	Figure: 14

STRAWBERRY VIEW 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	bag			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	bag				
10							
		S3	bag				Becomes very dense at 11 feet bgs.
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. G0602816	Log of Test Pit 8	Figure: 15

STRAWBERRY VIEW 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	0.5					OL	Soft, damp, dark brown, slightly organic, **SILT TOPSOIL**
1	1.5						
1.5	1						
2	1						
2.5	1					ML	Medium stiff, brown, tan, **SILT**, with trace gravel
3	2	S1	bag	26		GM	Medium dense, damp, tan to brown, **SILTY GRAVEL,** contains trace boulders, density increases with depth.
3.5	2						
4	3						Becomes dense at 4 feet bgs
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.
11.5		S2	bag				
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. G0602816	Log of Test Pit 9	Figure: 16

STRAWBERRY VIEW 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						
2	2					ML	Medium stiff, dry to damp, tan, **SILT**, contains abundant cobbles and trace boulders
	2						
3	2	S1	▨	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. G0602816	**Log of Test Pit 10**	**Figure: 17**

STRAWBERRY VIEW SUBDIVISION
JOHN DAY, OREGON

Logged by: Ryan Houser — Date Excavated: 06-16-06

Location: See Figure 2 — Surface Elevation: 3,163 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							
2						ML	Stiff, dry to damp, tan, **SILTY SAND**
3							Sand content increases with depth.
4							Becomes very stiff at 4 feet bgs
5						SM	Dense, dry to damp, tan, **SILTY SAND**
6							
7		S1	▨	14			Percent passing the U.S. Standard No. 200 Sieve: 33%
8							
9							
10							Excavation terminated at 9 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. G0602816	Log of Test Pit 11	Figure: 18

STRAWBERRY VIEW SUBDIVISION
JOHN DAY, OREGON

Logged by: Ryan Houser

Date Excavated: 06-16-06

Location: See Figure 2

Surface Elevation: 3,168 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							
2						ML	Stiff, dry to damp, tan, **SANDY SILT**
3							
4							Becomes very stiff at 4 feet bgs
5							
6							
7							
8							
9							Excavation terminated at 9 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.
10							
11							
12							
13							
14							
15							
16							
17							

Job No. G0602816 | **Log of Test Pit 12** | **Figure: 19**

STRAWBERRY VIEW SUBDIVISION
JOHN DAY, OREGON

Logged by: Ryan Houser
Date Excavated: 06-16-06

Location: See Figure 2
Surface Elevation: 3,546 ft

Depth (ft)	Pocket Penetrometer (tsf)	Sample Number	Sample Type	Moisture Content	Groundwater	United Soil Classification	**Material Description**
1						OL	Dense, damp, brown, slightly organic, **SILTY GRAVEL TOPSOIL**
2 3 4 5 6 7						GM	Dense, damp, tan, **SILTY GRAVEL** clast supported
8 9 10 11 12 13 14 15 16 17							Excavation terminated at 8 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.

Job No. G0602816	**Log of Test Pit 13**	**Figure: 20**

STRAWBERRY VIEW SUBDIVISION
JOHN DAY, OREGON

Logged by: Ryan Houser
Date Excavated: 06-16-06

Location: See Figure 2
Surface Elevation: 3,404 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, **GRAVELLY SILT TOPSOIL**
1	1						
	1.5						
2	1.5	S1	bag			ML	Medium stiff, dry, tan, **GRAVELLY SILT**
	2						
3	4						Becomes very stiff at 3 feet bgs
	4						
4	4						
5							
6							
7							
8							
9							
10							
11							
12							
13							
		S2	bag				
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. G0602816	Log of Test Pit 14	Figure: 21

STRAWBERRY VIEW SUBDIVISION
JOHN DAY, OREGON

Logged by: Ryan Houser
Date Excavated: 06-16-06

Location: See Figure 2
Surface Elevation:3,363 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, **GRAVELLY SILT TOPSOIL,** contains roots and rootlets
1	1						
	2					ML	Medium stiff, dry, tan, **SANDY SILT** with trace gravel
2	2						
	2						
3	4						Becomes very stiff at 3 feet bgs
	3.5						
4	4						
5							
6							
7							
8							
9							
10							
		S1	▨	16			
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. G0602816	Log of Test Pit 15	Figure: 22

STRAWBERRY VIEW SUBDIVISION
JOHN DAY, OREGON

Logged by: Ryan Houser	Date Excavated: 06-16-06
Location: See Figure 2	Surface Elevation: 3,260 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						
2	0.5						
	4						
3	4					CL	Very stiff, brown with white caliche deposits, **GRAVELLY CLAY**
	4						
4	4+	S1	▨	14			
5							
6							No caliche deposits, very stiff.
7							
8		S2	▨	16			
9							
10							
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. G0602816	**Log of Test Pit 16**	**Figure: 23**

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STRAWBERRY VIEW SUBDIVISION
JOHN DAY, OREGON

Logged by: Ryan Houser

Date Excavated: 06-16-06

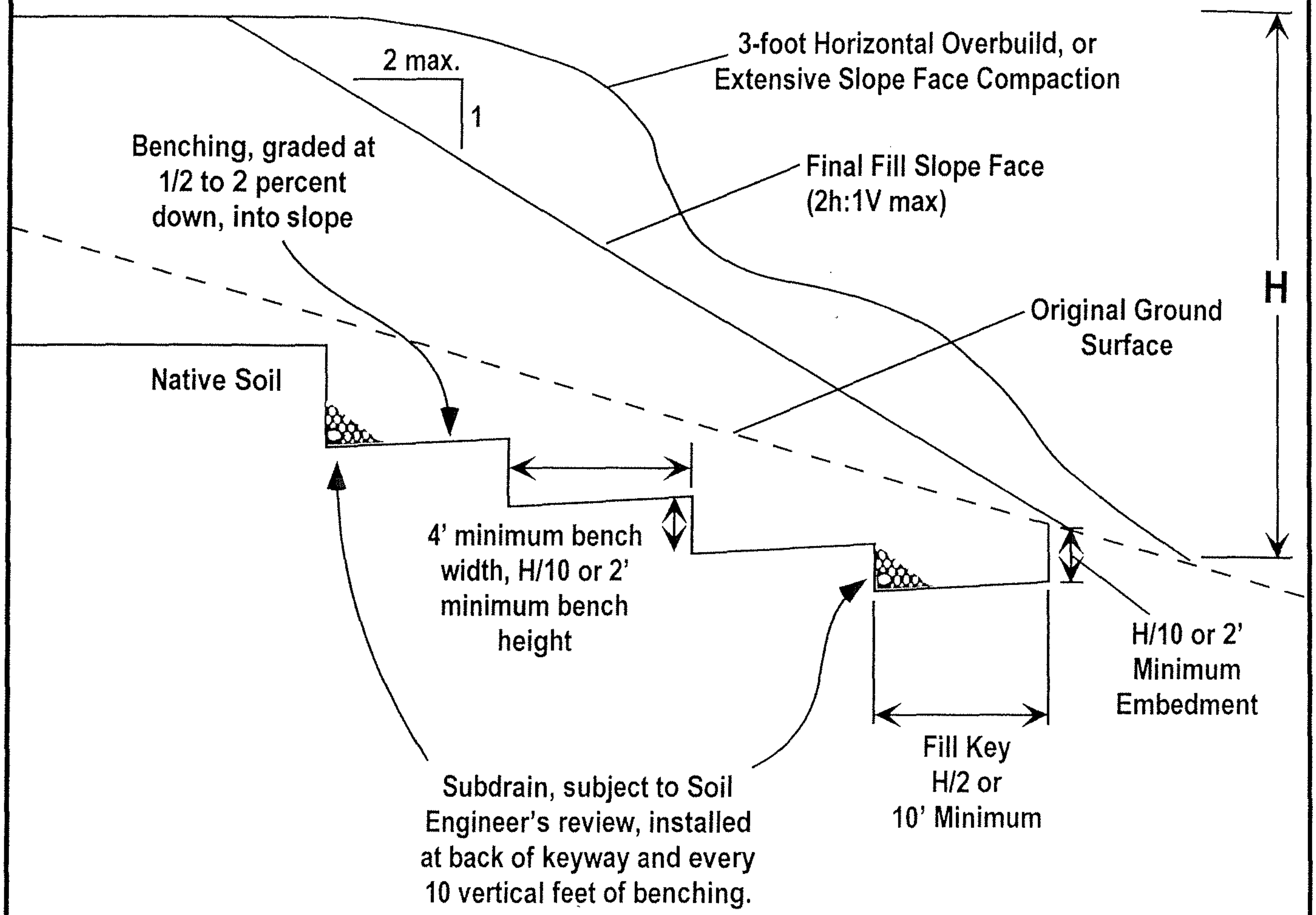
Location: See Figure 2

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	▨	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.
8							Groundwater not encountered.
							Test pit excavated by Winegar Excavating using a CAT 330B excavator.
9							Test pit loosely backfilled with cuttings upon completion.
							Ground surface elevation based on topographic map provided by client.
10							
11							
12							
13							
14							
15							
16							
17							

Job No. G0602816	Log of Test Pit 17	Figure: 24

STRAWBERRY VIEW ESTATES, JOHN DAY, OREGON
TYPICAL KEYWAY, BENCH, AND FILL SLOPE DETAIL



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

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FIGURE 25