Geologic Hazards and Geotechnical Investigation
Tax Lots 7300 and 7303, Map 2-1E-36DD
Oregon City, Oregon

Prepared for:
Robert D. Green, General Contractor
7537 S.E. 116th Avenue
Portland, Oregon 97266-5975

Project #Y174029 June 5, 2017

H.G. Schlicker & Associates, Inc.

H.G. Schlicker & Associates, Inc.
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To: **Robert D. Green, General Contractor**
7537 S.E. 116th Avenue
Portland, Oregon 97266-5975

Subject: **Geologic Hazards and Geotechnical Investigation**
Tax Lots 7300 and 7303, Map 2-1E-36DD
Oregon City, Oregon

Dear Mr. Green:

The accompanying report presents the results of our geologic hazards and geotechnical investigation for the above subject site.

After you have reviewed our report, we would be pleased to discuss the report and to answer any questions you might have.

This opportunity to be of service is sincerely appreciated. If we can be of any further assistance, please contact us.

H.G. SCHLICKER & ASSOCIATES, INC.

J. Douglas Gless, MSc, RG, CEG, LHG
President/Principal Engineering Geologist

JDG:cjh

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Dear Mr. Green:

1.0 Introduction

At your request and authorization, the undersigned representative of H.G. Schlicker and Associates, Inc. (HGSA) visited the subject site on March 24 and April 21, 2017 to complete a geologic hazards and geotechnical investigation of Tax Lots 7300 and 7303, Map 2-1E-36DD in Oregon City, Oregon (Figures 1 and 2; Appendix A). It is our understanding that you plan to construct a single family residential home on the site.

This report addresses the geologic hazards and geotechnics at the site with respect to constructing a home. The scope of our work consisted of site visits, site observations and measurements, subsurface test pit explorations, a slope profile, limited review of the geologic literature, review of topographic maps, lidar and aerial photography, and preparation of this report with our findings, conclusions and recommendations.

2.0 Site Description

The subject site consists of two adjacent rectangular undeveloped lots (Tax Lots 7300 and 7303) totaling approximately 0.22 acres, located on a northwest facing slope southeast of the Willamette River and south of Willamette Falls in the Canemah neighborhood of Oregon City, Oregon (Figures 1 and 2). Together, the subject lots measure approximately 100 feet × 100 feet. The site is bound to its west by Miller Street, to its south by 4th Avenue, to its east by a developed lot, and to its north by an undeveloped property. The site is vegetated with Douglas fir, maple and alder trees, grasses, blackberry, Oregon grape and other brush.

3.0 Geology

The site is mapped in an area of Quaternary landslide deposits which consist of chaotically mixed and deformed masses of rock, colluvium and soil that have moved downslope in one or more events (Madin, 2009). Landslide deposits at the site overlie older volcanic rocks mapped as Miocene Basalt of Gingko, which is a characteristic unit of the Wanapum Basalt, Frenchman Springs Member of the Columbia River Basalt Group (CRBG). The Basalt of Gingko consists of black medium-grained flows with abundant plagioclase phenocrysts and well developed columnar jointing (Madin, 2009). The Columbia River Basalt Group is comprised of flood basalts which were erupted from fissures in eastern Oregon and Washington and western Idaho from approximately 16.5 to 6 million years ago. The CRBG generally consists of reddish-brown to gray-brown weathered, and blue-black unweathered, tholeiitic basalt lava flows with minor interbeds of basalt breccia, ash and baked tuffaceous sediments. These flows are typically massive columnar jointed to close cubic jointed, dense, and with vesiculation near the tops of most flows (Madin, 1990). Miocene Basalt of Gingko is mapped as the primary geologic unit north, west and northwest of the site along the southeast bank of the Willamette River (Madin, 2009).

During our April 21, 2017 site visit we explored the subsurface with three test pits using a Yanmar tracked excavator with 24" bucket. Soils encountered in the test pit explorations were visually classified by a Certified Engineering Geologist from our office according to the Unified Soil Classification System (USCS). Approximate locations of the test pits are shown on Figure 3, and detailed logs are provided in Appendix B. The test pits encountered clayey, sandy silt fill soils with basaltic gravels to depths of approximately 4.5 to 6 feet, underlain by native gravels in a generally clayey, sandy silt matrix. Fills are present due to grading of 4th Avenue and minor grading on site.

3.1 Structures

A potentially active fault named the Bolton fault is located approximately 1.3 miles northeast of the site. This fault is upthrown to the southwest and has several intersecting secondary, northeasterly trending faults to its west. The Bolton fault displaces late Pleistocene flood deposits approximately 11,000 to 14,000 years old, and is believed to have a low probability of being active (Geomatrix, 1995). The Bolton fault is part of the Portland Hills Fault Zone that includes the Oatfield fault located approximately 2.7 miles northeast of the site, the Portland Hills fault, and the East Bank fault. These are northwest trending, normal faults with their downthrown sides to the southwest, northeast, and southwest, respectively and are characterized by dextral strike-slip motion. Data is scarce regarding the age of displacement along the Portland Hills faults, but they are believed to have a potential of generating magnitude 6.6 to 7.1 earthquakes (Geomatrix, 1995; Wong et al., 2000). The Bolton fault is believed to have a potential of generating a magnitude 6.1 to 6.3 earthquake (Wong et al., 2000).

The Concord fault, River Forest fault, and Marylhurst fault all lie north and northwest of the site, roughly paralleling the trend of the Oatfield fault (Beeson et al., 1989; Madin 1990). They appear to be conjugate splays off of the Oatfield fault, and the level of activity on these faults is unknown. They are recognized by offset of well-defined stratigraphy (typically the Columbia River Basalt Group) in limited exposures along the Willamette River (Madin, 1990).

The site lies approximately 7 miles southwest of the Damascus-Tickle Creek Fault Zone (Personius et al., 2003; Geomatrix, 1995). The Damascus-Tickle Creek Fault Zone is believed to be comprised of primarily and typically north-northwest to northwest trending, right-lateral strike-slip to high-angle oblique, en echelon wrench faults, with some internal fault strands which trend northeast and are left-lateral strike-slip to high-angle oblique (Personius, 2003; Geomatrix, 1995). These faults offset the Pliocene-Pleistocene Springwater Formation and Pleistocene Boring Lavas as young as 100,000 years old, and evidence from trench excavations indicate that latest Pleistocene to early Holocene catastrophic flood sediments (Missoula Floods) may be deformed and/or offset, indicating possible recent activity within the zone (Madin, 1994).

The faults within the Damascus-Tickle Creek Fault Zone are considered to be potentially active by Geomatrix (1995), with a probability of 0.5 given for certainty of activity. Slip rates have been estimated at 0.01 mm/year to 0.1 mm/year, with the maximum and minimum slip rates in this range being given equal probability (0.2), and the median rate (0.05 mm/yr) being given the highest probability (0.6) due to the lack of continuity of the fault traces. Individual fault strands average approximately 4 miles long (7 km), with maximum lengths of approximately 6 miles (10 km), and an overall total combined length of approximately 11 miles (17 km).

4.0 Slope Stability and Erosion

The site is located along a moderately sloping hillside overlooking the Willamette River to its northwest. The property slopes moderately to steeply down to the northwest (Figures 3 and 4). DOGAMI's SLIDO-3 landslide mapping shows that the subject site lies within the toe area of a large, northwest facing complex deep seated earth slide (Oregon_City_201), with a similar northwest facing slide (Oregon_City_200) mapped adjacent to it and approximately 150 feet east of the subject site (Burns and Madin, 2006; Burns and Mickelson, 2010). Based on the mapping, the headscarps of these 2 landslides join along the area of South End Road approximately 1,000 feet southeast of the site. No other significant landslide features are mapped within a ¼ mile radius of the subject site (Burns and Mickelson, 2010; Burns and Watzig, 2014). Stormwater at the site generally flows north-northwesterly and towards the river. The site is mapped in an area of very high landslide susceptibility ("very high" means an existing landslide in this rating system) based on the DOGAMI methodology (Burns, Mickelson, and Madin, 2016). Ground movement has been historically reported in the Canemah area, some of it within the last few decades.

5.0 Regional Seismic Hazards

The historical earthquake record for the Willamette Valley and Portland basin is dominated by small to moderate earthquakes and an appreciable lack of significantly large earthquakes. Most of the earthquakes which have occurred within the Portland metropolitan area and surrounding areas could not be associated with any known faults. There have been at least 17 earthquake events of Richter magnitude (M) 4 or larger which have occurred in the Portland area in historic time, of which 6 of these events have been of magnitude (M) 5.0 and greater. The largest historic earthquakes within the Willamette Valley and adjacent areas have been the 1993 Scotts Mills earthquakes (M 5.6) northeast of Salem, Oregon, the 1964 Vancouver, Washington earthquake (M 5.3), the 1962 Portland earthquake (M 5.5), and the 1961 earthquake (M 5.0) northwest of Portland. There are at least three crustal faults beneath the Portland metropolitan area which researchers believe could generate earthquakes of M 6.5 or larger (Wong et al., 2000). These larger earthquakes may occur at an average interval of approximately 1,000 years (Bott and Wong, 1993).

Abundant evidence indicates that a series of geologically recent large earthquakes related to the Cascadia Subduction Zone have occurred along the coastline of the Pacific Northwest. Evidence suggests that more than 40 great earthquakes of magnitude 8 and larger have struck western Oregon during the last 10,000 years. The calculated odds that a Cascadia earthquake will occur in the next 50 years range from 7–15 percent for a great earthquake affecting the entire Pacific Northwest, to about a 37 percent chance that the southern end of the Cascadia Subduction Zone will produce a major earthquake in the next 50 years (OSSPAC, 2013; OSU News and Research Communications, 2010; Goldfinger et al., 2012). Evidence suggests the last major earthquake occurred on January 26, 1700 and may have been of magnitude 8.9 to 9.0 (Clague et al., 2000). Locally, these great coastal earthquakes would likely have about the same effects as a possible local large earthquake.

As noted above, faults within the Portland Fault Zone have a potential of generating magnitude 6.5 to 7.1 earthquakes (Geomatrix, 1995; Wong et al., 2000). Based on mapping from Clackamas County's CMap online GIS, the subject site lies in an area designated as having relatively higher hazards associated with earthquakes. This mapping was derived from unpublished maps produced by DOGAMI during 2001-2003 for Clackamas County, with methodology similar to that used for the 1997 Relative Earthquake Hazard Map of the Portland Metro Region (Mabey et al., 1997). The degree of relative hazard was based on the factors of ground motion amplification, liquefaction, and slope instability.

6.0 Flooding Hazards

Based on the 2008 Flood Insurance Rate Map (FIRM, Panel #41005C0276D) the site lies in an area rated as Zone X which is defined as an area determined to be outside the 0.2% annual chance floodplain.

7.0 Conclusions and Recommendations

The main engineering geologic concerns at the site are:

1. The site is located in landslide terrain, on a hillside which has been formed by downcutting and lateral erosion related to the catastrophic Missoula Floods (Bretz floods) and subsequent erosion.

2. Foundations will need to be footings stepped up the slope, or grade beams supported on deep foundations such as augered or driven pile. Please note that prior to design of a deep foundation system, the site will need to be drilled to obtain deep subsurface information.

3. There is an inherent risk of earthquakes in Oregon which could cause harm and damage structures, and the subject site is located in the vicinity of Portland area seismically active faulting. These risks must be accepted by the owners, future owners, developers, and residents of the site.

Provided the following recommendations are adhered to during design and construction, we expect that this development will not negatively affect slope stability on the subject lot or adjacent lots.

As with all hillside sites lying in ancient landslide terrain, the subject property has some inherent risk of ground movement; however, based on our research, site reconnaissance observations, subsurface exploration and analysis, the risk of landslide related ground movement is typical of the Canemah area and no evidence of recent ground movement was observed.

7.1 Site Preparation

From an engineering geologic perspective the proposed home location on the southern part of the property is reasonable provided that the recommendations below are adhered to. A stepped foundation design would be appropriate for the site.

Building loads may be supported on individual and continuous spread footings bearing in undisturbed, native, non-organic, firm soils or properly designed and compacted structural fill placed on these soils. All footing areas should be stripped of all organic soils, organic debris and any existing fills. We anticipate that non-organic, firm soils will be encountered at depths of approximately 4 to 6 feet, however depths may vary substantially which will necessitate HGSA's professional site observations during excavation for the foundations.

Care should be taken during excavation so that materials exposed in the excavation are not disturbed or softened. Protection of footing areas from deterioration may be necessary, and can be accomplished by placing 2 to 3 inches of well compacted crushed aggregate in footing and slab areas.

Any tree stumps, including the root systems, should be removed from beneath footing and pavement areas, and the resulting holes backfilled with compacted non-organic structural backfill placed in lifts not exceeding 8 inches and compacted to a dry density of at least 90 percent of the Modified Proctor maximum dry density (ASTM D1557). Each test pit exploration at the site as shown on Figure 3 should be backfilled with structural fill in a similar manner.

7.2 Soil Bearing Capacities

Footings bearing in undisturbed, native, non-organic, firm soils or properly compacted structural fill placed on these soils may be designed for the following:

ALLOWABLE SOIL BEARING CAPACITIES	
Allowable Dead Plus Live Load Bearing Capacity [a]	1,500 psf
Passive Resistance	200 psf/ft embedment depth
Lateral Sliding Coefficient	0.35

[a] Allowable bearing capacity may be increased by one-third for short term wind or seismic loads.

7.3 Footings

Our recommended minimum footing widths and embedment depths are as follows:

MINIMUM FOOTING WIDTHS & EMBEDMENT DEPTHS			
Number of Stories	One	Two	Three
Minimum Footing Width	12 inches	15 inches	18 inches
Minimum Exterior Footing Embedment Depth [a]	12 inches	18 inches	24 inches
Minimum Interior Footing Embedment Depth [b]	6 inches	6 inches	6 inches

[a] All footings shall be embedded as specified above, or extend below the frost line as per Table R301.2(1) of the 2014 ORSC, whichever provides greater embedment.

[b] Interior footings should be embedded a minimum of 6 inches below the lowest adjacent finished grade, or as otherwise recommended by HGSA during construction. In general, interior footings placed on sloping or benched ground should be embedded or set back from cut slopes in such a manner as to provide a minimum horizontal distance between the foundation component and face of the slope of one foot per every foot of elevation change.

7.4 Slabs-On-Ground

All areas beneath slabs should be excavated a minimum of 6 inches into native, non-organic, firm soils. The exposed subgrade in the slab excavation should be cut smooth, without loose or disturbed soil and rock remaining in the excavation.

SLABS-ON-GROUND	
Minimum thickness of 3/4 inch minus crushed rock beneath slabs	6 inches
Compaction Requirements	92% ASTM D1557, compacted in 8 inch lifts maximum

The slab excavation should then be backfilled with a minimum of 6 inches of ¾ inch minus, clean, free-draining, crushed rock placed in 8 inch lifts maximum which are compacted to 92 percent of the Modified Proctor (ASTM D1557). Reinforcing of the slab is recommended and the slab should be fully waterproofed in accordance with structural design considerations. An underslab drainage system is recommended for all below grade slabs, such as basement slabs, as per the architect's recommendations.

7.5 Seismic Requirements

The structure and all structural elements should be designed to meet current ORSC seismic requirements. Based on our knowledge of subsurface conditions at the site, and our analysis using the guidelines recommended in the ORSC, the structure should be designed to meet the following seismic parameters:

SEISMIC DESIGN PARAMETERS	
Site Class	D
Seismic Design Category	D_1
Mapped Spectral Response Acceleration for Short Periods	S_S = 0.829 g
Site Coefficients	F_a = 1.200 F_v = 1.927
Design Spectral Response Acceleration at Short Periods	S_{DS} = 0.663 g

7.6 Retaining Walls

Free standing retaining walls should be designed for a lateral static active earth pressure expressed as an equivalent fluid density (EFD) of 35 pounds per cubic foot, assuming level backfill. An EFP of 45 pounds per cubic foot should be used assuming sloping backfill of 2H:1V.

At rest retaining walls should be designed for a lateral at-rest pressure expressed as an equivalent fluid density (EFD) of 60 pounds per cubic foot, assuming level backfill behind the wall equal to a distance of at least half of the height of the wall. Walls need to be fully drained to prevent the build-up of hydrostatic pressures.

The above EFDs assume static conditions, and no surcharge loads from vehicles or structures. If surcharge loads will be applied to the retaining walls, forces on the walls resulting from these loads will need to be added to the pressures given above.

RETAINING WALL EARTH PRESSURE PARAMETERS	
Static Case, Active Wall (level backfill/grades)	35 pcf [a]
Static Case, Active Wall (2H:1V backfill/grades)	45 pcf [a]
Static Case, At-Rest Wall (level backfill/grades)	60 pcf [a]
Seismic Loading (level backfill/grades)	7.6 pcf $(H)^2$ [b]

[a] Earth pressure expressed as an equivalent fluid density (EFD). The location of the earth pressure can be assumed to act at a distance of 0.33H above the base of the wall.

[b] Seismic loading expressed as a pseudostatic force, where H is the height of the wall in feet. The location of the pseudostatic force can be assumed to act at a distance of 0.6H above the base of the wall.

For seismic loading a unit pseudostatic force equal to 7.6 pcf $(H)^2$, where H is the height of the wall in feet, should be added to the static lateral earth pressure. The location of the pseudostatic force can be assumed to act at a distance of 0.6H above the base of the wall.

Backfill for walls should be placed in 8 inch horizontal lifts and machine compacted to 90 percent of the maximum dry density as determined by ASTM D1557. Compaction within 2 feet of the wall should be accomplished with light weight hand operated compaction equipment to avoid applying additional lateral pressure on the walls. Drainage of the retaining wall should consist of slotted drains placed at the base of the wall on the backfilled side and backfilled with free-draining crushed rock (less than 5% passing the 200 mesh sieve using a washed sieve method) protected by non-woven filter fabric (Mirafi® 140N or equivalent) placed between the native soil and the backfill. Filter fabric protected free-draining crushed rock should extend to within 2 feet of the ground surface behind the wall, and the filter fabric should be overlapped at the top per the manufacturer's recommendations. All walls should be fully drained to prevent the build-up of hydrostatic pressures. All retaining walls should have a minimum of 2 feet of embedment at the toe, or be designed without passive resistance. The EFDs provided above assume that free draining granular material (less than 5% passing the 200 mesh sieve on a wet sieve analysis) will be used for the retaining wall backfill.

7.7 Structural Fills

Structural fills supporting building or wall loads, and slabs should consist of granular material, free of organics and deleterious materials, and contain no particles greater than 1½ inches in diameter so that nuclear methods (ASTM D2922 & ASTM D3017) can be easily used for field density and moisture testing. All areas to receive fill should be stripped of all soft soils, organic soils, organic debris, existing fill, and disturbed soils.

STRUCTURAL FILL	
Compaction Requirements	92% ASTM D1557, compacted in 8 inch lifts maximum, at or near the optimum moisture content (± 2%).

Proper test frequency and earthwork documentation usually requires daily observation during stripping, rough grading, and placement of structural fill. Field density testing should generally conform to ASTM D2922 and D3017, or D1556. To minimize the number of field and laboratory tests, fill materials should be from a single source and of a consistent character. Structural fill should be approved and periodically observed by HGSA and tested by a qualified testing firm. All failing areas shall be re-tested after additional compaction. Test results will need to be reviewed and approved by HGSA. We recommend that three density tests be performed for at least every 18 inches of fill placed and every 200 cubic yards, whichever requires more testing. Because testing is performed on an on-call basis, we recommend that it be scheduled by the earthwork contractor. Relatively more testing is typically needed on these smaller projects.

7.8 Erosion Control

Vegetation should be removed only as necessary and exposed areas should be replanted following construction. Disturbed ground surfaces exposed during the wet season (November 1 through April 30) should be temporarily planted with grasses, or protected with erosion control blankets or hydromulch.

Temporary sediment fences should be installed downslope of any disturbed areas of the site until permanent vegetation cover can be established. Exposed sloping areas steeper than 3 horizontal to 1 vertical (3H:1V) should be protected with a straw erosion control blanket (North American Green S150 or equivalent) to provide erosion protection until permanent vegetation can be established. Erosion control blankets should be installed as per the manufacturer's recommendations.

7.9 Cut and Fill Slopes

Temporary unsupported cut slopes less than 8 feet high should be no steeper than 1 horizontal to 1 vertical (1H:1V). All cuts greater than 8 feet high should be approved by a representative of HGSA. All permanent unsupported slopes should be no steeper than 2 horizontal to 1 vertical (2H:1V), or as approved by a representative of our firm.

If the above cut slope recommendations cannot be achieved due to construction and/or property line constraints, temporary or permanent retention of cut slopes may be required, as determined by a representative of our firm.

TEMPORARY AND PERMANENT CUTS	
Temporary Cuts	1H:1V (maximum) [a]
Permanent Cuts	2H:1V (maximum) [a]
[a] All cuts greater than 8 feet high, or cuts where water seepage is encountered, should be approved by a representative of H.G. Schlicker & Associates, Inc.	

7.10 Drainage

Surface water should be diverted from building foundations and walls to approved disposal points by grading the ground surface to slope away a minimum of 2 percent for at least 6 feet towards a suitable gravity outlet to prevent ponding near the structures. Permanent subsurface drainage of the building perimeter using footing drains is recommended.

Footing drains should be installed adjacent to the perimeter footings and sloped a minimum of 0.5 percent to a gravity outlet. A suitable perimeter footing drain system would consist of a 4-inch diameter, perforated PVC pipe (typical) embedded below and adjacent to the bottom of footings and backfilled with approved drain rock. The type of PVC pipe to be utilized may depend on building agency requirements and should be verified prior to construction. HGSA also recommends lining the drainage trench excavation with a non-woven geotextile filter such as Mirafi® 140N or equivalent to increase the life of the footing drain and prevent the drain from being clogged by soil. The perimeter drain excavation should be constructed in a manner which prevents undermining of foundation or slab components or any disturbance to supporting soils.

In addition to the perimeter foundation drain system, drainage of any crawlspace areas is required. Each crawlspace should be graded to a low point for installation of a crawlspace drain that is tied into the perimeter footing drain and tightlined to an approved disposal point.

All roof drains should be collected and tightlined in a separate system independent of the footing drains, or an approved backflow prevention device shall be used. All roof and footing drains should be discharged to an approved disposal point. If water will be discharged to the ground surface, we recommend that energy dissipaters, such as splash blocks or a rock apron, be utilized at all pipe outfall locations. Water collected on the site should not be concentrated and discharged to adjacent properties.

7.11 Plan Review and Site Observations

We should be provided the opportunity to review all site development, foundation, drainage, and grading plans prior to construction to assure conformance with the intent of our recommendations (Appendix C). The plans, details and specifications should clearly show that the above recommendations have been implemented into the design.

We should observe foundation excavations prior to placing structural fill, forming and pouring concrete to assure that suitable bearing materials have been reached (Appendix C). Please provide us with at least five (5) days notice prior to any needed site observations. There will be additional costs for these services.

8.0 Limitations

Our investigation was based on engineering geological reconnaissance, limited review of published information, and our subsurface exploration and analyses. The data presented in this report are believed to be representative of the site. The conclusions herein are professional opinions derived in accordance with current standards of professional practice and budget constraints. No warranty is expressed or implied. The performance of the site during a seismic event has not been evaluated. If you would like us to do so, please contact us.

The test pit logs and related information depict generalized subsurface conditions only at these specific locations and at the particular time the subsurface exploration was completed. Soil and groundwater conditions at other locations may differ from the conditions at these test pit locations. Also, the passage of time may result in a change in the soil and groundwater conditions at the site.

This report pertains to the subject site only, and is not applicable to adjacent sites nor is it valid for types of development other than that to which it refers. Geologic conditions including materials, processes and rates can change with time and therefore a review of the site and/or this report may be necessary as time passes to assure its accuracy and adequacy. This report may only be copied in its entirety.

9.0 Disclosure

H.G. Schlicker & Associates, Inc. and the undersigned Certified Engineering Geologist have no financial interest in the subject site, the project or the Client's organization.

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It has been our pleasure to serve you. If you have any questions concerning this report, or the site, please contact us.

Respectfully submitted,

H.G. SCHLICKER AND ASSOCIATES, INC.

CERTIFIED
OREGON
J. DOUGLAS GLESS
E-902
ENGINEERING GEOLOGIST
EXPIRES: 10/31/2017

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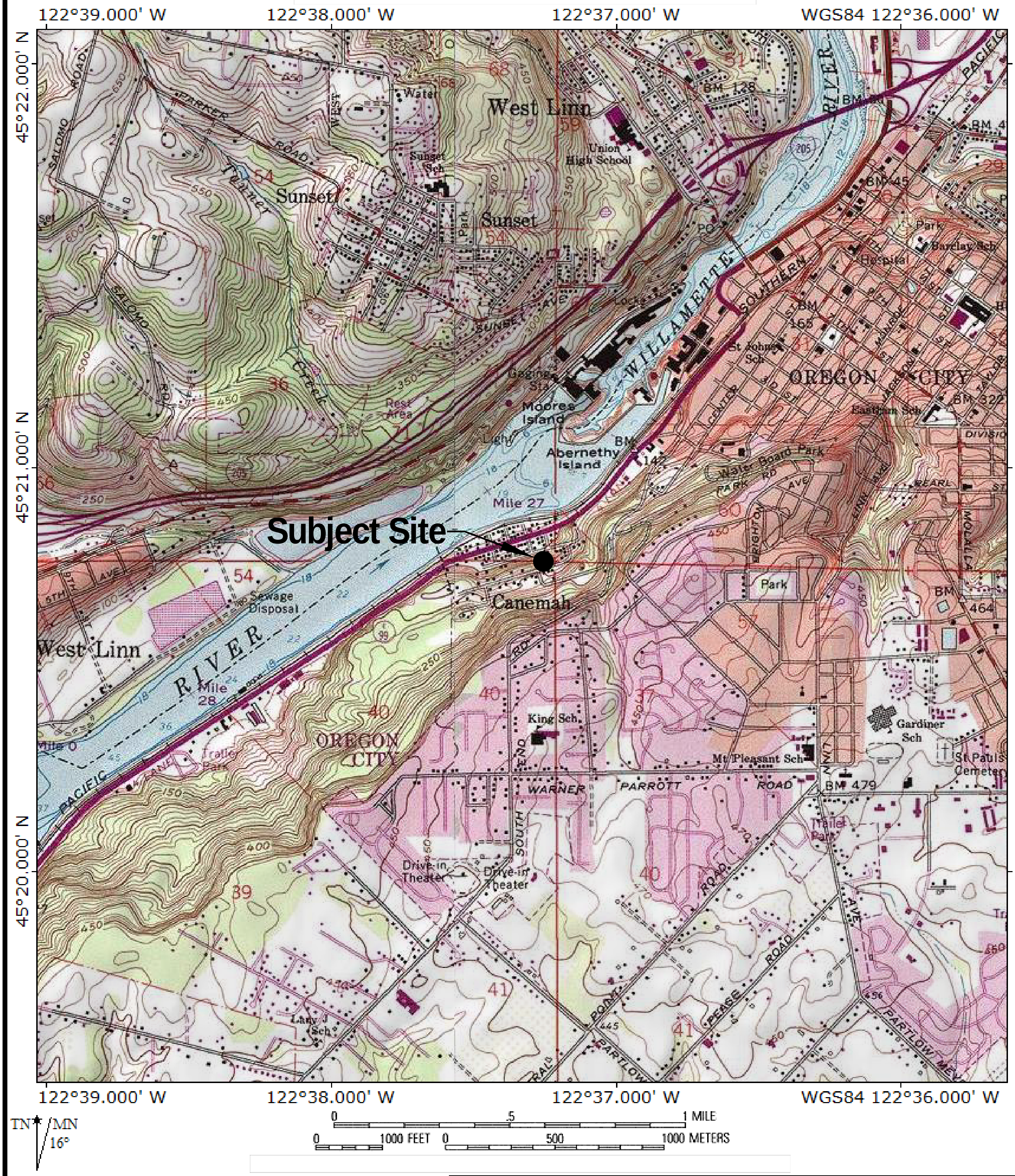
President/Principal Engineering Geologist

JDG:cjh



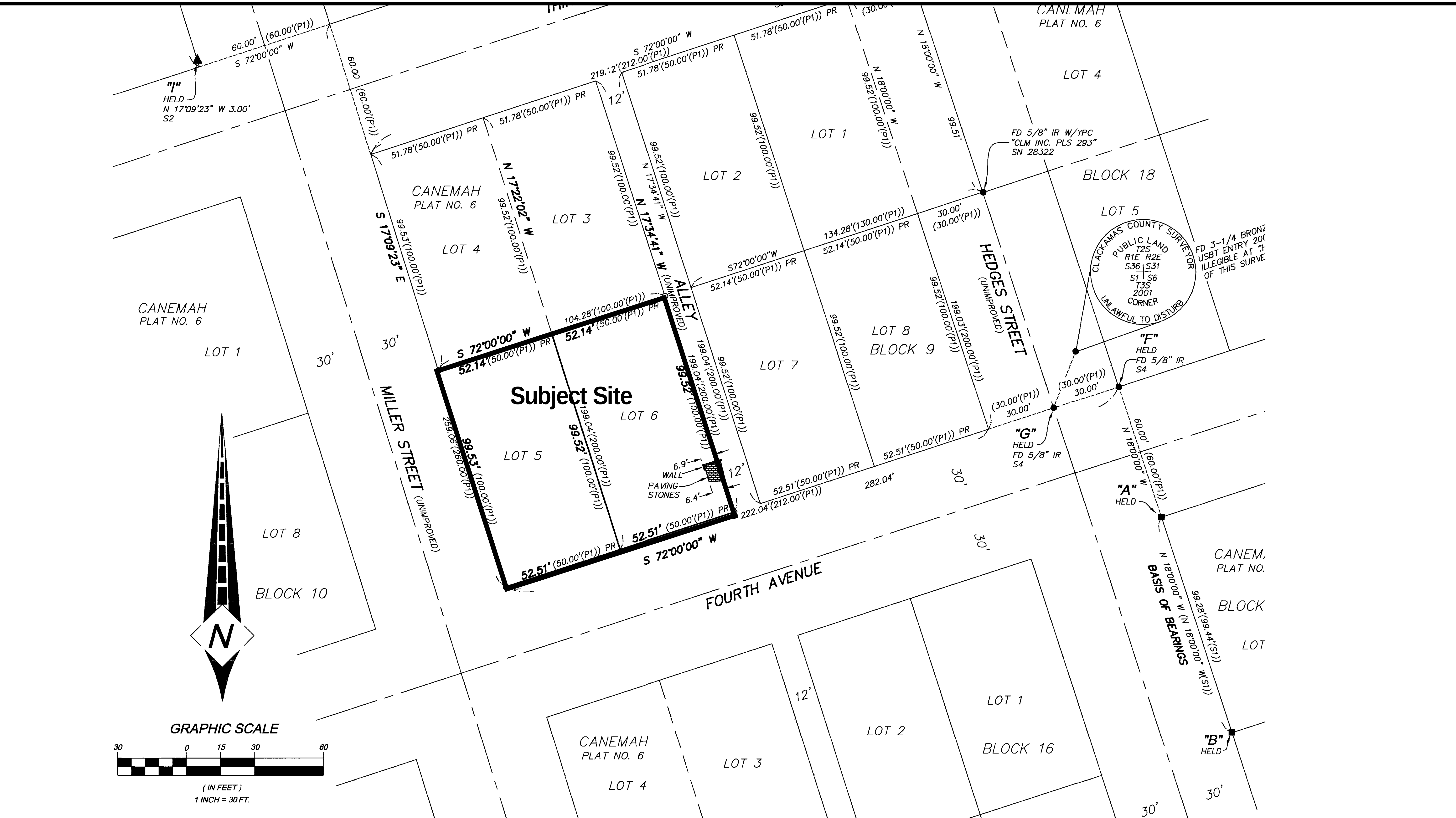
N

Date: 06/05/2017	**Project #Y174029**	Prepared by: CJH
Scale: 1" = 2,000'		Approved by: JDG

Location Map
Tax Lots 7300 and 7303, Map 2-1E-36DD
Oregon City, Oregon

H.G. Schlicker & Associates, Inc.

Figure 1



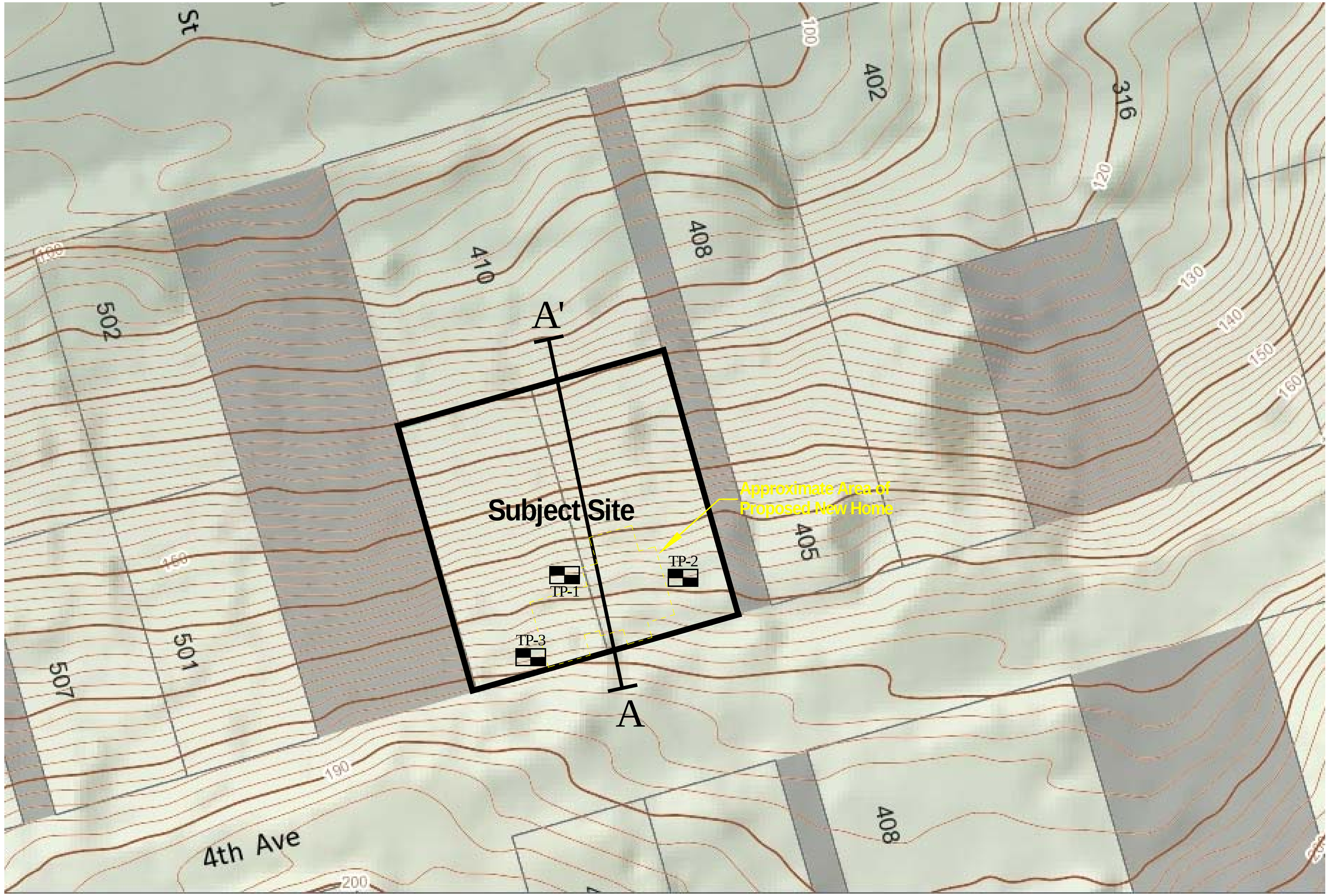
Date: 06/05/2017	**Project #Y174029**	Prepared by: CJH
Scale: 1" = 40'		Approved by: JDG

Site Map
Tax Lots 7300 and 7303, Map 2-1E-36DD
Oregon City, Oregon

H.G. Schlicker & Associates, Inc.

Figure 2

From Centerline Concepts Land Surveying, Inc. survey drawing provided by Client.
All locations and dimensions are approximate.

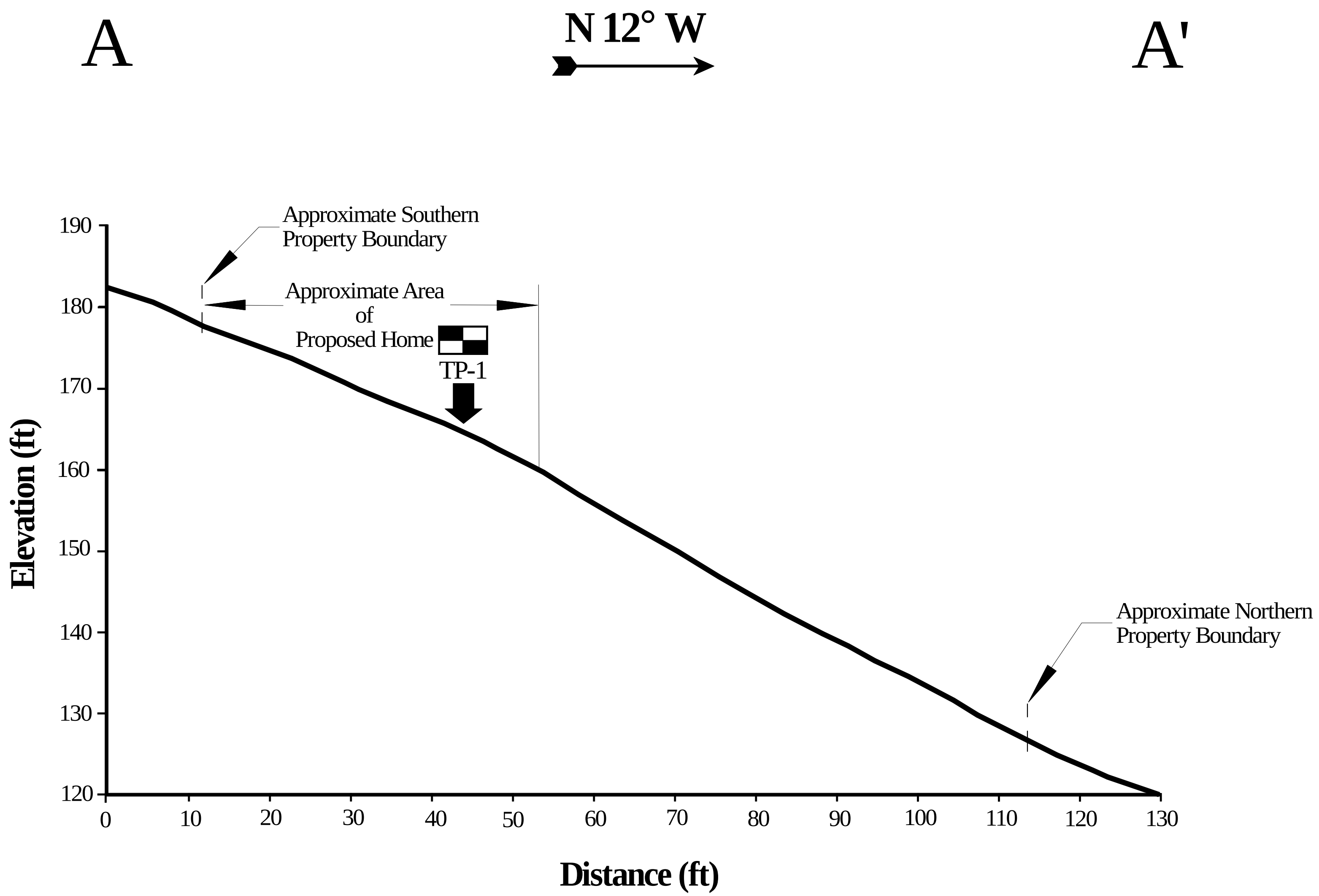


A A' = Approximate trend of profile line

= Approximate location of test pit
TP-1

From City of Oregon City WebMaps GIS mapping with 2 foot and 10 foot contour intervals and shaded relief displayed.
All locations and dimensions are approximate.

Date: 06/05/2017	Project #Y174029	Prepared by: CJH
Scale: 1" = 40'		Approved by: JDG
Site Topographic Map Tax Lots 7300 and 7303, Map 2-1E-36DD Oregon City, Oregon		
H.G. Schlicker & Associates, Inc.		**Figure 3**



TP-1 = Approximate location of test pit

Elevations from City of Oregon City WebMaps GIS mapping as shown on Figure 3.
All locations and dimensions are approximate.

Date: 06/05/2017	**Project #Y174029**	Prepared by: CJH
Scale: 1" = 20'		Approved by: JDG

Slope Profile A-A'
Tax Lots 7300 and 7303, Map 2-1E-36DD
Oregon City, Oregon

H.G. Schlicker & Associates, Inc.

Figure 4

Appendix A
- Site Photographs -

Photo 1 - Spoils from test pit TP-1. Note the rocky nature of the excavated materials.

Photo 2 - Test pit TP-2.

Photo 3 - Spoils from test pit TP-2.

Photo 4 - Test pit TP-3.

Photo 5 - Spoils from test pit TP-3.

Appendix B
- Test Pit Logs -

TEST PIT LOG EXPLANATION

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS), ASTM D2487			
MAJOR DIVISIONS		GROUP SYMBOL *	GROUP NAME
COARSE-GRAINED SOILS	GRAVELS	GW	Well-graded gravel
		GP	Poorly-graded gravel
		GM	Silty gravel
		GC	Clayey gravel
	SANDS	SW	Well-graded sand
		SP	Poorly-graded sand
		SM	Silty sand
		SC	Clayey sand
FINE-GRAINED SOILS	SILTS AND CLAYS Liquid Limits Less than 50	ML	Silt with low plasticity
		CL	Clay with low plasticity
		OL	Organic silt or organic clay with low plasticity
	SILTS AND CLAYS Liquid Limits 50 or more	MH	Silt with high plasticity
		CH	Clay with high plasticity
		OH	Organic silt or organic clay with high plasticity
HIGHLY ORGANIC SOILS		PT	Peat, Muck, and other highly organic soils.

Project #Y174029

TEST PIT LOGS

TP-1

Depth (ft.)	**USCS**	**Description**
0 - 1.2	ML (Fill)	CLAYEY SILT FILL, dark brown, moist, very soft; colluvial topsoil.
1.2 - 4.5	GW (Fill)	GRAVEL FILL, medium gray/orange-brown, wet, loose; subangular basaltic rock cobbles and boulders generally < 14" diameter, in clayey, sandy silt matrix.
4.5 - 6.5	GW	GRAVEL, medium gray/orange-brown, wet to saturated, loose; subangular basaltic rock gravel, cobbles and boulders generally < 12" diameter, in low plasticity clayey, sandy silt matrix. Becoming saturated at approximately 5½ feet depth.

TP-2

Depth (ft.)	**USCS**	**Description**
0 - 0.8	ML (Fill)	CLAYEY/SANDY SILT FILL, dark brown, moist, very soft.
0.8 - 5.0	GW (Fill)	GRAVEL FILL, medium gray/orange-brown, wet, loose; subangular basaltic rock cobbles and boulders generally < 14" diameter, in clayey, sandy silt matrix.
5.0 - 7.0	GW	GRAVEL, medium gray/orange-brown, wet to saturated, loose; subangular basaltic rock gravel, cobbles and boulders generally < 12" diameter, in low plasticity clayey, sandy silt matrix. Becoming saturated at approximately 6 feet depth.

TP-3

Depth (ft.)	**USCS**	**Description**
0 - 2.5	ML (Fill)	CLAYEY/SANDY SILT FILL, medium tan-brown, moist, very soft.
2.5 - 6.0	GW (Fill)	GRAVEL FILL, medium gray/orange-brown, wet, loose; angular to subangular basaltic rock gravel and cobbles generally < 12" diameter, in clayey, sandy silt matrix.
6.0 - 10.5	GW	GRAVEL, medium gray/orange-brown, saturated, loose; subangular basaltic rock gravel, cobbles and boulders generally < 18" diameter, in clayey, sandy silt matrix.

Appendix C
- Checklist of Recommended Plan Reviews and Site Observations -

Project #Y174029

APPENDIX C
Checklist of Recommended Plan Reviews and Site Observations
To Be Completed by a Representative of H.G. Schlicker & Associates, Inc.

Item No.	Date Done	Procedure	Timing
1*		Review site development, foundation, drainage, grading and erosion control plans.	Prior to construction.
2*		Observe foundation excavations.	Following excavation of foundations, and prior to placing fill, forming and pouring. **
3*		Review Proctor (ASTM D1557) and field density test results for all fills placed at the site.	During construction.

* There will be additional charges for these services.
** Please provide us with at least 5 days notice prior to all site observations.