Appendix G-4: Camas Solar Project Geotechnical Engineering Study

Geotechnical Engineering Study, Phase 1
Camas Solar Array Site
4561 NE 6 Road
Ellensburg, Washington
Project No. 170019:GES

Site Description

The subject site is a single tax lot, Kittitas County Tax Parcel No. 10566, and is located at 4561 No. 6 Road in Ellensburg, Washington. The site is roughly elongate in shape with the long axis oriented roughly northeast to southwest with the west property line abutting Highway 97 and comprises 45.08 acres. The site is currently being used to cultivate hay. The site is bounded to the west by Highway 97 and to the east by a perennial creek that drains from the northeast to the southwest. The site is bounded to the north by Tjossem Road. A high-pressure gas main extends across the property from northeast to southwest across the eastern one-third of the property. The site is sloped gently from north to south with an overall inclination of about four percent. Access to the lot is from the northeast corner onto a gravel lot that contains an older wood-framed barn with storage for flood-irrigation pipe and other accoutrements.

Proposed Site Development

It is our understanding that a series of linear solar panel racks will be constructed on the property along with access/maintenance roads and power infrastructure for distribution to the power grid. Locations for the racks have not been selected and we have not been provided with detailed plans and specifications for installation of the solar panels nor have we been provided with a grading plan. However, based on review of similar projects, we believe that very little grading will be required to construct the solar panel racks. We understand that the solar panels are attached end to end on horizontal steel rods that are supported on driven H-beams. The solar panels are attached to the horizontal supports which are oriented north-south. The panels rotate so that they are always exposed to the sun as it moves from east to west. We assume that the maintenance roads will run between the racks, provide access to the electrical infrastructure elements, and will probably run around the perimeter of the array.

If development plans should change or if our assumptions regarding development are incorrect, Swiftwater Environmental and Geotechnical (SEG) should be contacted as soon as possible to review and/or revise the recommendations contained herein. SEG should be provided an opportunity to review final plans and specifications to confirm that these recommendations have been incorporated into the plans.

Subsurface Conditions

On April 26, 2017, SEG personnel observed the installation of two test borings using a track-mounted vibratory drilling rig – a GeoProbe 8140 LC – also known as a sonic rig. A four-inch steel hollow-stem drill string is driven into the soil under high-frequency vibratory loading. Standard Penetration Testing (SPT) is then carried out at standard depths for this type of report. For this project, we directed the drillers to complete at least one SPT at the surface before drilling commenced to collect N-values for the surficial soils. SPTs were then variously carried out at standard depths of 2.5', 5.0', 7.5', 10.0', 12.5' and 15.0'. Based on hardness of drilling during advancement of the core barrel, we decided to forego some of the intermediate samples since we would be reviewing a complete sample after drilling was completed and would be able to identify unusual strata within the samples. Samples were collected from the SPT split-spoons at the appropriate depths and prepared for storage and removal from the site

One of the advantages of using the sonic drill rig is that a continuous four-inch diameter sample is collected in the hollow stem drill string as it is advanced. The continuous soil sample is then extruded into polyethylene bags and removed from the immediate area of the drill rig. The continuous sample can then be examined by cutting the bag open to expose the entire 2.4 to 4-foot sample. We were thus able to closely examine the soil profile and to identify bearing soils with unusual precision. The borings were advanced to a maximum depth of 16.5 feet below existing grade. We understand that standard H-beam penetration for this type of installation is six to eight feet below grade. Boring locations are shown on the attached Boring Location Plan (Figure 2) along with the Boring Logs on subsequent pages. Please refer to the Boring Logs for detailed descriptions of the site soils. A general description of soils and groundwater conditions is provided below.

The boring locations were selected to attempt to be representative of the entire site. Boring C-1 was completed in the north-northwest quadrant of the site, immediately to the south and west of the barn and staging area and Boring C-2 was located in the southeast quadrant of the site about 100 meters west of the creek alignment as shown on the attached Boring Location Plan. The soil profiles in both borings were very consistent and we believe that based on the depositional environment in available mapping and also on the locally flat topography, that the soil profile across the site will be similar to the profiles found in the borings.

In Boring C-1, we observed less than six inches of very dark brown highly organic sod underlain by a brown, moist medium dense "topsoil" like loam soil with varying amounts of fine gravel. We encountered this material to a depth of about four feet below grade. The driller reported an extensive change to very hard drilling at about 4.5 to 5 feet below grade and the SPT sample at 5.0 feet revealed a gray to dark gray silty sandy, partially cemented gravel with thin (<l" fine sand seams) that contained perched groundwater. N-values in this material was in excess of 40 and remained above that until termination of the hole at 16.5 feet below grade.

In Boring C-2, we observed a soil profile that was nearly identical to that found in C-1. Boring C-2 was terminated in this unit at 16.5 feet below grade.

In both borings, we observed that drilling was difficult with depth beginning at about three and a half to four feet below grade indicating that adequate embedment soils are present from about three to four feet below grade down to the depth of our test borings.

Groundwater

We encountered minor seepage in both borings from about eight and a half to 10 feet below grade. This water was encountered in thin relative clean sand seams and appears to have been "perched" within the seams as additional groundwater was not noted below these depths. Additional groundwater flow may be observed during the wetter winter months.

If grading and/or construction is carried out during the winter or spring months, the contractor should anticipate that possible groundwater seepage might be present and should plan accordingly. The entire profile of the site soils is moisture sensitive and those soils will be difficult to use as structural fill during the rainy winter and spring months. The underlying partially cemented sandy gravel soils will be less moisture-sensitive, but natural variability of the fine-grained fraction (e.g. silts and clay minerals) might cause these soils to be moisture-sensitive as well. In any case, if excavated site soils are to be used as structural fill, they should be protected from moisture while stock-piled.

Geologic Setting

According to the USGS Open File Report 1127, *Late Cenozoic Deposits, Landforms, Stratigraphy, and Tectonism in Kittitas Valley, Washington*, Richard B. Waitt, Jr., 1979, the subject site and surrounding area are underlain by Qs (Quaternary Alluvium, Sidestream Facies) which is characterized as downstream aggradation deposits with their source being upstream glacial moraines located to the west and northwest areas of the Kittitas Valley. These deposits consist primarily of basaltic gravels and sands with varying amounts of silt and clay minerals. The gravel varies from fine to coarse with occasional specimens in excess of one and a half meters. These undifferentiated sandy GRAVEL deposits are overlain by varying thicknesses of topsoil, weathered sandy gravel horizons, and loessal (wind) deposits that comprise the silty SAND and sandy SILT units that we observed from the surface down to the relatively un-weathered, partially cemented gravel. The gravel deposits consistently displayed some level of cementation that is most likely caused by breakdown of the basaltic rock to silt and clay minerals and then subsequent re-lithification under normal loading. The soils we observed in our borings at the subject site are consistent with this mapping.

In Boring C-2, we encountered a fine-grained, reddish brown to tan silty clay to clayey silt unit underlying the sandy gravel deposits. We contacted Dr. Nick Zentner, Professor of Geological Sciences at Central Washington University about this unit. Dr. Zentner indicated that it is probably an alluvial deposit that develops in slow-water areas and ox-bows proximate to streams and to the Yakima River. Dr. Zentner stated that these deposits are horizontally discontinuous and are found throughout the valley. The deposit on the subject site is thus likely limited in lateral extent, especially given that we did not encounter it in Boring C-1.

General Discussion and Recommendations

Based on the results of our site investigation, construction of the proposed solar panel array is feasible from a geotechnical standpoint provided that strong enough vertical H-beam supports are provided. The density of the soil matrix combined with the weight of the hammer might possibly damage the pile leading to less than satisfactory bearing capacity values. In this case, it would be prudent to complete several test borings to determine if the piles can be placed with damage.

We understand that the site will be developed by constructing linear racks of structural members to which the solar panels will be attached. The racks will be oriented north to south, which support the solar panels on east to west oscillating horizontal members. Those horizontal members are in turn supported on a series of driven steel H-beams. We have reviewed plans for similar installations and it appears that there is variation in the length of the horizontal supports meaning there will be variation on the total loading for each individual H-beam. For example, some H-beams will support eight solar panels along with their mounting and support equipment while others will support six panels, etc. This means that for a design bearing capacity, pile embedment depths will vary or possibly, the H-beam sizing will need to be increased. *In either case, it is critical that final plans be reviewed by SEG to ensure that appropriate bearing capacities and embedment lengths have been incorporated into the plans and specifications.* Further, SEG should be retained to provide construction monitoring to ensure that the H-beams are installed in accordance with the plans and specifications and to document that adequate bearing capacity is developed.

Final bearing capacities and embedment lengths can be computed once the loading for the piles has been provided to us. Alternatively, we can estimate the loading for each pile if we are provided with information about the weight of the panels, supporting members and ancillary equipment. In either case, we have been informed that this information will be forthcoming and we can then complete the pile designs. We have attached a figure that shows the appropriate embedment zone for the H-beams.

We strongly recommend that once the bearing capacities and embedment lengths have been determined, a series of test piles should be driven using the same equipment that will be used during construction. The purpose of this testing is two-fold: 1) it is necessary to determine that the piles can be driven into the bearing soils to the required embedment depth without damaging the pile and, 2) it is required in order to load test the resulting piles to determine that adequate bearing capacity is being developed.

Wind Loading

The Kittitas Valley, particularly the Ellensburg area, is known for year-around windy conditions. We understand that the solar panels that will be used for this project are 32-square-foot panels mounted immediately adjacent to one another in linear "bays" of seven or eight panels. Therefore, for purposes of computing wind-loading, each bay will effectively be a single panel of 224 or 256 square feet.

We computed the maximum wind load for a 32-square-foot panel using the Uniform Building Code (UBC, 1997) formula with the preferred alternate methodology for computing wind pressure. We also used the Enigneering News Record (ENR) methodology in or order to check consistency across methodology. The following are wind loads for a four-foot by eight-foot panel erected vertically that is exposed to wind that is moving over a flat surface with few obstructions. Wind speeds were obtained from the Applied Technology Council (ATC) website wherein maximum wind speeds are obtained by location. The ASCE site-specific wind speed values are obtained from the ASCE manuals noted, but were retrieved from the

ATC Windspeed by Location website. The wind speed values were then used in the UBC formula to calculate the following values for wind pressure in pounds for a single four-foot by eight-foot solar panel.

• ASCE 7-93 Wind speed (fastest mile in mph	70 mph	593 lbs
• ASCE 7-05 Wind speed (3 second peak gust)	85 mph	878 lbs
• ASCE 7-10 100-year Mean Recurrence Interval	91 mph	1006 lbs
• ASCE 7-10 Risk Category II	110 mph	1470 lbs

The wind pressure values are the functional equivalent of lateral pressure on the H-beams. These values are maximum values. Wind pressure and equivalent lateral pressure will be less than the values shown because the solar panels are rarely, or never, in a vertical position and presumably can be rotated to a horizontal position before or during high-wind events to "spill" wind over time. Further, we believe that end to end placement of the panels will lead to an overall increase in per-panel value due to the failure of the wind to "spill" over the panel edges. Once we have received the H-beam size information, we can compute actual lateral loads based on the wind pressure values above.

Site Preparation and Earthwork

We anticipate that site preparation for the subject site will consist of installation of Temporary Erosion and Sedimentation Control (TESC) measures, installation of grade staking, establishing clearing and grading limits, site clearing and stripping, and possible stockpiling of strippings and organic material in non-structural areas. Once these activities have been completed, installation of underground utilities and final grading can occur and construction of the solar panel array can commence.

Infiltration and Temporary Erosion and Sedimentation Control (TESC)

It is important to understand and utilize Low Impact Development (LID) practices and LID Best Management Practices (BMP) in Eastern Washington to reduce or eliminate concentrated storm water runoff and erosion. BMP also help limit the introduction of pollutants/contamination into Eastern Washington's arid land soils and rangeland soils. BMP that address these issues can be found in the Washington State Department of Ecology (DOE) publication 13-10-036, *Eastern Washington Low Impact Development Guidance Manual*, June 13, 2013.

Construction of the solar array will tend to create a *variable* increase in the total *and effective* impervious area of the site that is equivalent to the area of the solar panels and associated infrastructure. The increase is variable because the panels move in accordance with the position of the sun and are tilted most of the time. There will also be an increase in less pervious area because of the proposed gravel access roads.

Based on the results of our subsurface investigation, we believe that infiltration into the upper, topsoil-like silty sand/sandy silt soils is not only feasible, but is ongoing. This site, like many others in the vicinity of Ellensburg, have been cultivated using flood irrigation methods. This consists of running a perforated PVC pipe along the upslope side of a site and simply flooding the entire area. Irrigation water percolates into the soil and is stored above the underlying relatively impervious layer found throughout the area. Based on our observations at several of these sites and on publications and anecdotal testimonies, it is clear that these soils are quite capable of infiltrating storm water during an average year. According to the referenced DOE publication, Ellensburg is located in Climate Region 2 – Central Basin – and receives an average of about eight inches of precipitation per year, some of it in the form of snow. Given the variable nature of the solar panel position, and relatively low precipitation in the area, combined with the natural permeability of the upper soil horizon, it is our opinion that infiltration of normal storm water amounts will occur and that normal levels of storm water will not be concentrated to a significant extent.

Based on the texture and class of the soil types that we encountered, and on a review of various documents related to storm water control in Eastern Washington, it is our opinion that it is reasonable to assume an infiltration rate of 1.02 inches/hour for the upper, silty sand unit and 0.27 inches/hour for the underlying sandy gravel. The rate for the sandy gravel unit is assumed to be low because of the presence of fine-grained silt and clay minerals in the interstitial spaces and fractures of this partially cemented unit.

Temporary construction ingress and egress should be completed prior to the start of on-going construction traffic. A temporary construction entrance should be constructed of 8-12 inches of quarry spalls. If the soils in the entrance location are soft, a layer of geotextile fabric can be laid down as a barrier prior to placement of quarry spalls. The quarry spalls will provide a stable entrance/exit to the site and will limit tracking of mud onto Tjossem Road during and after wet weather. TESC measures consist of installation of silt fencing *as needed* around the site entrance, around the perimeter of the low side of the site, and at discharge points where sediment-laden surface water may enter off-site drainage features. Because the subject site is flat and slopes very gently to the south, silt fencing will probably not be required at the south perimeter unless desired by Kittitas County.

Stripping

No well-developed sod or heavily organic topsoil layer was observed at this site because of ongoing cultivation, thus stripping should not be required. If a topsoil horizon is observed in areas where maintenance roads are proposed, the topsoil should be removed down to mineral soil and replaced with crushed rock or structural fill. It is important that all deleterious material is removed prior to placement of structural fill. Topsoil strippings can be stockpiled for use in non-structural areas, as desired, but should not be allowed to mix with soils that will be used for structural fill.

Native Soils and Imported Soils

Native site soils encountered below the surface contain significant fines and are very moisture sensitive. However, if moisture content is near optimum, the soil can be used as compacted structural fill. Excavated site soils should be stockpiled and covered immediately if they are to be saved and used as structural fill. If the soils are above optimum moisture content, it may be possible to aerate them to reduce moisture content. This is possible during the warmer summer months, but it is difficult to achieve uniform moisture content. It may also be possible to use Portland cement as an admixture to reduce moisture content. If the site soils cannot be adequately compacted, it may be necessary to use imported soil for structural fill. Imported soil should be a well-graded granular mineral soil with fines content below five percent (i.e. less than five percent passing the No. 200 sieve) and should be at or slightly above the optimum moisture content. If construction is scheduled to occur during periods where precipitation is expected, a contingency should be built into the project budget for imported soil/crushed rock base (CRB) and other costs associated with placement of imported structural fill.

Subgrade Preparation

Once the site has been prepared for construction of the maintenance roads and placement of the H-beams, a SEG representative should observe subgrade conditions to confirm that they are as expected and to provide additional recommendations, if necessary. If disturbed native soil is encountered in structural areas (e.g. maintenance road prism or foundations for solar panel infrastructure) the fill should re-compacted in accordance with the specifications for structural fill or should be removed and replaced with structural fill as required to reach design grade. As an alternative, CRB can be placed and compacted. SEG should observe and confirm subgrade conditions as construction progresses. The contractor should be prepared to retain a local materials testing firm to sample soils to be used as structural fill, collect samples for Proctor testing, and to provide compaction testing as structural fill is placed, as needed.

Structural Fill

Structural fill is fill that is deliberately placed in thin lifts and compacted to a design specification. Structural fill is intended to support overlying structures in a manner that produces little or no post-construction movement. It is typically used under foundations, slabs, in utility trenches, roads, behind retaining walls, and in constructed slopes. Structural fill should be placed in loose lifts that do not exceed 12 inches and compacted to a relative compaction of 95 percent of maximum dry density as determined by Modified Proctor (ASTM D1557). Compaction specifications may vary, especially in utility trenches in public or private roads as specified by the local jurisdiction. Moisture content is critical to achieving adequate densification (compaction) and the upper unit of the site soils is very moisture sensitive, e.g. a small change in moisture content can make them unusable as structural fill. *If the soils are stockpiled and not covered, precipitation will make them difficult or impossible to use as structural fill.*

Foundations

We believe that foundations for the electrical infrastructure elements of the project can be supported on undisturbed, competent, native sandy gravel soils found below the upper topsoil-like horizon, on re-compacted native soils, on structural fill, or on CRB. Where loose or unsuitable soils are encountered at design subgrade, it will be necessary to re-compact the native soils to structural fill specifications or to over-excavate down to competent native soils then place structural fill or CRB up to design subgrade.

If the subgrade is prepared as described above, the following parameters may be used for design:

• Allowable soil bearing capacity	1,500 psf
• Passive earth pressure	300 pcf (equivalent fluid)
• Coefficient of friction	0.35

A one-third increase in the allowable soil bearing capacity may be assumed for short-term wind and seismic loading conditions. The passive pressure and friction values above include a factor-of-safety of at least 1.5. With anticipated structural loads, total settlement of one inch and differential settlement of one-half inch is anticipated. Most settlement should occur during construction, as dead loads are applied.

Seismic Design

The 2015 International Building Code recognizes the American Society of Civil Engineers (ASCE) for seismic site class definitions. In accordance with Table 20.3-1 of the ASCE Minimum Design Loads for Buildings and Other Structures manual, Site Class D should be used for design.

Based on our observations of the upper native silty sand soils and the underlying partially cemented sandy gravels, it is our opinion that the subject site has very low susceptibility to liquefaction. Liquefaction is a phenomenon wherein loose, saturated soils suddenly lose shear strength and begin to behave as a fluid. Liquefaction typically occurs under seismic loading conditions and if structures are supported on soils that liquefy, structural damage can occur. The site groundwater and native soil conditions we observed in the test borings have allowed us to reach this conclusion.

Drainage

We observed minor seepage in both borings but it appears to have been water in a "perched" condition in thin fine sand seams. We do not anticipate appreciable amounts of seepage during any excavation however, during the rainy winter months, it is prudent to anticipate seepage in excavations and groundwater control measures should be on-site or readily available, including trash pumps, sumps and discharge ditches. Seepage may create instability in the walls of excavations – SEG should be notified if seepage is observed in excavations so that control measures may be discussed with the contractor and properly implemented. The site should be graded such that surface water is directed away from structures and slopes. Surface water must never be allowed to pond near the tops or toes of slopes. Based on the soil types observed in our test pits, it is our opinion that infiltration is not feasible at this site. Storm water discharge BMPs should be implemented to control runoff from the site. Sediment-laden surface water must be treated such that water discharged from the site meet all water quality requirements. Storm water should not be discharged over the slope to the north of the site.

Excavations/Slopes

Soils observed in the upper one and a half to two feet of the test borings would be classified as OSHA/WISHA Type C. Temporary excavations like utility excavations and foundation excavations with heights in excess of four feet must be sloped no steeper than 1.5H:1V. If seepage is observed in these excavations, they may need to be sloped at 2H:1V to prevent sloughing due to seepage pressure. The dense native sandy gravel soil observed below about two feet would be considered OSHA/WISHA Type B soils and can be laid back at 1H:1V.

SEG should be contacted to observe temporary slopes and utility excavations as they are constructed or excavated to assess slope stability and recommend additional measures, if necessary.

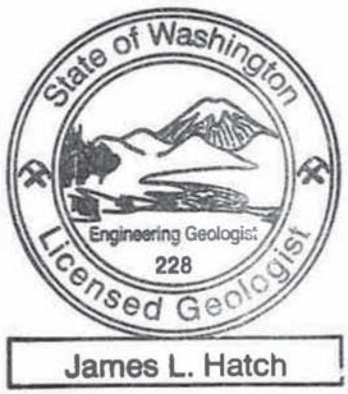
Utility Support, Trenches, and Trench Backfill

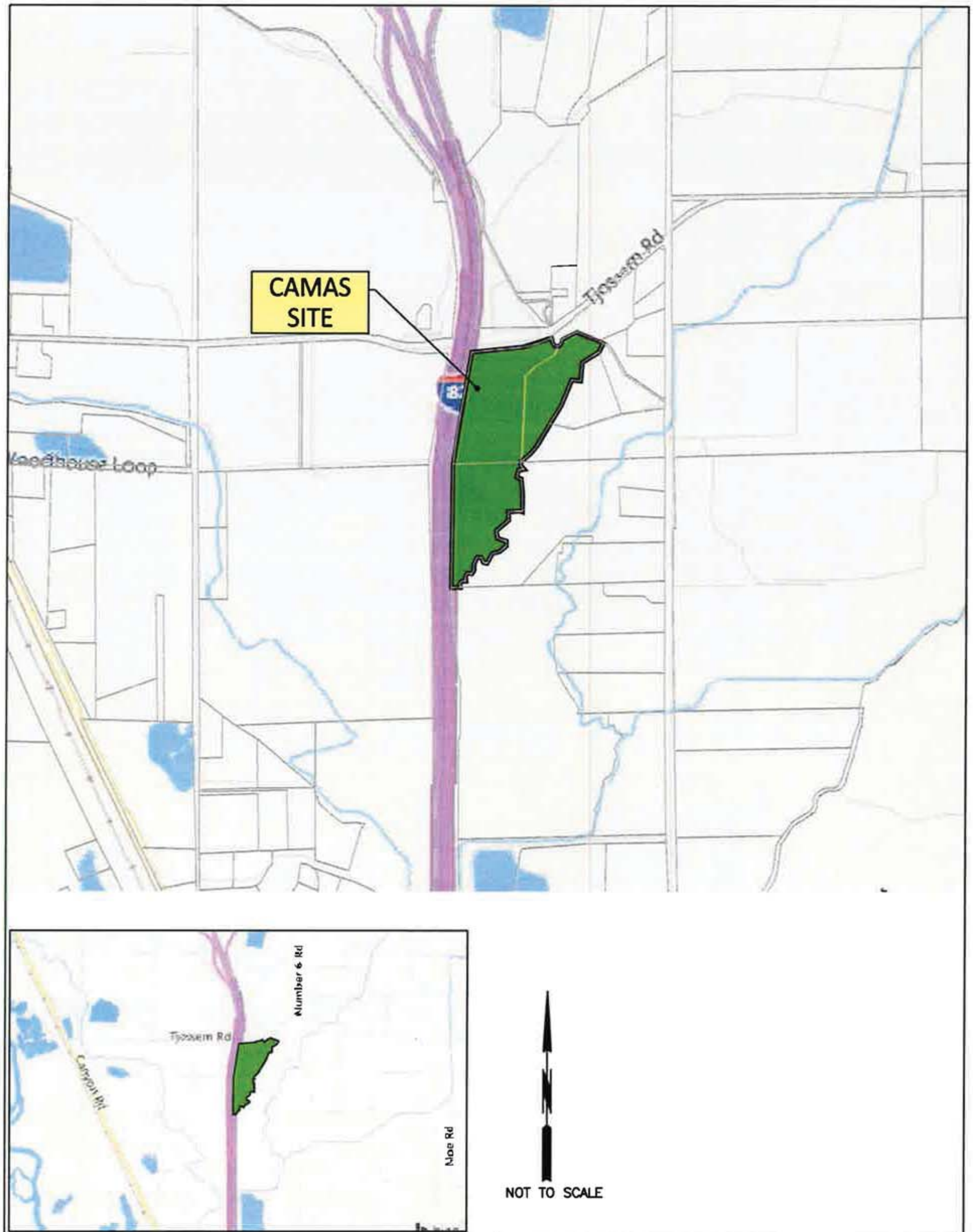
Site soils should be suitable for support of solar panel infrastructure and utilities. In shallower trenches, particularly shallower than about two feet, it may be necessary to over-excavate loose or wet soil down to suitable, stable soils, and then replace them with compacted structural fill or CRB. Groundwater seepage may be encountered in trench walls, particularly if deeper than two to three feet. Seepage may cause caving of the trench walls and temporary shoring may be required. Dewatering measures may also be needed to control seepage and SEG should be contacted to assess the need for such measures.

Site soils may be suitable for use as backfill provided the moisture content is optimal as determined in the laboratory. Trench backfill should be placed and compacted in accordance with the specifications for structural fill as described above or in accordance with specifications provided by the local jurisdiction. CRB should be placed in six to eight inch lifts and compacted with a plate compactor or other compaction device.

Limitations

This report is an instrument of service and has been prepared for the exclusive use of Tuusso Energy, LLC and their representatives and agents. The recommendations and conclusions provided in this study are professional opinions consistent with the level of care and skill that is typical of other members of the profession currently practicing in this area. A warranty is neither expressed nor implied. Variations in the soil and groundwater conditions observed in the test boring locations may exist and may not become apparent until construction commences. SEG should reevaluate the conclusions contained in this study if such variations are encountered.





CAMAS VICINITY MAP

Swiftwater
Environmental & Geotechnical
Western Washington Division
165 NE Juniper Street, Suite 201A • Issaquah, WA 98027 • Phone: (425) 321-0787
Eastern Washington Division
407 Swiftwater Blvd., Suite A • Cle Elum, WA 98922 • Phone: (425) 321-0787

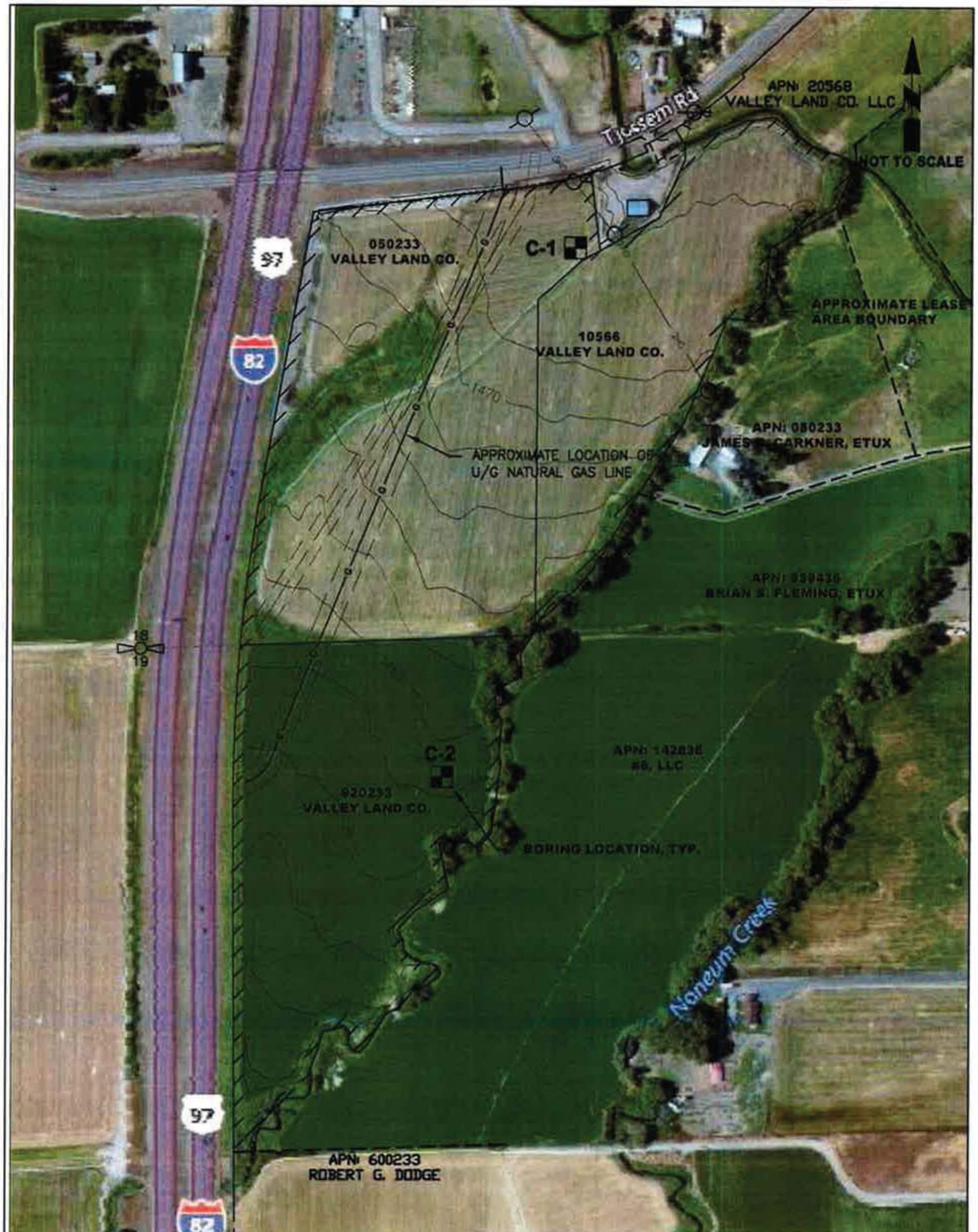


FIG. 2
CAMAS SITE
TEST BORING LOCATIONS

Swiftwater
Environmental & Geotechnical
Western Washington Division
165 NE Juniper Street, Suite 201A • Issaquah, WA 98027 • Phone: (425) 321-0787
Eastern Washington Division
407 Swiftwater Blvd., Suite A • Cle Elum, WA 98922 • Phone: (425) 321-0787

Swiftwater
Environmental & Geotechnical

Project: Tuusso Energy - CAMAS Site	170019	Date: 4/26/2017
Location 4561 No. 6 Road	Surface Elevation	Groundwater 8.5'

Earthwork Contractor	Excavation Equip.	Logged By	
Holocene Drilling	GeoProbe 8104 LC	JH	

Depth (feet)			USCS Soil	
From	To	Tests	Class.	
0.0	4.5 +/-	n - 8	SM	Dark brown silty fine SAND to sandy SILT, moist, loose to medium
				dense.
4.5	5.0	N - 3	SM/GP	Gray to bown silty sandy GRAVEL to gravelly SAND, moist, medium
				dense grading to very dense with depth. Some cementation
10.0	11.5	N - 43	SM/GP	Same
15.0	16.5	N - 43	SM/GP	Same - Test boring terminated at 16.5 feet below grade

Notes: 3-6" Topsoil / Sod @ Surface

Minor seepate noted at -8.5

TEST BORING NO.	C-1
Page	1 OF 1

Swiftwater
Environmental & Geotechnical

Project:	**Tuusso Energy - CAMAS Site**	**170019**	**Date:**	**4/26/2017**
Location	**4561 No. 6 Road**	**Surface Elevation**	**Groundwater**	8.5'
Earthwork Contractor Holocene Drilling	**Excavation Equip.** GeoProbe 8104 LC	**Logged By** JH		

Depth (feet) From	To	Tests	USCS Soil Class.	
0.0	4.5 +/-	n - 7	SM	Dark brown silty fine SAND to sandy SILT, moist, loose to medium
				dense.
4.5	5.0	N - 4	SM/GP	Gray to bown silty sandy GRAVEL to gravelly SAND, moist, medium
				dense grading to very dense with depth. Minor cementation
10.0	11.5	N - 32	SM/GP	Same - 6 inch silt seam with minor perched seepage
15.0	16.5	N - 30	SM/GP	Same - Test boring terminated at 16.5 feet below grade

Notes: 3-6" Topsoil / Sod @ Surface

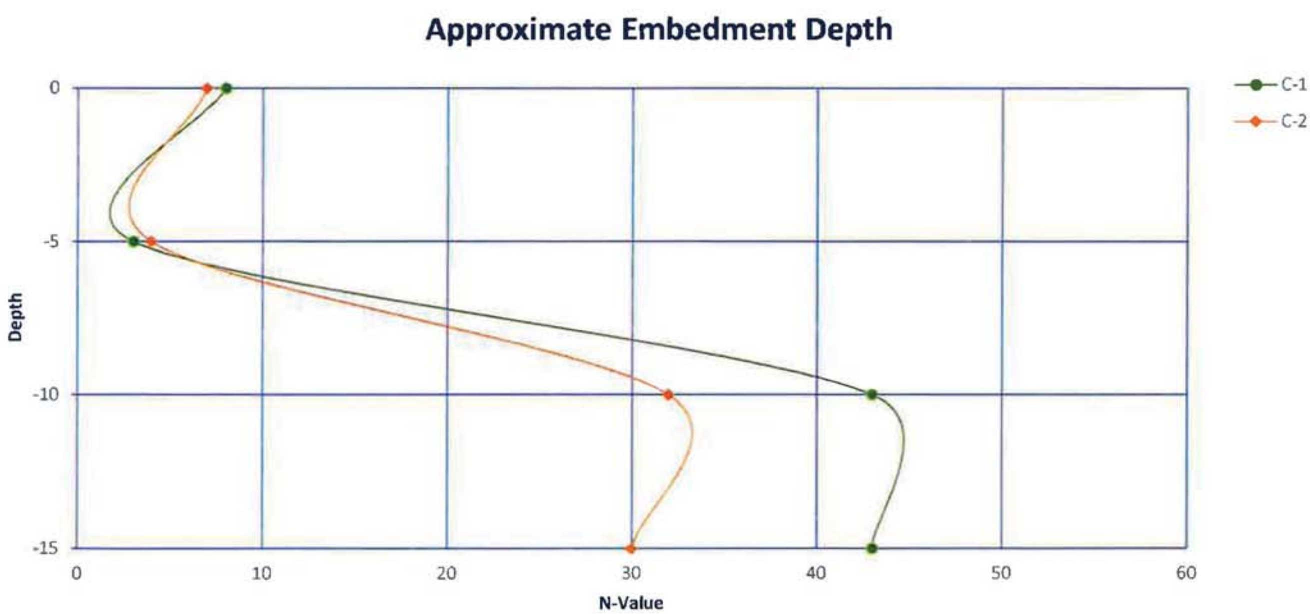
Minor seepate noted at -8.5

TEST BORING NO.	**C-2**
Page	1 OF 1

Project: Tuusso - Camas Site	Project No. 170019:GES	Date: 6/6/2017
Location 4561 No. 6 Road	Surface Elevation	Groundwater N/A

Earthwork Contractor	Excavation Equip.	Logged By	
Holocene Driling	Geoprobe 8104	JH	

		C-1				C-2			
		Depth (ft)	N			Depth (ft)	N		
		0.0	8.0			0.0	7.0		
		-5.0	3.0			-5.0	4.0		
		-10.0	43.0			-10.0	32.0		
		-15.0	43.0			-15.0	30.0		



Note: Tests conducted per ASTM D1586-11 (Standard test method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils)

Page	1 OF 1