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MEMORANDUM

To: Vee Godley / Northwest Innovation Works
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Date: February 6, 2015

GRI Project No.: W1159

Cc: Tom Pender / WorleyParsons

From: Matt Shanahan, PE, and Brian Bennetts, PE

Re: Preliminary Geotechnical Evaluation
Northwest Innovation Works Methanol Terminal
Port of Kalama, Washington

DRAFT

At your request, GRI has completed a preliminary geotechnical evaluation for the proposed Northwest Innovation Works Methanol Terminal located at the North Port area of the Port of Kalama, Washington. The Vicinity Map, Figure 1, shows the general location of the property. The purpose of the work was to evaluate available geotechnical information for the site and provide preliminary geotechnical recommendations for initial planning of the proposed methanol terminal. Limited geotechnical analyses were completed to evaluate seismic hazards and foundation support on a preliminary basis. Subsurface explorations and detailed geotechnical analyses will be required for final design. This memorandum documents the work accomplished and provides the results of our studies.

Our scope of work has also included preliminary field infiltration testing to assess the feasibility of on-site infiltration of stormwater. The results of this work were provided in our January 15, 2015, draft memorandum to you entitled, "Preliminary Stormwater Infiltration Evaluation, Northwest Innovation Works Methanol Plant, Port of Kalama, Washington."

PROJECT DESCRIPTION

Development of the terminal will include construction of a variety of chemical processing units that support the production of methanol. Our understanding of the number and types of structures is incomplete as the facility plan and layout are being developed at this time. However, we understand the project will include construction of large product tanks to store methanol and several smaller liquid storage tanks; cooling towers; large air scrubber units (ASUs); a large number of pipe support rack systems; structural supports and foundations for mechanical and chemical process units, including generators, boilers, tanks, and relatively tall towers; and partially enclosed metal-clad buildings and small pre-engineered metal equipment control buildings. A dock will be constructed to support methanol export operations.

SITE DESCRIPTION

The subject site is located in the North Port area of the Port of Kalama, north of the existing Steelscape facility in an area mantled with dredged sand fill. The site is bordered by the Columbia River to the west, a forested area and backwater slough to the north, and Burlington Northern Railroad (BNRR) right of way to the east. The site was filled with dredged sand to about elevation +20 to +23 ft (Columbia River Datum) in the 1980s and 1990s. Two large stockpiles of dredged sand constructed in 2007 and 2008 cover a significant portion of the site. The easternmost stockpile was constructed in a uniform shape and height, and the westernmost stockpile located adjacent to the river is more irregularly shaped and has a variable height. Some of the easternmost stockpile has been removed. Access to the area is provided by asphaltic-concrete pavement or gravel-surfaced perimeter roads.

SUBSURFACE CONDITIONS

Geotechnical data from the following sources were reviewed and form the basis of our understanding of the subsurface conditions at the site.

> GRI, June 11, 1993, Preliminary Geotechnical Investigation, Proposed Steel Mill, North Port Site, Port of Kalama, Washington; report prepared for Nucor Steel
>
> GRI, April 25, 2003, Geotechnical Investigation and Site-Specific Hazards Study, Proposed Dock Expansion for North Port Terminal, Port of Kalama, Washington; report prepared for Berger/ABAM Engineers and the Port of Kalama
>
> PBS Engineering and Environmental, Inc., July 28, 2006, Geotechnical Data Report, Pacific Mountain Energy Center, Kalama, Washington; report prepared for Energy Northwest

The materials and conditions encountered in subsurface explorations by GRI and as described by PBS Engineering and Environmental, Inc. can be generally grouped into the following categories:

1. **FILL: Dredged SAND**
2. **SILT and SAND**
3. **SAND**

1. FILL. The explorations reviewed for this study indicate site is mantled with dredged sand fill that ranges from 11 to 21 ft thick and averages about 15 ft. The dredged sand is fine to coarse grained and contains up to a trace of silt and fine gravel. The fill is typically loose to medium dense.

2. SILT and SAND. The fill is underlain by interbedded layers of alluvial silt and sand. The sand is typically gray and fine grained and contains varying percentages of silt, ranging from a trace of silt to silty. The relative density of the sand is typically loose to medium dense. The silt is typically soft to medium stiff, gray, and contains varying percentages of sand, ranging from some sand to sandy. Our experience at the site indicates the silt has a low to moderate compressibility in the overconsolidated range of pressures and a high compressibility in the normally consolidated range of effective pressures. The silt and sand typically extend to depths of 35 to 45 ft.

3. SAND. Gray, fine- to coarse-grained sand was encountered below the interbedded silt and sand deposit. The sand contains varying percentages of silt, ranging from trace of silt to silty, and scattered, thin lenses of fine organic material, fine pumice gravel, and volcanic ash. The relative density of the sand is medium dense to dense. Subsurface explorations provided in the referenced reports were terminated in sand at depths of about 101.5 to 149 ft.

Groundwater levels in the project area fluctuate in response to seasonal river levels, precipitation, and daily tidal fluctuations in the river. Ordinary low and high water levels of the Columbia River at the site are 0.0 and +11.6 ft, respectively, and the 100-year flood elevation is approximately +18.5 ft. Shallow perched groundwater conditions could develop within the dredge sand fill and approach the ground surface during flood stages of the Columbia and Kalama rivers and periods of prolonged or intense rainfall.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

The North Port site is mantled with about 10 to 20 ft of dredged sand fill. This material is underlain by relatively compressible silt soils to depths of 35 to 45 ft. The silt soils are underlain by medium dense sand. The depth to groundwater at the site will reflect the level of the Columbia River. Silt and sand located below the groundwater table are susceptible to liquefaction as a result of the MCE_R earthquake. Liquefaction could result in significant lateral displacement toward the river and ground surface settlement. Ground improvement and/or deep foundations will likely be required to mitigate the risk of liquefaction and to support heavily loaded structures. Lightly loaded structures that can tolerate the estimated liquefaction and lateral spreading deformations can be supported by shallow foundations.

The following sections of this memorandum summarize our preliminary conclusions and recommendations for design of the proposed development based on our review of available subsurface information.

Seismic Considerations

Code-Based Response Spectrum. We understand seismic design of the terminal will be in accordance with the 2012 International Building Code (IBC), which references ASCE Standard 7-10, Minimum Design Loads for Buildings and Other Structures (ASCE 7-10). The IBC design methodology uses two spectral response coefficients, S_S and S_1, corresponding to periods of 0.2 and 1.0 second, to develop the earthquake response spectrum. The spectral response coefficients were obtained from the U.S. Geological Survey (USGS) Uniform Hazard Response Spectra Curves for the coordinates of 46.05° N latitude and 122.87° W longitude. The S_S and S_1 coefficients identified for the site are 0.94 and 0.43 g, respectively, for Site Class B conditions. These bedrock spectral ordinates are adjusted for Site Class with the 0.2- and 1.0-second period site coefficients, F_a and F_v, based on the soil profile in the upper 100 ft. This spectrum is designated the MCE_R-level spectrum. The design-level response spectrum is calculated as two-thirds of the Site Class-adjusted MCE_R spectrum.

According to Section 20.3.1 of ASCE 7-10, soil profiles containing potentially liquefiable soil would classify as Site Class F and require a site-specific response analysis to determine the response spectrum. An exception to this requirement is provided in Section 20.3.1 of ASCE 7-10 for structures that have a fundamental period less than 0.5 second. For structures with a fundamental period of less than or equal to 0.5 second, Section 20.3.1 of ASCE 7-10 allows for the site coefficients to be equal to the site coefficients if the soil profile were not susceptible to liquefaction. In the absence of liquefaction, the site would classify as Site Class E (Soft Soil). For Site Class E, a value of 0.98 should be used for site coefficient F_a and 2.40 for site coefficient F_v. Recent literature also suggests the use of Site Class E is conservative for structures having a fundamental period in the range of 0.5 to 1.0 second.

Liquefaction. Liquefaction is a process by which saturated granular materials, such as sand, and non-plastic and low-plasticity silts, temporarily lose strength during and immediately after a seismic event. Liquefaction occurs as seismic shear stresses propagate through saturated soil and distort the soil structure causing loosely packed groups of particles to contract or collapse. If drainage is impeded and cannot occur quickly, the collapsing soil structure increases the porewater pressure between the soil grains. If the porewater pressure increases to a level approaching the weight of the overlying soil, the granular layer temporarily behaves as a viscous liquid rather than a solid. Liquefaction will result in ground surface settlement, decreased bearing capacity and settlement of shallow foundations, a reduction in the axial and lateral capacity of pile foundations, and lateral deformations towards the Columbia River.

The potential for liquefaction at the site was evaluated using the simplified procedure as described by Idriss and Boulanger (2008). As required by Section 11.8.3 of ASCE 7-10, liquefaction and lateral spreading susceptibility was evaluated for the MCE-level peak ground acceleration (MCE_G) of 0.37 g. In our analysis, we used a design earthquake magnitude of 9.0, based on the 2008 USGS interactive deaggregations, which forms the basis of seismic ground motion maps provided in ASCE 7-10. The near-surface silt has a very low plasticity index or is non-plastic and was considered to be susceptible to liquefaction in our evaluation. For the purpose of our liquefaction studies, we have assumed the groundwater level is approximately 16 ft below the ground surface, i.e., at about elevation +6 ft. Our preliminary analysis indicates the loose to medium dense sand and soft to medium stiff silt present below the groundwater table is susceptible to liquefaction.

It should be noted that the simplified method used to evaluate the liquefaction potential is based on historical databases of liquefied sites with shallow liquefaction, i.e., less than about 50 ft. For this reason, the simplified method may not be able to accurately predict liquefaction at depths greater than 50 to 60 ft. Accordingly, the depth of liquefaction described above and the potential settlement and lateral spreading displacements could be less than predicted. For preliminary planning, it should be assumed that potentially liquefiable soil extends to a depth of 80 to 100 ft below existing site grades. Available geotechnical data indicate variability in the soil conditions could locally affect the depth of liquefaction. For final design, we recommend that an effective stress non-linear site response analysis be completed to further evaluate the potential for deep liquefaction.

Liquefaction-Induced Settlement. We estimated the liquefaction-induced free-field settlement using empirical methodologies developed by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992). The empirical methods are based on case histories of areas that had undergone liquefaction. Using these empirical methodologies, we estimate liquefaction-induced settlement as a result of the MCE_R earthquake could be on the order of 1 1/2 to 2 1/2 ft.

Lateral Spreading. The methodology presented by Youd et. al. (2002) was used to estimate liquefaction-induced lateral spreading toward the river. Our analysis indicates the MCE_R earthquake could result in 10 and 5 ft of lateral spreading deformation at the crest and a point 1,000 ft beyond the crest of the Columbia River, respectively. Lateral displacements at distances greater than 1,000 ft beyond the crest of the riverbank are difficult to quantify using available analytical methods. Lateral spreading displacements of a smaller magnitude could also occur toward the north backwater. It should be acknowledged that the available analytical methods do not predict localized effects, such as flow failures, that may occur near the crest of slopes. If not mitigated, lateral spreading will result in horizontal displacement of structures and additional lateral structural loads on piles and walls.

Ground Improvement. A ground improvement program can be designed to improve the existing subsurface soils and reduce potential seismic-induced settlement and lateral spreading. We anticipate ground improvement, if used under tanks or other improvements, would be designed by a specialty ground improvement contractor to meet specified performance criteria. Ground improvement design to reduce lateral spreading near the river could be designed by the project team or specialty contractor to limit seismic deformation to tolerable levels.

Lateral spreading is often mitigated by constructing a zone, or buttress, of improved soil along the riverbank that will not liquefy. The buttress needs to be of sufficient width and extend to adequate depth to maintain stability following ground shaking and minimize or prevent lateral displacement toward the river of the upland portion of the site behind the buttress. We estimate a buttress composed of stone columns would have a minimum width of 50 ft and extend to a depth of at least 60 ft. The width of a buttress constructed using soil mixing or jet grouting techniques will likely be less than the width of a stone column buttress.

Several ground improvement alternatives, including vibro-replacement (stone columns), soil mixing, and jet grouting, are feasible to mitigate seismically induced settlement and lateral spreading. Vibro-replacement (stone columns) is a ground improvement technique that can densify (and reduce liquefaction potential) relatively clean granular soils using a vibratory probe. The probe is vibrated and jetted into the ground until reaching the bottom of the improvement zone. Stone aggregate is added to the void created by the probe after reaching the bottom of the treatment zone. The aggregate is densified by lowering the probe into the aggregate in small lifts until reaching the ground surface, creating columns of compacted aggregate. Stone columns are typically most effective in densifying relatively clean sand with less than about 15% fines (percentage of material passing the No. 200 sieve). Stone columns can also be used in silty soil; however, in these soils, the stone columns are installed in a tighter configuration and act more as reinforcement elements rather than to densify the adjacent ground.

Soil mixing and jet grouting are ground improvement methods that mix cement into the in situ soils to create columns of soil with improved strength and stiffness. During soil mixing, wet or dry cement is mixed with the in situ soils by use of a mechanical paddle that is advanced similar to a drill. The diameter of the soil-cement column is dependent on the diameter of the paddle tool. Jet grouting makes soil/cement columns by injecting cement grout through high-velocity grout jets. The jets erode the in situ soil and mix it with cement and sometimes air and water. Jet grouting can be used to construct improved soil/cement columns or columns can be overlapped to create continuous panels. While jet grouting or deep soil mixing can be used to improve sandy and silty soils, these methods are typically more expensive than stone column ground improvement.

Foundations

Shallow Foundations. Conventional continuous or isolated spread footings established in the clean sand fill that mantles the site can be used to support lightly loaded structures, provided the structure can tolerate the seismically induced settlement discussed on page 4. Continuous and isolated spread footings should have a minimum width of 18 and 24 in., respectively. The footings should be embedded about 18 in. below the lowest adjacent finished grade for frost protection. Upon completion of the footing excavations, the bottom of the excavation should be moisture conditioned and compacted to at least 95% of the maximum dry density as determined by ASTM D 698.

For preliminary evaluation and based primarily on static settlement considerations, shallow foundations can be designed based on a maximum allowable bearing capacity of 3,000 psf. This value applies to the total of dead load plus permanently and/or frequently applied live loads and can be increased by one-third for the total of all loads: dead, live, and wind or seismic.

Assuming dredged fill thickness of about 15 ft and the subgrade is established in accordance with the above criteria, we estimate the settlement of spread footings supporting a dead load plus permanently and/or frequently applied loads of up to 150 kips for isolated spread footings or 12 kips/ft for continuous footings will be less than 1 in. Differential settlement between adjacent spread footings should be less than half the total settlement. Differential settlement between pile-supported foundations and spread footings will approach the estimated total settlement of 1 in. Total and differential settlements could be larger in areas with a thinner layer of dredged fill. Due to the presence of soft, compressible silt beneath the sand fill, footings established more than 2 ft below existing grade, if required, should be evaluated individually.

The total frictional resistance between the footing and the soil can be computed as the normal force, i.e., the sum of all vertical forces (dead load plus real live load), times the coefficient of friction between the soil and the base of the footing. We recommend an ultimate value of 0.45 for the coefficient of friction for concrete cast on sand structural fill. If additional lateral resistance is required, passive earth resistance against embedded footings can be computed using a pressure based on an equivalent fluid with a unit weight of 250 pcf. This value also assumes the ground surface in front of the foundation is horizontal, i.e., does not slope downward away from the toe of the footing, and the foundation is cast neat against undisturbed soil or the foundation excavation is backfilled with compacted structural fill. We recommend the upper 1 ft of passive resistance be neglected in the design if the adjacent soil is not covered with pavement or a concrete floor.

Deep Foundations. Heavily loaded structures or structures that cannot tolerate the estimated liquefaction-induced total and differential settlement should be supported by pile foundations. We anticipate the pile tip will need to be located at least 10 ft below the maximum depth of liquefaction. The actual tip elevation will depend on structural loads and settlement tolerances. We anticipate the feasible pile types for this project could include closed-end steel pipe piles or driven grout piles. Driven grout piles are installed by driving a hollow mandrel, fitted with a sacrificial boot at the tip, to a predetermined depth using an impact pile driving hammer. As the mandrel is driven and withdrawn, grout is pumped through the mandrel to maintain the diameter of the pile shaft. Typical feasible depths of driven grout piles are in the range of 100 to 120 ft. If longer piles are needed, steel pipe piles may be a better alternative.

For preliminary planning, we estimate that 16- to 18-in.-diameter, approximately 100-ft-long driven grout piles can develop a static allowable compressive capacity on the order of 300 kips. For seismic loading, an allowable compressive capacity of about 200 kips can be assumed. Our seismic pile capacity accounts for the reduction of side resistance due to liquefaction. For our preliminary assessment, we have assumed there are no significant uplift loads on the piles. The allowable pile capacities are based on soil support characteristics and include an estimated factor of safety of about 2.0 for static loading and about 1.5 for seismic loading conditions. The actual pile lengths will be based on the specific soil conditions, driving resistance, and testing during pile installation. Driven steel pipe piles of similar diameter would likely need to be installed to greater depths to achieve similar capacities.

The allowable pile capacities assume the piles have a minimum center-to-center spacing of three pile diameters and are located outside of heavily loaded floor areas. If the piles are installed at tighter spacing or if heavy floor loads are present that could result in ground surface settlement (and downdrag loads), driven grout piles may need to be installed to deeper depths to achieve the same capacity.

Static and seismic pile settlement depends on the structural loads and the lengths of the piles and should be evaluated as part of the final design. However, under static loading conditions, individual piles are typically designed to limit settlement to approximately the elastic shortening of the piles plus 1/4 in. Larger settlements can be expected for seismic conditions unless the piles are designed to minimize seismic settlement.

LIMITATIONS

This memorandum has been prepared to aid Northwest Innovation Works in the preliminary planning and cost estimating for the proposed methanol export facility at the Port of Kalama, Washington. The preliminary findings presented herein are based on available subsurface information developed by GRI and others. The scope of our investigation was limited by the fact that actual plans for development are indefinite; hence, only preliminary opinions are presented. Significant limitations are inherent in a study of this type, and additional site investigations should be conducted as specific construction plans and designs are developed. The information provided in this memorandum is not intended for final design of the project. Additional exploration work and engineering analyses will be required to develop criteria and guidelines for final design.

Please contact the undersigned if you have any questions regarding this memorandum.

Submitted for GRI,

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Project Engineer

References

Idriss, I.M., and Boulanger, R.W., 2008, Soil liquefaction during earthquakes: Earthquake Engineering Research Institute (EERI), MNO-12 p. 226.

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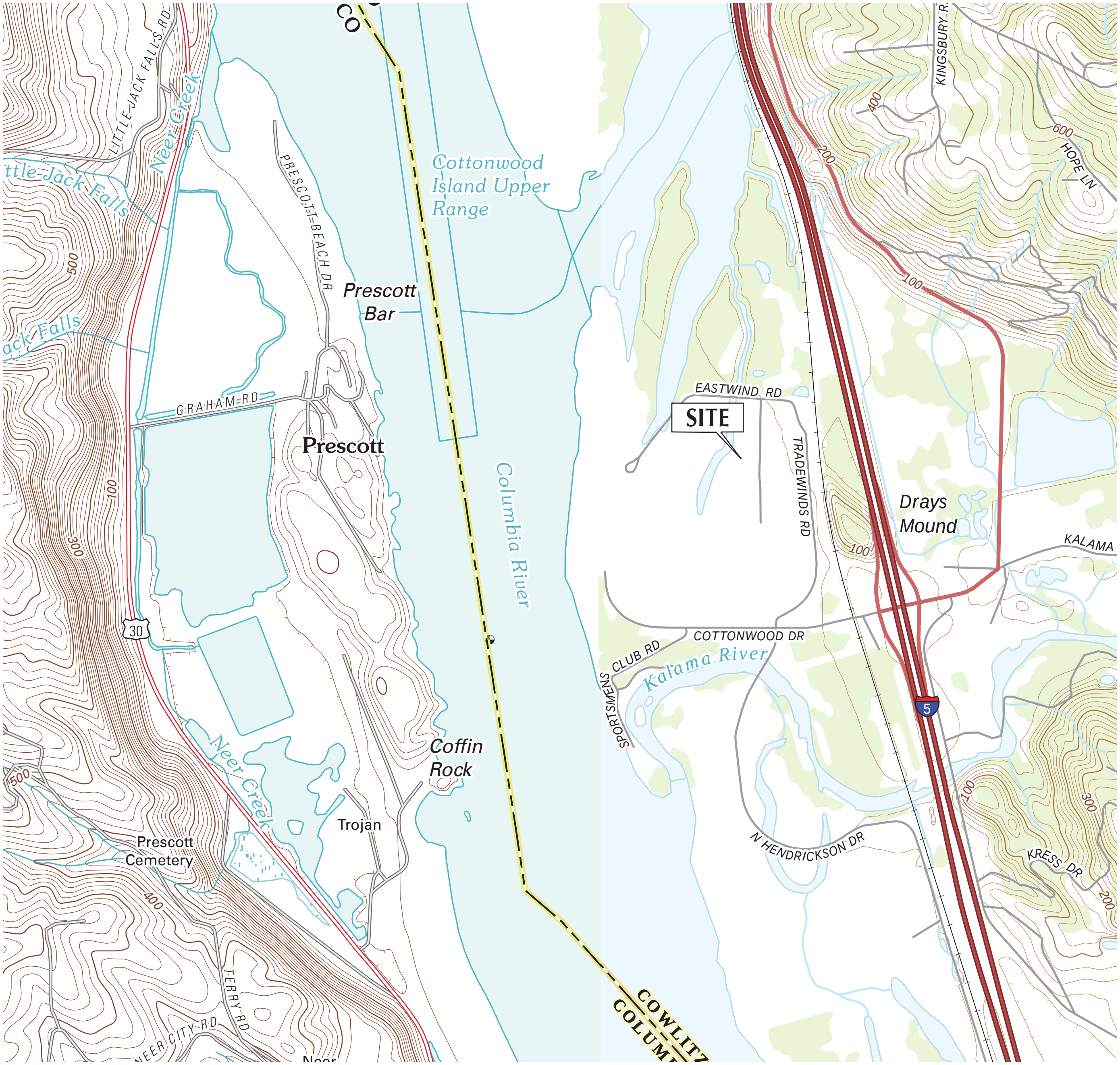
Youd, T.L., Hansen, C.M., and Bartlett, S.F., 2002, Revised multilinear regression equations for prediction of lateral spread displacement: Journal of Geotechnical and Geoenvironmental Engineering, ASCE, vol. 130, no. 8, pp 861-871.

W1159 PRELIMINARY MEMO



USGS TOPOGRAPHIC MAP
RAINIER, OREGON (2014), AND KALAMA, WASHINGTON (2013)

North

0 1/2 1 MILE

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NORTHWEST INNOVATION WORKS
KALAMA METHANOL PLANT

VICINITY MAP

FEB. 2015 JOB NO. W1159 FIG. 1