BEFORE THE STATE OF WASHINGTON
ENERGY FACILITY SITE EVALUATION COUNCIL

In the Matter of: Application No. 2013-01 TESORO SAVAGE, LLC TESORO SAVAGE DISTRIBUTION TERMINAL	CASE NO. 15-001 SWORN PRE-FILED TESTIMONY OF MATTHEW S. SHANAHAN

I, Matthew S. Shanahan, state as follows:

1. I swear under the penalty of perjury of the laws of Washington and the United States that the following statements are true and correct.

2. I am over eighteen years of age and am otherwise competent to testify in this case. My testimony is based upon my education, training, experience, professional qualifications, and understanding of the matters herein.

I. INTRODUCTION, EDUCATION AND PROFESSIONAL BACKGROUND, OTHER QUALIFICATIONS, AND PURPOSE OF TESTIMONY

3. I am a Principal with Geotechnical Resources, Inc. ("GRI"). A copy of my Curriculum Vitae is attached hereto as Attachment A.

4. GRI is a full-service geotechnical engineering and engineering geology firm with supporting expertise in environmental services, GIS, and pavement design and evaluation. The firm was founded in 1984 and operates from offices in Vancouver, Washington, and Beaverton and Brookings, Oregon. GRI currently has a total staff of 38, with a technical staff of 33 geotechnical engineers, geologists, and hydrogeologists.

5. Since 1984, GRI has completed over 5,600 projects, which includes more than 50 projects for the Port of Vancouver and hundreds of other projects for ports along the Columbia and Willamette Rivers and Pacific coast. GRI is very familiar with the

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subsurface, shoreline, and environmental conditions at the Port, existing Port facilities, and associated considerations for project development, design, and construction.

6. The purpose of my testimony is to provide testimony regarding Applicant Tesoro Savage Petroleum Terminal LLC, d/b/a Vancouver Energy's (hereinafter, "VE" or the "Applicant") Application for Site Certification ("ASC") for the Vancouver Energy Terminal ("the Project") and its compliance with Washington Building Code (including the International Building Code) and all other applicable standards and regulations.

7. I have reviewed relevant portions of the Applicant proposed ASC, Preliminary Draft Environmental Impact Statement ("PDEIS"), and associated plans, as well as the Draft Environmental Impact Statement ("DEIS") and key comments thereto to form my opinions and testimony.

8. GRI and my opinions and conclusions on the geotechnical analysis and design criteria based on current geologic conditions are contained in the report attached hereto as Attachment B, which is incorporated herein by reference.

9. GRI was retained to support permitting and design efforts for the Project at the Port of Vancouver, Washington.

II. PURPOSE OF TESTIMONY

10. The purpose of my testimony is to provide information on the geotechnical aspects of the project. As explained below, based upon my review of the application and available information, the design recommendations provided by GRI on the basis of its geotechnical analysis is intended to protect the public interest with respect to risks from geological or soil hazards.

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III. GEOTECHNICAL EVALUATION OF THE PROJECT SITE

A. Task 1 – Assist with EFSEC Site Certification Application

11. GRI analyzed the existing environment, project impacts, and mitigation measures for elements related to the Earth including:

- Review the results of a geologic survey describing conditions at the site, the nature of foundation materials, and potential seismic activities.
- Identify soil type present.
- Identify original topography and any changes likely to occur as a result of the Project construction and related activities.
- Identify unusual or unique geologic or physical features in the Pproject area or areas potentially affected by the Project.
- Identify potential for erosion, deposition, or change of any land surface, shoreline, beach, or submarine area due to construction activities, placement of permanent or temporary structures, or changes in drainage resulting from construction or placement of facilities associated with construction or operation of the Project.

12. GRI completed sufficient analyses to develop geotechnical criteria to describe foundation support methods, site preparation, earthwork, seismic hazard mitigation, berm construction, and other necessary geotechnical design information. As noted above, geotechnical design recommendations were developed as part of the Task 2 geotechnical investigation and design report. When the ground improvement design experts, structural engineers and designers, and others reference "design criteria" they are referring to this portion of GRI's work and opinion.

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B. Task 2 – Geotechnical Investigation for Project Design

13. GRI relied on the investigations and information obtained from the actions described below to analyze and conclude how each element of the Project complies. In the event that an element failed to comply, GRI analyzed potential impacts and mitigation for such impacts as it relates to soil inputs for ground improvements and foundation design.

14. GRI reviewed existing information, field explorations, laboratory testing, and engineering analysis.

15. Ten borings in the locations of the planned storage tanks, to the top of the underlying gravel unit, and five extending about 20 feet into the gravel to depths of about 80 feet. GRI obtained disturbed split-spoon samples and/or undisturbed Shelby tube samples from the borings at about 2.5 foot intervals of depth in the upper 15 ft and at 5-ft intervals below this depth. GRI conducted the Standard Penetration Test while the disturbed split-spoon samples were being taken.

16. GRI installed standpipes or vibrating-wire piezometers in three of the borings to permit measurements of the groundwater level. The piezometers were protected at the ground surface with a metal monument; a data logger will be installed to monitor groundwater levels if vibrating-wire piezometers are used.

17. Five electric cone penetration test ("CPT") probes were made in the locations of the planned storage tanks to a depth of about 65 feet or practical refusal of the CPT probe. The purpose of the electric CPT data evaluates the liquefaction potential of the deposit and for the design of pile foundations. The CPT is a static penetrometer used for soil explorations. In use, a hardened steel cone is forced vertically into the soil at a constant rate of penetration by a slow push or static thrust. The thrust required to cause penetration at a constant rate can be related to the bearing capacity of the soil immediately surrounding the point of the penetrometer cone. In addition to the cone, measurements are

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obtained of the magnitude of thrust required to force a special friction sleeve, attached above the cone, through the soil. The thrust required to move the friction sleeve can be related to the undrained shear strength of fine-grained soils and the frictional resistance of cohesionless soils. The dimensionless ratio of sleeve friction to point bearing capacity provides an indicator of the type of soil penetrated. The electric CPT is equipped with a variety of electronic sensors that permit measurement of the magnitude and dissipation rate of the shear-induced soil porewater pressure and the shear wave velocity. Shear wave velocity measurements will be made for use in the seismic studies.

18. GRI also made two borings to about 50 feet in the location of the planned boiler plant, and obtained disturbed split-spoon samples and/or undisturbed Shelby tube samples will be obtained from the borings at about 2.5-ft intervals of depth in the upper 15 feet and at 5 foot intervals below this depth. GRI conducted the Standard Penetration Test while the disturbed split-spoon samples were being taken.

19. GRI also completed three borings in the railcar unloading area. One boring was advanced to the top of gravel, estimated to be at a depth of about 80 feet. The two remaining borings were advanced to a depth of about 20 feet. GRI took disturbed split-spoon samples and/or undisturbed Shelby tube samples from the borings at about 2.5foot intervals of depth in the upper 15 feet and at 5foot intervals below this depth. It conducted the Standard Penetration Test while the disturbed split-spoon samples are being taken.

20. GRI took one 20-oo-deep boring for the modular office building, and ten 15-foot-deep borings will be completed along the pipeline. It obtained disturbed split-spoon samples and/or undisturbed Shelby tube samples from the boring at about 2.5-foot intervals of depth in the upper 15 feet and at 5-foot intervals below this depth. It

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conducted the Standard Penetration Test while the disturbed split-spoon samples are being taken.

21. GRI conducted laboratory tests to provide data on the important physical characteristics of the subsoils, essential for engineering studies and analyses. The laboratory tests included standard classification tests, such as natural water content and unit weight determinations, as well as strength and consolidation testing. The latter will provide the quantitative data necessary for the various foundation design studies, such as foundation types and estimated settlements.

22. GRI conducted engineering studies and analyses including: (1) general suitability of the site with respect to geotechnical considerations and the project design; (2) earthwork, including cut and fill slopes, wet-weather construction, and the suitability of on-site soils for use as structural fill; (3) shallow foundation design for lightly loaded structures, including allowable bearing pressures, modulus of subgrade reaction, and estimated settlements; (4) tank support, including estimated settlements (total and differential) and time rate of settlement, deep foundation types and design criteria if necessary, preliminary design criteria for ground improvement; (5) seismic design criteria, including a Site Class in accordance with the current American Society of Civil Engineers ("ASCE") 7 Standards; (6) subdrainage requirements; (7) pavement design; (8) design lateral earth pressures and coefficient of base friction; and (9) design criteria for temporary excavation shoring systems.

23. GRI completed a site-specific seismic hazard study for the facility to address the requirements of the current ASCE 7-10 Standards. The study included a review of the potential seismicity of the site, development of the ground response for the site during the appropriate design earthquakes, and evaluation of potential geologic hazards. The seismic hazard study included the following tasks:

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- Review of the literature, including published papers; maps; open-file reports; seismic histories and catalogs; works in progress; and other sources of information regarding the tectonic setting, regional and local geology, and historical seismic activity that might have a significant effect on the site.

- An examination and evaluation of the subsurface data for the site and vicinity, with particular emphasis on the potential for amplification or incoming seismic energy and liquefaction.

- Office studies and analyses that will lead to the preparation of conclusions and recommendations concerning: (1) seismic events that might have a significant effect on the site, including the proximity and potential seismicity of known faults; (2) the potential for site-specific seismic energy amplification at the site; (3) the ground response analysis for design earthquakes, which will include estimates of the peak horizontal ground acceleration; and (4) conclusions regarding seismic hazards, including liquefaction, lateral spreading, slope instability, ground rupture, and ground shaking.

C. Task 3 – Geotechnical Investigation Berth 13 and 14 Dock Modifications

24. GRI also assisted in design of modifications to the dock, working closely with Scott McCormack and the dock modification design team at BergerABAM to ensure that the design took into account the soil conditions in that area of the facility (Area 400). This soil-structure interaction ("SSI") design took into account the sloping, layered liquefiable soil profile; the potential for lateral spreading; and multi-hazard level MOTEMS-based design. GRI completed two borings in the vicinity of the Berth 13 trestle. One boring was located along the top of the riverbank. A second boring was drilled between Bents 2 and 4 adjacent to the Berth 13 trestle. The second boring was drilled from the beach during low water. The riverbank boring was advanced to the top of the underlying gravel. The boring drilled adjacent to the Berth 13 trestle was drilled a minimum of 20 feet into the underlying gravel and up to 40 feet into the gravel if possible

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during the 2 days of drilling scheduled. The Standard Penetration Test was conducted while the disturbed split-spoon samples are being taken.

25. One CPT probe was made near the riverbank between Berths 13 and 14. The probe was advanced to the top of the underlying gravel at a depth of about 100 feet. Shear wave velocity measurements were made for use in the seismic studies.

26. Laboratory tests were conducted to provide data on the important physical characteristics of the subsoils, essential for engineering studies and analyses. The laboratory tests included standard classification tests, such as natural water content, unit weight, grain size, and Atterberg indices determinations.

27. Ground surface seismic design criteria were provided for two seismic hazard levels in accordance with MOTEMS and for ground motions that are in accordance with the current International Building Code ("IBC"). GRI evaluated the site response for two representative soil profiles. One soil profile was representative of the trestle abutment, and a second representative of the dock. Both analyses were completed using a non-linear site response program.

28. Estimates and patterns of lateral spreading and seismic-induced earth pressures were provided for the trestle piles and abutment. GRI concluded that the soil in the Berth 13 trestle area would benefit from ground improvement, and worked collaboratively with Hayward Baker, the ground improvement specialist for the Project, to ensure their ground improvement design took into account the current site conditions confirmed by our study, detailed above.

29. GRI completed studies and prepared a report addressing the earth-related elements of the project as required by the City of Vancouver ("City") and other local standards and the Energy Facility Site Evaluation Council ("EFSEC"). These included the

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Washington Building Code and ASCE standard Seismic Design of Pile-Supported Piers and Wharves.

30. GRI evaluated static and seismic slope stability.

31. GRI made recommendations for pile axial compression and uplift capacities and lateral resistance during both static and seismic conditions for the selected pile types and sizes. The axial loading included downdrag effects where appropriate and provided recommended factors of safety for static and seismic conditions. GRI also provide LPILE parameters to determine lateral resistance.

32. GRI also provided a tip resistance Q-Z curve for the underlying gravel layer, where strength loss is not anticipated during the design-level earthquakes.

III. ANALYSIS OF EXISTING ENVIRONMENT AT THE SITE

A. Geology

33. On a regional scale, the Port of Vancouver lies within the Willamette-Puget Sound lowland trough of the Cascadia Subduction Zone ("CSZ"), an active convergent tectonic plate boundary. The subduction zone is a broad, eastward-dipping zone of contact between the upper portion of the subducting Juan de Fuca tectonic plate and the over-riding North American Plate. The convergent tectonic forces have generated northwest-trending fault zones and crustal blocks, resulting in areas of uplifted mountainous terrain and depressed structural basins such as the Portland Basin (Orr and Orr 1996).

34. The Portland Basin is a northwest-elongated structural basin bordered by the foothills of the Cascade Mountains to the east, the Tualatin Mountains to the west, the Clackamas River to the south, and the Lewis River to the north (Evarts et al. 2009). The Portland Basin began to form about 20 million years ago with folding and uplift of Tertiary basement rocks. About 15 to 16 million years ago, flood-basalt flows of the Columbia River Basalt Group ("CRBG") entered the basin (Beeson et al. 1989). CRBG

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consists of numerous dark gray to black, dense, crystalline basalt lava flows. By 14 million years ago, the uplift of the Tualatin Mountains diverted the Columbia River northward (Evarts et al. 2009).

35. Sandy River Mudstone is fine-grained river and lake sediment deposited into the subsiding Portland Basin by the Columbia River (Trimble 1963). Overlying the Sandy River Mudstone is consolidated and cemented sandstone and conglomerate of the Troutdale Formation (Tolan and Beeson 1984). The Troutdale Formation (Tt) was eroded during the last ice age by the ancestral Columbia and Willamette Rivers and by catastrophic glacial outburst floods (Allen et al. 2009). Glacial outburst floodwaters from Montana washed across eastern Washington and through the Columbia River Gorge to spread out in the Portland Basin, depositing boulders, cobbles, and gravel sediment grading to thick blankets of sand. This deposit is subdivided into two facies by Phillips (1987): a fine-grained facies (Qff) that consists of primarily coarse sand to silt and a coarse-grained facies (Qfc) that consists of pebble to boulder gravel with a coarse sand to silt matrix. The sea level rose by about 300 feet after the last of the glacial outburst floods about 15,000 years ago, forming an estuary environment that extends far upstream in the Columbia River. These low-energy environments rapidly filled with Holocene sandy alluvium (Qal) and broad floodplains developed along the primary Columbia River channel (Peterson et al. 2011).

B. <u>Surface Soils</u>

36. Soil types in the vicinity of the location of the Porject have been identified by NRCS (McGee 1972). Since the soil survey was completely approximately 40 years ago, industrial activities have significantly modified the soils within the project footprint. Surface soil at the Project site likely consists entirely of modified or fill material and may no longer resemble soils described by NRCS in the 1972 Clark County Soil Survey.

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C. Topography of the Site

37. The ground surface in the upland portion of the project Proposed Action area is relatively flat and ranges from about elevation 22 to 35 feet (National Geodetic Vertical Datum of 1929 [NGVD29]). The riverbank near the dock area slopes down from the top of the bank at about a horizontal run to vertical rise of 2:1 horizontal (H) to 1 vertical (V) (51 percent%) to a more level beach below the OHWL mark.

D. Analysis of Seismic Hazards

i. Discussion of conclusions regarding ground loading criteria

38. Ground motion/shaking and prolonged earthquake ground shaking has the potential to damage buildings, pipelines, or storage tanks. Ground motion is shaking that is caused by seismic waves during an earthquake. The Project is located in an area that has the potential for strong earthquake ground motion. The potential ground motion during an earthquake event is generally represented by horizontal peak ground motion acceleration ("PGA") and is expressed in gravity units (g). The ground motion hazard depends on the magnitude of an earthquake, its distance, and the subsurface conditions. Based on average seismic shear wave velocity and average field standard penetration resistance in the upper 100 feet, a PGA ranging from 0.28 to 0.34 g was estimated for a 475-year return period earthquake (10% probability of exceedance in 50 years), and a PGA ranging from 0.37 to 0.45 g was estimated for a 2,475-year return period earthquake (2% probability of exceedance in 50 years).

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IV. DESIGN CRITERIA AND COMPLIANCE OF PROJECT FACILITY DESIGN WITH APPLICABLE STANDARDS

A. Design Criteria

39. Based on the above analysis of current conditions, including seismic hazard, at the Project site, GRI established geotechnical design recommendations for each element of the Project design and construction.

40. GRI provided geotechnical seismic recommendations in accordance with applicable design standards and building codes for use by the designer to reduce the likelihood of negative impacts from ground motion. Engineers use seismic design parameters to develop specific levels of structure performance during an earthquake. The performance levels range from prevention of collapse to protect human lives to designing structures that continue to function at a high level immediately following the earthquake.

41. The Applicant has stated that the oil tanks in the storage area (Area 300) would be designed to the seismic provisions in Annex E of the twelfth edition of the API 650 standard, which is aligned with the ASCE 7-10 standard (BergerABAM 2014). The upland aboveground facilities, other than the oil storage tanks, would meet the provisions of IBC 2012, which incorporates the ASCE 7-10 standard by reference. Marine terminal dock modifications in Area 400 would conform to the IBC 2012, as amended and adopted by the State of Washington and the City, with the exception of mooring and berthing design, structural load combinations, and seismic design. The seismic design of piers and wharves not accessible to the general public is beyond the scope of the ASCE 7-10 standard. The recently released ASCE 61-14 standard, Seismic Design of Piers and Wharves, would be used for the seismic design of such structures in the marine terminal (Area 400). If these design standards are implemented, the risk of severe structural damage or failure of Facility elements from facility shaking resulting from earthquake

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ground motion associated with a great earthquake on the CSZ or other lesser earthquakes from the CSZ or other faults would be reduced. It is important to note, however, that while engineering design can reduce the adverse effects of the anticipated design earthquake event, the risk is never completely eliminated irrespective of design and construction used at a site.

42. Geotechnical engineering studies were completed to evaluate soil layers underlying the proposed Facility site. These soils primarily comprise silt and sand of varying strength down to approximately 60 to 100 feet bgs. Some of these soils fall within the National Earthquake Hazards Reduction Program s ("NEHRP") site, Class F, meaning they are unstable soils prone to liquefaction during very strong ground motion. These soils are underlain by the dense Troutdale gravel, which falls within NEHRP site, Class C, meaning the gravel is very resistant to liquefaction. Given potential site ground motions (without proposed ground improvements), liquefaction-induced settlements of the Site Class F soils could be approximately 10 to 16 inches in the vicinity of the unloading and office area (Area 200) and the boiler building (Area 600); 6 to 10 inches in the storage area (Area 300); 3 to 15 inches in the vicinity of the transfer pipelines (Area 500); and 12 to 24 inches at the marine terminal (Area 400) (GRI 2013).

43. Structures that may otherwise withstand ground movement could be damaged if underlying soils liquefy. Geotechnical assessments of the proposed Facility location (URS 2011a, GRI 2013, Hayward Baker 2014) have concluded that soils in portions of the site could experience liquefaction during an earthquake, resulting in significant dynamic settlement and lateral spreading deformations in some areas, especially near the riverbank. Ground settlement was estimated to be approximately 10 to 16 inches in the unloading and office area and the boiler building (Areas 200 and 600), 6 to 10 inches in the storage area (Area 300), 3 to 15 inches in the transfer pipelines (Area

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500), and 12 to 24 inches in the marine terminal (Area 400) (GRI 2013). Such settlement has the potential to damage overlying structures, including buildings, pipelines, or storage tanks. Estimates of lateral spread magnitudes at the shoreline for Terminal 5 may be up to approximately 12 feet at that site (URS 2011b), which could impact slope stability along the banks of the Columbia River.

44. GRI's analysis and conclusions above provided completed sufficient analyses to develop geotechnical criteria to describe foundation support methods, site preparation, earthwork, seismic hazard mitigation, berm construction, and other necessary geotechnical design information. As noted above, detailed geotechnical design recommendations were developed as part of the Task 2 geotechnical investigation and design report.

45. If these design standards are implemented, the risk of severe structural damage or failure of Facility elements from facility shaking resulting from earthquake ground motion associated with a great earthquake on the CSZ or other lesser earthquakes from the CSZ or other faults would be minor. It is important to note, however, that the risk is never completely eliminated irrespective of design and construction used at a site.

46. GRI summarized seismic design criteria for the facility in geotechnical reports for the upland area of the facility, and for the dock modifications, and provided a general summary of seismic hazards in the earth chapter. These documents were provided to the project design team for use in seismically designing the facility. GRI established design recommendations for use by the team to design for the code based earthquake. Seismic design of the project elements, except the dock structure, will be in accordance with the 2012 International Building Code (2012 IBC), which was recently adopted by the Washington State Building Code and incorporates recommendations from the ASCE 7-10, Minimum Design Loads for Building and Other Structures (ASCE 7-10). The 2012 IBC

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and ASCE 7-10 seismic hazard levels are based on a Risk-Targeted Maximum Considered Earthquake (MCE_R). The ground motion associated with the probabilistic MCE_R represents a targeted risk level of 1% in 50 years probability of collapse in the direction of maximum horizontal response. In general, these risk-targeted ground motions are developed by applying adjustment factors of directivity and risk coefficients to the 2% probability of exceedance in 50 years, or a 2,475-year return period hazard level ground motion developed from the 2008 U.S. Geologic Survey ("USGS") probabilistic seismic hazard maps. Analysis identified a potential risk of liquefaction throughout the site. In accordance with ASCE 7-10, sites with subsurface conditions identified as vulnerable to failure or collapse, such as liquefied soils, shall be classified as NEHRP site, Class F. For Class F sites, ASCE 7-10 section 20.3 requires completion of a site-specific ground motion analysis unless the structures have a fundamental period of vibration less than or equal to 0.5 second. The range of fundamental periods of the structures in areas 300 and 400 exceeds 0.5 second and, therefore, for the potentially liquefiable soil profiles in Areas 300 and 400, a site-specific ground motion analysis was completed with the aid of the computer software D-MOD, a non-linear seismic soil response software developed by GeoMotions, LLC. As a result of the liquefaction and lateral spread estimates, the Project design will include pile foundations and ground improvement to reduce the magnitude of liquefaction and lateral spread.

47. I have attached the following exhibits to my testimony:

Attachment A: Matthew S. Shanahan, PE, Curriculum Vitae

Attachment B: GRI Report

[Signature on the Following Page]

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DATED this 13 day of May, 2016.

Matthew S. Shanahan, Declarant

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