February 10, 2015

BergerABAM
33301 9th Avenue South, Suite 300
Federal Way, WA 98003-6395

Attention: Darrell Joque

SUBJECT: Review and Update of Seismic and Geotechnical Recommendations
Kalama North Port Methanol Plant Dock
Port of Kalama, Washington

At your request, GRI has reviewed and updated the results of our geotechnical investigation and site-specific seismic hazard study for the proposed methanol plant dock at the North Port terminal at the Port of Kalama, Washington. The site is located on the Washington shore of the Columbia River, downstream from the mouth of the Kalama River, at the general location shown on the Vicinity Map, Figure 1.

The results of our original study are summarized in our April 25, 2003, report to BergerABAM entitled, “Geotechnical Investigation and Site-Specific Seismic Hazards Study, Proposed Dock Expansion for North Port Terminal, Port of Kalama, Washington.” GRI also conducted a geotechnical investigation for the existing dock, which is documented in our June 14, 1990, report to the Port of Kalama entitled, “Foundation Investigation, North Port Marine Terminal, Proposed Dock Structure, Port of Kalama, Kalama, Washington.”

The original design for the proposed dock expansion was completed in accordance with Marine Oil Terminal Engineering and Maintenance Standards (MOTEMS) guidelines. We understand the dock expansion will now be designed in accordance with ASCE 60-41, Seismic Design of Pile-Supported Piers and Wharves.

This investigation consisted of reviewing available geotechnical information for the site (GRI, 1993 and 2003; PBS, 2006), a site-specific seismic hazard study in accordance with ASCE 60-41, and updating the results of previous engineering studies and analyses to the new standards. This report describes the work accomplished and provides our revised conclusions and recommendations for the design and construction of the proposed dock.

PROJECT DESCRIPTION
As shown on the Site Plan, Figure 2, the proposed dock will be located downstream of the existing dock and will be about 525 ft long and between 24 and 100 ft wide. An approximately 363-ft-long by 34-ft-wide trestle will provide access to the new dock from the shore. Mooring dolphins will be constructed downstream and upstream from the proposed dock, and two breasting dolphins will be constructed along the face of the dock. The mooring and breasting dolphin piles and several piles for the main dock will be installed at a 4H:12V (horizontal to vertical) batter. We understand 24-in. solid octagonal precast, prestressed concrete piles will be used to support the dock, dolphins, and trestle piles, and the maximum
factored load will be about 340 tons for compression and 40 tons for uplift. Fender piles will consist of open-ended steel pipe piles that will be installed using vibratory methods. The deck of the dock and trestle will be at elevation +18 ft Columbia River Datum (CRD). The portion of the Columbia River beyond the riverward face of the main dock will be dredged at an inclination of 3H:1V to elevation -48 ft CRD. Evaluation of the riverbank stability with respect to the preliminary dredge design is not part of our scope for this project; however, we assume the stability will be evaluated during final design.

SITE DESCRIPTION
Topography and Bathymetry
Available topographic information indicates the ground surface behind the top of the riverbank is about elevation +20 ft CRD. A relatively steep wave-cut slope separates the top of the riverbank from a gently sloping beach along the shore of the river. The slope of the river bottom between the beach and the face of the dock is typically at about 4H:1V or flatter. The river bottom at the dock currently ranges from about elevation -35 to -40 ft CRD.

The project site is located just downstream of Columbia River Mile 72, where the CRD is 0.12 ft above the 1947 adjusted National Geodetic Vertical Datum (NGVD 47). Information provided by BergerABAM indicates the mean lower low water (MLLW) and ordinary high water (OHW) levels at the site correspond to 0 ft and +11.6 ft CRD, respectively. The 100-year flood level at the site corresponds to elevation +19.3 ft CRD.

Geology
Available geologic literature and review of existing geotechnical information indicates the shore of the river is mantled with hydraulically placed dredged sand fill. The fill is underlain by floodplain deposits that typically consist of silt and clayey silt with lenticular interbeds of sandy silt and silty sand. The floodplain deposits are underlain by river channel deposits that typically consist of fine- to medium-grained sand with lenses of gravel. The log of a water well drilled at the site in 1989 indicates the river channel deposits extend about 300 ft below the upland elevation. The river channel deposits are underlain by basalt; the upper surface of the basalt is weathered. Geologic mapping in the area indicates the basalt is underlain by marine sedimentary deposits at a depth of more than 1,000 ft.

SUBSURFACE CONDITIONS
General
As part of our original geotechnical investigation for the dock, soil and groundwater conditions were investigated in 2002 with four borings, designated B-1 through B-4. The approximate locations of the borings are shown on Figure 2. Borings B-1 and B-2 were drilled on shore, and borings B-3 and B-4 were drilled over water from a barge near the face of the proposed dock. The onshore and offshore borings were drilled to a total depth of 101.5 ft beneath the ground surface or mudline, respectively. Summary boring logs and laboratory test results developed during our previous field investigation and laboratory testing programs for this study are provided in Appendix A.

Soils
For the purpose of discussion, the soils disclosed during the subsurface investigation have been grouped into the following categories based on their physical characteristics and engineering properties.
1. **FILL**
2. **Interbedded SAND and SILT**
3. **SAND**

The following paragraphs provide a detailed description of these soil units and a discussion of the groundwater conditions at the site.

**1. FILL.** The ground surface at the location of landside borings B-1 and B-2 is mantled with fill that primarily consists of dredged sand. The sand is typically gray, fine to medium grained, and contains a trace of silt and scattered pumice pebbles or thin layers of coarser grained sand or gravel. The fill is estimated be about 17 and 14 ft thick at the locations of boring B-1 and B-2, respectively. The relative density of the fill is generally loose to medium dense based on standard penetration test N-values in the range of 3 to 19 blows/ft. The natural moisture content of the fill ranges from about 19 to 29%.

**2. Interbedded SAND and SILT.** The fill is underlain by interbedded alluvial deposits of sand and silt to a depth of about 46 to 47 ft at the locations of landside borings B-1 and B-2. The sand is typically gray and fine grained and contains varying percentages of silt, ranging from a trace of silt to silty. The silt is typically gray and contains varying percentages of sand, ranging from some sand to sandy. Scattered fine organic material and small woody debris were also encountered within the deposit. Based on N-values ranging from about 3 to 11 blows/ft, the relative density of the sand is generally loose to medium dense, and the relative consistency of the silt is generally soft to medium stiff. Torvane shear strength values for the silt are typically in the range of about 0.15 to 0.35 tsf. The natural moisture content of the sand and silt ranges from about 29 to 50%.

**3. SAND.** Offshore borings B-3 and B-4 disclosed sand from the mudline to the maximum depth explored, 101.5 ft. Upland borings B-1 and B-2 encountered the sand below depths of 46 to 47 ft. The sand is gray, fine to medium grained, and is generally clean or contains a trace of silt, except in the uppermost 5 ft of the deposit, which may contain some silt. The sand is interbedded with thin layers of subrounded gravel-size pumice between about elevation -63 and -96 ft CRD. Occasional organic debris was also encountered in the sand, including a zone of woody debris up to 2.5 ft thick in boring B-3. A thin layer of volcanic ash was disclosed at a depth of 80.5 ft in boring B-4. N-values ranging from 6 to 42 blows/ft indicate the relative density of the sand ranges from loose to dense and gradually increases with depth. The natural moisture content of the sand typically ranges from about 10 to 35%.

**Groundwater**

Groundwater levels at the site will fluctuate in response to varying river levels, but will tend to lag somewhat in time. Groundwater may also tend to perch seasonally within portions of the dredged sand cap above the less-permeable silt soils.

**CONCLUSIONS AND RECOMMENDATIONS**

**General**

The landward portion of the site is mantled with sand fill. The fill in the vicinity of the planned trestle abutments has an average thickness of about 16 ft and is underlain by layered deposits of alluvial silt and sand that are underlain by medium dense sand below about elevation -25 ft (CRD). The river bottom along the dock expansion is underlain by sand. We understand the dock will be designed in accordance with
the ASCE 60-41. The results of our investigation, which included a site-specific seismic hazard study, indicate that soil located above elevation -60 ft could potentially liquefy during a Contingency Level Earthquake (CLE) and Design Earthquake (DE), resulting in some settlement and lateral ground displacement toward the river. Liquefaction would also result in a reduction in axial and lateral support of the piles and settlement-induced downdrag loads. The following sections of this report provide our conclusions and recommendations for design and construction of the project.

**Seismic Design Considerations**

**Spectral Response.** ASCE 60-41 defines ground motions for three seismic hazard levels: the Operating Level Earthquake (OLE), the Contingency Level Earthquake (CLE), and the Design Earthquake (DE), which are defined below.

OLE is defined by 50% probability of exceedance in 50 years, which corresponds to an earthquake with an expected recurrence interval of 72 years and represents a performance level with minimal structural damage.

CLE is defined by 10% probability of exceedance in 50 years, which corresponds to an earthquake with an expected recurrence interval of 475 years, and represents a performance level of controlled and repairable structural damage.

DE is defined per ASCE 7-05, which develops the response spectra based on ground motions associated with the Maximum Considered Earthquake (MCE), which is generally represented by a probabilistic earthquake with a 2% probability of exceedance in 50 years (return period of about 2,500 years), except where subject to deterministic limitations (Leyendecker and Frankel, 2000). The design-level response spectrum that represents the DE is obtained by taking two-thirds of the MCE level ground motions.

The bedrock earthquake motions for each of the hazard levels were selected from the 2008 U.S. Geologic Survey (USGS) probabilistic seismic hazard maps for the coordinates of 46.05° N latitude and 122.88° W longitude. The code-based spectra are developed using two spectral response coefficients, $S_s$ and $S_1$, corresponding to periods of 0.2 and 1.0 second. These bedrock spectral ordinates are adjusted for Site Class with the short- and long-period site coefficients, $F_s$ and $F_v$, based on subsurface conditions or with a site-specific response analysis. The code-based OLE, CLE, and DE hazard level $S_s$ and $S_1$ coefficients for the dock are provided in Table 1.

<table>
<thead>
<tr>
<th>Hazard Level</th>
<th>$S_s$</th>
<th>$S_1$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Operating Level Earthquake (OLE)</td>
<td>0.12</td>
<td>0.04</td>
</tr>
<tr>
<td>Contingency Level Earthquake (CLE)</td>
<td>0.44</td>
<td>0.16</td>
</tr>
<tr>
<td>Design Earthquake (DE)</td>
<td>0.86</td>
<td>0.34</td>
</tr>
</tbody>
</table>

Per Section 20.4 of ASCE 7-05, the soil profile in the dock area is generally designated as Site Class D based on average N-values in excess of 15 blows per foot in the upper 100 ft. In the upland portion of the site, i.e., along the trestle, the soil profile would classify as Site Class E because of the upper looser/softer interbedded layers of sand and silt.
Our analysis has identified a potential risk of liquefaction for the CLE and DE hazard levels. In accordance with ASCE 60-41, sites with subsurface conditions identified as vulnerable to failure or collapse, such as liquefiable soils, are classified as Site Class F. For Site Class F sites, Section 4.3.2 of ASCE 60-41 requires a site-specific ground motion analysis for structures with a fundamental period of vibration greater than 0.5 second. BergerABAM has indicated the fundamental period of the dock may be greater than 0.5 seconds. Due to these anticipated longer periods, a site-specific seismic ground motion analysis was completed for each of the three seismic hazard levels. The analysis was completed with the aid of the computer software D-MOD2000, a non-linear seismic soil response software developed by GeoMotions, LLC. The D-MOD2000 analyses are discussed in Appendix B.

The results of the site-specific response modeling were compared with Site Class D and E code-based spectra. The modeling indicates that 80% of code-based Site Class E spectral accelerations provide an appropriate estimate of the OLE hazard level spectral accelerations and corresponds to the minimum spectral acceleration allowed by ASCE 7-05 for liquefied conditions. The site-specific response modeling indicates that 80% of the code-based Site Class E spectral accelerations generally provide an appropriate estimate of the spectral accelerations for the CLE hazard level. Between periods of about 0.9 and 1.5 seconds, the ground motion analysis indicates the spectral acceleration is slightly greater than 80% of the code-based Site Class E spectral acceleration. Therefore, we recommend a slight increase over 80% of the code-based Site Class E spectral acceleration at these periods for the CLE hazard level.

The site-specific response modeling for the MCE hazard level indicates spectral accelerations are greater than 80% of Site Class E spectral accelerations at fundamental periods less than about 1.9 seconds. At periods less than 1.9 seconds, we conservatively recommend the design spectrum include an increase above Site Class E (but generally less than Site Class D) to envelop the estimated site-specific ground surface response. At periods greater than approximately 1.9 seconds, the site-specific response spectrum is less than 80% of the code-based site response spectrum for Site Class E, which we recommend using to estimate the spectral accelerations for the MCE hazard level at these longer fundamental periods. The DE is determined by taking two-thirds of the MCE. The recommended response spectra for each of the three hazard levels are tabulated below.

| Table 2: RECOMMENDED OLE, CLE DE HAZARD LEVEL SPECTRA, 5% DAMPING |
|-------------------------------|-------------------|-------------------|-------------------|
| Period, s         | OLE Level          | CLE Level          | DE Level          |
|                  | Spectral Values, g | Spectral Values, g | Spectral Values, g |
| 0                | 0.10              | 0.27              | 0.25              |
| 0.05             | 0.18              | 0.34              | 0.32              |
| 0.08             | 0.24              | 0.38              | 0.36              |
| 0.10             | 0.24              | 0.57              | 0.39              |
| 0.13             | 0.24              | 0.67              | 0.44              |
| 0.2              | 0.24              | 0.67              | 0.53              |
| 0.28             | 0.24              | 0.67              | 0.65              |
| 0.43             | 0.24              | 0.67              | 0.65              |
| 0.68             | 0.19              | 0.67              | 0.65              |
| 1                | 0.10              | 0.47              | 0.65              |
| 1.5              | 0.07              | 0.29              | 0.43              |
| 2                | 0.05              | 0.22              | 0.24              |
| 2.5              | 0.04              | 0.17              | 0.19              |
**Liquefaction.** Liquefaction is a process by which loose, saturated, granular materials, such as sand, and to a somewhat lesser degree soft, non-plastic and low-plasticity silts, temporarily lose strength during and immediately after a seismic event. Liquefaction occurs as seismic shear stresses propagate through a 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 as a solid. As strength is lost, there is an increased risk of settlement, lateral spread, and/or slope instability, particularly along waterfront areas. Liquefaction-induced settlement occurs as the elevated porewater pressures dissipate and the soil consolidates after the earthquake.

The potential for liquefaction is calculated by comparing the cyclic shear stresses induced within a soil profile during an earthquake to the ability of the soils to resist these stresses. For the dock portion of the project, the cyclic shear stresses developed within the soil profile was estimated from the D-MOD2000 analysis. The equivalent uniform stress profile obtained from D-MOD2000 is normalized by the vertical effective stress to develop a cyclic stress ratio (CSR) profile for each of the design earthquakes. The ability of the soils to resist these stresses, known as the cyclic resistance ratio (CRR), is based on soil strength as characterized by standard penetration test N-values normalized for overburden pressures and corrected for such factors as fines content in accordance with the recommendations of Idriss and Boulanger (2008). The factor of safety against liquefaction is then defined as the ratio of CRR to CSR. A similar approach was used to evaluate the factor of safety for the upland portion of the site, except the CSR was determined in accordance with Idriss and Boulanger rather than the results of our ground motion analysis.

The peak horizontal ground accelerations used in our evaluation were based on the results of our site-specific ground motion analysis. The earthquake magnitudes chosen to represent the earthquake hazard levels for our liquefaction studies were based on the 2008 USGS interactive deaggregations for the OLE, CLE, and DE return intervals as well as the results our site-specific ground motion analysis. The input values used for our liquefaction studies are provided in Table 3.

<table>
<thead>
<tr>
<th>Hazard Level</th>
<th>PHGA, g</th>
<th>Earthquake Magnitude, M</th>
</tr>
</thead>
<tbody>
<tr>
<td>Operating Level Earthquake (OLE)</td>
<td>0.10</td>
<td>6.2</td>
</tr>
<tr>
<td>Contingency Level Earthquake (CLE)</td>
<td>0.27</td>
<td>7.5</td>
</tr>
<tr>
<td>Design Earthquake (DE)</td>
<td>0.25</td>
<td>8.5</td>
</tr>
</tbody>
</table>

To evaluate the potential for liquefaction, we have assumed a groundwater level at elevation +6 ft, which corresponds to the average daily river level. Our analysis indicates the near-surface soil at the site is not

<table>
<thead>
<tr>
<th>Period, s</th>
<th>OLE Level Spectral Values, g</th>
<th>CLE Level Spectral Values, g</th>
<th>DE Level Spectral Values, g</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.0</td>
<td>0.03</td>
<td>0.14</td>
<td>0.16</td>
</tr>
<tr>
<td>3.5</td>
<td>0.03</td>
<td>0.12</td>
<td>0.14</td>
</tr>
<tr>
<td>4.0</td>
<td>0.03</td>
<td>0.11</td>
<td>0.12</td>
</tr>
<tr>
<td>5.0</td>
<td>0.02</td>
<td>0.09</td>
<td>0.10</td>
</tr>
</tbody>
</table>
susceptible to liquefaction during the OLE; however, the entire soil column below the groundwater table and above elevation -60 ft is potentially liquefiable during the CLE and DE. The effects of liquefaction, including seismically induced settlement, lateral spreading toward the river, reduction in pile capacity due to soil strength loss, and downdrag loads are discussed below.

**Seismically Induced Settlements.** We have estimated the potential for liquefaction-induced free-field settlement using the empirical methodologies developed by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992). The empirical methods are based on case histories of areas that have undergone liquefaction. For the upland portions of the site, we estimate 1 to 2 ft of seismically induced settlement may occur during the CLE and DE hazard levels. Where there is thinner zone of potentially liquefiable soil near the face of the proposed dock, we estimate seismically induced settlement could be about ½ ft during the CLE and DE hazard level. Settlements of this magnitude could induce downdrag loads on piles, which is discussed in the CLE and DE Hazard Pile Axial Capacity section of this report. Associated slope movement toward the river may also contribute to vertical ground surface displacements at the site.

**Lateral Spreading.** The methodology presented by Youd, et. al. (2002) was used to estimate liquefaction-induced lateral spreading toward the river. These studies indicate lateral spreading could be in the range of ¼ to 1 ft as a result of the CLE hazard level earthquake. Lateral displacement as a result of the DE hazard level earthquake could be in the range of 2 to 9 ft. It should also be acknowledged that the available analytical methods do not predict localized effects, such as flow failures, that may occur near the crest of slopes. The estimated horizontal displacements indicate that sufficient lateral movement could occur during the CLE and DE hazard level to induce full lateral spreading earth pressures on structural components, such as the trestle abutments and piles. These seismically induced lateral loads are further discussed in the Lateral Pile Capacity and Lateral Spread Loading section of this report.

**Abutment Design**

**Lateral Earth Pressures.** We understand the abutments for the trestles near the top of the riverbank will be supported on piles. Consequently, we assume the abutments will be relatively rigid, and we recommend they be designed for at-rest earth pressures. We further recommend backfilling the abutments with free-draining, granular structural fill to prevent the build-up of excessive unbalanced water pressures behind the abutments. We recommend that rigid walls be designed for an equivalent fluid weight of 60 pcf for static conditions. Additional lateral earth pressures due to surcharge loads, such as from heavy wheel loads behind the abutments, may be estimated using the methods provided on Figure 3. We recommend neglecting any passive earth pressures in front of the abutments due to the potential long-term effects of scour.

As discussed previously, the CLE and DE hazard level earthquake may induce more than 6 in. of lateral deformation toward the river. In our opinion, this estimated magnitude of embankment deformation requires consideration of alternative approaches for evaluation of seismic earth pressures greater than the typical seismic earth pressures calculated using Mononobe-Okabe that are commonly used for upland structures. For these hazard levels, the load imposed by movement of non-liquefied soil at the abutments can be calculated as a passive earth pressure based on an equivalent fluid pressure of 420 pcf. The reduced lateral earth pressures for liquefiable soil loads on piles below the water table are provided in the Design Estimates for Lateral Displacement Forces section of this report. Neither the at-rest earth pressure
nor any surcharge loads need to be added when applying the passive earth pressure for seismic design. For the OLE hazard level, the seismic lateral earth pressures are negligible and can be neglected in design.

**Wall Backfill and Compaction.** Fill placed behind the abutments should consist of sand, sandy gravel, or fragmental rock up to a maximum size of about 3 in. and having less than 5% passing the No. 200 sieve. Fill materials placed below water, if necessary, should consist of fragmental rock with a maximum size on the order of 4 to 6 in. and less than about 5% passing the 1/4-in. sieve. Depending on the gradation of the fill material used below water, it may be necessary to install a suitable filter material to prevent infiltration of the sandy material used as backfill above the water level. Fill should be placed in maximum 12-in.-thick loose lifts and be compacted to at least 95% of the maximum dry density as determined by ASTM D 698 (standard Proctor) at a moisture content within about 3% of optimum. To avoid overstressing the wall, structural fill placed within 3 ft of the back of the wall should be compacted using hand-operated compaction equipment. Thinner lifts may be necessary to compact the material to the required density if hand-equipment is used.

**Vertical Pile Support**

**General.** We have prepared recommended axial compression and tension capacities for 24-in. solid octagonal precast, prestressed concrete piles. The estimated ultimate capacities assume that groups of piles will have a minimum center-to-center pile spacing of at least three pile diameters.

**Static and OLE Hazard Pile Axial Capacity.** Figure 4 can be used to evaluate the anticipated penetration of piles necessary to achieve a given pile capacity. The plot provides the average ultimate unit skin friction ($q_s$) for solid concrete piles for up to 110 ft of penetration below the mudline or ground surface. Graphs of average ultimate unit skin friction versus depth below the mudline for the trestle piles and dock and dolphin piles are provided on Figure 4. The average ultimate unit skin friction value for a given pile location and penetration can be multiplied by the pile perimeter ($P_p$) and the embedded length of the pile ($L_s$) to obtain the ultimate skin friction capacity for the pile ($Q_s$). The ultimate uplift capacity at the static and OLE hazard level ($Q_{up}$) is equal to 70% of the ultimate skin friction capacity for the pile.

To limit liquefaction-induced settlement of the piles, we recommend driving the piles a minimum of 30 ft below the mudline or elevation -70 ft, whichever is deeper. Provided the piles achieve the recommended minimum embedment, the unit end bearing capacity of the driven concrete piles ($q_t$) can be assumed to be 75 tsf. This value should be multiplied by the pile tip area ($A_t$) to determine the ultimate pile end-bearing capacity ($Q_t$). The ultimate compressive capacity ($Q_{ult}$) of the piles is the sum of the skin friction capacity and the end-bearing capacity.

**CLE and DE Hazard Pile Capacity.** Our analysis indicates that the saturated soil located above elevation -60 ft is potentially liquefiable as a result of the CLE and DE hazard level earthquake. Liquefaction will result in settlement at the site and a temporary reduction in the axial pile capacity.

To analyze the piles in compression for the CLE and DE hazard levels, the portion of the ultimate skin friction capacity developed above elevation -60 ft ($Q_{s,red}$) should be subtracted from the total ultimate compressive capacity of the pile ($Q_{ult}$) to estimate the ultimate compressive capacity ($Q_{ult,seismic}$). The reduction in skin friction capacity ($Q_{s,red}$) is the static pile capacity above elevation -60 ft, which can be calculated as the product of the average unit skin friction ($q_s$) at the embedment depth corresponding to
elevation -60 ft, the pile perimeter \((P_s)\), and the embedded length of the pile above elevation -60 ft. The uplift capacity of the piles during seismic loading \((Q_{up,seismic})\) should be reduced in a similar manner as described above, except that 80% of the downdrag load can be used to resist uplift loads.

At the CLE and DE hazard level, seismically induced settlement will result in downdrag loads on the upper portion of piles. As shown in Table 4, the downdrag load is a function of the embedded length of the pile above elevation -60 ft, which is the maximum considered depth of liquefaction. Linear interpolation can be used to estimate downdrag loads for intermediate embedded lengths. The downdrag loads should be included with dead, live, wind, earthquake, etc., loads when sizing the piles for axial compressive capacity at the CLE and DE hazard level. Downdrag loads should not be subtracted from the reduced seismic pile capacity discussed above.

<table>
<thead>
<tr>
<th>Embedded Length of Pile Above Elevation -60 ft, ft</th>
<th>Total Downdrag Load, tons</th>
</tr>
</thead>
<tbody>
<tr>
<td>70</td>
<td>16.0 tons/ft x (P_s)</td>
</tr>
<tr>
<td>60</td>
<td>13.0 tons/ft x (P_s)</td>
</tr>
<tr>
<td>50</td>
<td>9.5 tons/ft x (P_s)</td>
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<tr>
<td>40</td>
<td>7.0 tons/ft x (P_s)</td>
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<tr>
<td>35</td>
<td>5.5 tons/ft x (P_s)</td>
</tr>
<tr>
<td>30</td>
<td>5.0 tons/ft x (P_s)</td>
</tr>
<tr>
<td>25</td>
<td>3.8 tons/ft x (P_s)</td>
</tr>
<tr>
<td>20</td>
<td>2.8 tons/ft x (P_s)</td>
</tr>
<tr>
<td>15</td>
<td>0.8 tons/ft x (P_s)</td>
</tr>
<tr>
<td>10</td>
<td>0.2 tons/ft x (P_s)</td>
</tr>
</tbody>
</table>

\(P_s\) = Pile Perimeter in feet

Factors of Safety. We recommend applying a factor of safety to the ultimate pile capacities provided above based on soil support properties. A minimum FS of 2 is recommended for typical non-seismic conditions. For seismic conditions we recommend a factor of safety of 1 in accordance with the ASCE SSDPW. For extreme moorage loading combinations such as flood and extreme wind conditions a FS of 1.5 is appropriate for dolphin piles.

Lateral Pile Capacity and Lateral Spread Loading

General. We understand the lateral response of the piles will be evaluated using the L-Pile program developed by Ensoft, Inc. For the purpose of evaluating lateral pile capacity and lateral spreading forces, we have delineated the subsurface profile into five zones, which were discretized based on the soil composition, factor of safety against liquefaction, residual shear strength, location within the slope, and other factors. The zones are presented graphically on Figure 5 and are described below.

Zone 1. Zone 1 includes the soils in the upper portion of the riverbank above the average river level (elevation +6 ft). These soils include the fill and unsaturated alluvial sand and silt. These soils are not considered liquefiable, but are subject to lateral spreading deformations during the CLE and DE hazard levels.
**Zone 2.** Zone 2 includes the soils below the average river level (elevation +6 ft) extending to the toe of the riverfront slope near the face of the proposed dock (elevation -35 ft). The soils in this zone consist of loose to medium dense sand with variable silt content or medium stiff to stiff, sandy silt, which are anticipated to be liquefiable during the CLE and DE hazard levels. The soils in this zone will be displaced toward the river by lateral spreading.

**Zone 3.** Zone 3 generally includes medium dense, clean sand to sand with a trace of silt and underlies Zone 2 in the inshore of the toe of the slope and is exposed on the post-dredge mudline toward the middle of the river channel. Zone 3 extends to elevation -60 ft, which is the extent of potentially liquefiable soil during the CLE and DE hazard level. The soils in this zone are assumed to be liquefiable, but will not be subject to significant lateral spreading.

**Zone 4.** Zone 4 includes the medium dense to dense sand below Zone 3. The soils in this zone are assumed to be non-liquefiable and will not be subjected to lateral spreading.

**Static L-Pile Parameters.** Our recommended static soil properties for use in the L-Pile analysis are provided in Table 5. The locations of the soil zones described above are shown on Figure 5.

<table>
<thead>
<tr>
<th>Soil Zone</th>
<th>L-Pile Soil Type</th>
<th>γ', pcf</th>
<th>K, pci</th>
<th>ϕ'</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone 1</td>
<td>Sand (Reese)</td>
<td>115</td>
<td>90</td>
<td>34°</td>
</tr>
<tr>
<td>Zone 2</td>
<td>Sand (Reese)</td>
<td>55</td>
<td>20</td>
<td>30°</td>
</tr>
<tr>
<td>Zone 3</td>
<td>Sand (Reese)</td>
<td>55</td>
<td>60</td>
<td>32°</td>
</tr>
<tr>
<td>Zone 4</td>
<td>Sand (Reese)</td>
<td>55</td>
<td>125</td>
<td>36°</td>
</tr>
</tbody>
</table>

Ground slope effects and pile batter can be accounted for in the lateral pile capacity evaluation by input of the ground surface inclination in the L-Pile program. Group effects can be modeled in L-Pile by applying an appropriate p-modifier. The p-modifier is a function of the center-to-center spacing, where D is the diameter of the pile. The recommended p-modifiers for group effects are provided in Table 6.

<table>
<thead>
<tr>
<th>Center-to-Center Pile Spacing</th>
<th>P-Modifiers for Rows 1, 2, and 3+</th>
</tr>
</thead>
<tbody>
<tr>
<td>3D</td>
<td>0.8, 0.4, 0.3</td>
</tr>
<tr>
<td>4D</td>
<td>0.9, 0.6, 0.5</td>
</tr>
<tr>
<td>5D</td>
<td>1.0, 0.85, 0.7</td>
</tr>
</tbody>
</table>

**Seismic L-Pile Parameters.** As described above, our seismic analysis indicates the soil in Zones 1 and 2 will be subject to lateral spreading deformations as a result of the CLE and DE hazard level earthquakes. Consequently, we anticipate the soil surrounding the portion of the piles located in these zones will not provide resistance to lateral loads. The soil in Zone 3 is potentially liquefiable as a result of the CLE and DE hazard level earthquake. The static L-Pile parameters provided in Table 5 should be modified to
account for the effects of lateral spreading and the reduced stiffness of the liquefied soils. Our analysis indicates that the soil in Zones 1 through 3 is not susceptible to liquefaction during the OLE hazard level and the soil in Zone 4 is not susceptible to liquefaction during any of the code-based hazard level. Recommended p-modifiers to account for the impacts of seismic loading during the CLE and DE hazard levels are provided in Table 7.

**TABLE 7: P-MODIFIERS SEISMIC L-PILE ANALYSIS**

<table>
<thead>
<tr>
<th>Soil Zone</th>
<th>Seismic P-Modifier</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone 1</td>
<td>Non-Liquefiable, 1.0</td>
</tr>
<tr>
<td>Zone 2</td>
<td>Non-Liquefiable, 1.0</td>
</tr>
<tr>
<td>Zone 3</td>
<td>Non-Liquefiable, 1.0</td>
</tr>
<tr>
<td>Zone 4</td>
<td>Non-Liquefiable, 1.0</td>
</tr>
</tbody>
</table>

For seismic evaluations, a group modifier of 1.0 should be used for liquefiable soil layers. The group modifiers provided in Table 6 should be used in seismic evaluations for all non-liquefiable soil layers.

**Design Estimates for Lateral Displacement Forces.** We recommend estimating lateral displacement forces on the piles using a force-based approach in accordance with the recommendations of Yokoyama et al. (1997). Based on the soil profile and bathymetry, we have assumed lateral spreading deformations will occur primarily in Zones 1 and 2, i.e., above elevation -35 ft. The estimated soil movement will result in two different pressures acting on the piles: 1) non-liquefied crustal soils moving against the piles and 2) saturated loose, liquefied sand or silt moving against the piles (Zone 2). In Zone 1, the lateral earth pressure exerted by the non-liquefied crust should be assumed to be equal to the passive pressure and should be applied over two pile diameters. In Zone 2, liquefied soil should be assumed to exert a lateral earth pressure on the piles equal to 30% of the total overburden pressure. The lateral pressure against piles in the saturated liquefied sand and silt material will act over one pile diameter. The recommended seismically induced lateral pressures for the piles are shown on Figure 5. Seismically induced lateral earth pressures will act in the direction toward the river channel.

**Pile Installation**

**Existing Dock.** Support for the existing dock is provided by vertical 18-in. octagonal, precast, prestressed, solid, reinforced concrete piles and battered steel pipe piles. GRI observed and documented installation of these piles. The concrete piles were driven using two different hammers. The contractor began the pile installation with a Delmag D30-32 diesel hammer with a maximum rated energy on the order of 70,000 ft-lbs. The contractor often had to restart the diesel hammer when driving the pile through the upper 10 to 20 ft of soft or loose soils. The contractor then changed to a Vulcan 020 air/steam hammer to help reduce driving times. The Vulcan 020 hammer has a rated energy of approximately 60,000 ft-lbs and operates at a constant 3-ft stroke. These piles were typically driven around 55 to 65 ft below the mudline. Observed terminal driving resistances with the Delmag D30-32 hammer averaged approximately 50 blows/ft, and terminal driving resistances with the Vulcan 020 hammer averaged about 30 blows/ft.

During the early portions of pile driving, the contractor had some difficulty maintaining pile locations within the specified tolerances. Observations indicated a significant portion of the pile movement was
occurring during the subsequent installation of adjacent piles. The pile movement was typically manifested by the displacement of the piles toward the river. When the pile installation sequence was modified so that the offshore piles were driven first and driving continued toward the shore, the displacements were significantly reduced.

**New Dock.** We anticipate an impact hammer will be required to drive the 24-in. octagonal concrete piles to the final tip elevation. In our opinion, jetting should not be permitted, unless it can be demonstrated that the piles cannot be installed to the design depths by continuous driving. We anticipate an impact hammer having a minimum rated energy of about 125,000 ft-lbs will be required to embed 24-in. octagonal concrete piles 100 ft below the ground surface/mudline. This minimum hammer energy should be considered preliminary and should be re-evaluated after the final pile loads and lengths are selected. In addition, the contractor should complete their own driveability analysis when selecting their driving hammer.

The contractor should submit a detailed description of the equipment and procedures that will be used to install the piles. Based on the difficulty experienced with maintaining the locations of the piles within the specified tolerances for the existing dock, the contractor should also submit a driving sequence for review. The contractor’s submittal should also include a wave equation analysis (WEAP) to evaluate driving stresses in the piles, which is particularly important for prestressed concrete piles. In addition to the analysis of stresses in the pile during driving, the WEAP can be used to estimate the terminal driving criteria of the pile. Figure 4 should be used by the contractor to estimate the ultimate pile capacity, skin friction distribution, and shaft capacity for use in their analysis. The analysis should be completed in accordance with Section 6-05.3(9)A of the WSDOT Standard Specifications.

We suggest an indicator pile-driving program be conducted as the initial step in the installation of the production piles or, preferably, prior to ordering production piles. The program would consist of installing several piles at regular intervals of distance along the alignment of the dock and the trestle. The indicator piles would be instrumented with Pile Driving Analyzer (PDA) equipment, which can be used to further evaluate driving stresses and pile capacities and to refine (if necessary) the terminal driving criteria determined using WEAP. After the indicator pile program is completed, we recommend a minimum of 2% of the production piles be instrumented with PDA equipment. All indicator and production pile driving operations should be observed by the geotechnical engineer, who will maintain a continuous record of the driving resistance (blows/ft) of each pile.

**Dredging Considerations**
We understand preliminary dredging design is being evaluated by others and may include dredge cuts in the range of 8 to 13 ft deep. The dredging plans should be evaluated for impact on riverbank slope stability, which is not in our scope of work. However, based on our experience along the Columbia River, we recommend that dredging proceed in increments from the top of the cut to the base of the slope, and box cut methods at the base of the slope should not be used.

**Design Review and Construction Services**
We welcome the opportunity to review and discuss construction plans and specifications for this project as they are being developed. In addition, GRI should be retained to review all geotechnical-related portions of the plans and specifications to evaluate whether they are in conformance with the recommendations.
provided in our report. Additionally, to observe compliance with the intent of our recommendations, design concepts, and plans and specifications, we are of the opinion that all construction operations dealing with earthwork and pile foundations should be observed by a GRI representative. Our construction-phase services will allow for timely design changes if site conditions are encountered that are different from those described in this report. If we do not have the opportunity to confirm our interpretations, assumptions, and analysis during construction, we cannot be responsible for the application of our recommendations to subsurface conditions that are different from those described in this report.

LIMITATIONS
This report has been prepared to aid the engineer in the design of this project. The scope is limited to the specific project and location described herein, and our description of the project represents our understanding of the significant aspects of the project relevant to the design and construction of the new dock. In the event that any changes in the design and location of the improvements as outlined in this report are planned, we should be given the opportunity to review the changes and to modify or reaffirm the conclusions and recommendations of this report in writing.

The conclusions and recommendations submitted in this report are based on the data obtained from the borings made at the locations indicated on Figure 2 and from other sources of information discussed in this report. In the performance of subsurface investigations, specific information is obtained at specific locations at specific times. However, it is acknowledged that variations in soil conditions may exist between boring locations. This report does not reflect any variations that may occur between these explorations. The nature and extent of variation may not become evident until construction. If, during construction, subsurface conditions different from those encountered in the explorations are observed or encountered, we should be advised at once so that we can observe and review these conditions and reconsider our recommendations where necessary.

Submitted for GRI,

Matthew S. Shanahan, PE
Associate

Scott M. Schlechter, PE
Principal

Brian A. Bennetts, PE
Project Engineer

References


LINE LOAD PARALLEL TO WALL

\[ \sigma_h = \frac{Q_i}{H} \left( \frac{0.2n}{0.16 + n} \right) \]

For \( m \leq 0.4 \):

\[ \sigma_h = \frac{0.128m^2n}{(m^2 + n)} \]

For \( m > 0.4 \):

\[ \sigma_h = \frac{2m}{n} \left( \beta \sin \beta \cos 2\alpha \right) \]

\( \beta \) in radians

STRIP LOAD PARALLEL TO WALL

\[ \sigma_h = \frac{2m}{n} \left( \beta \sin \beta \cos 2\alpha \right) \]

\( \beta \) in radians

POINT LOAD, \( Q_p \)

\[ \sigma_h = \frac{Q_p}{H} \left( \frac{0.28n^2}{0.16 + n} \right) \]

For \( m \leq 0.4 \):

\[ \sigma_h = \frac{1.77m^2n^2}{(m^2 + n)} \]

DISTRIBUTION OF HORIZONTAL PRESSURES

[Diagram of horizontal pressures]

VERTICAL POINT LOAD

NOTES:

1. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.3 FOR BACKFILL MATERIALS.

2. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.

GRI
BERGERABAM
KALAMA NORTH PORT METHANOL PLANT DOCK

SURCHARGE-INDUCED LATERAL PRESSURE

FEB. 2015 JOB NO. W1153 FIG. 3
STATIC AND SEISMIC OLE HAZARD LEVEL EVALUATION

ULTIMATE SKIN FRICTION CAPACITY (TONS):
\[ Q_s = q_s \times P_s \times L_s \]

ULTIMATE END BEARING CAPACITY (TONS):
\[ Q_t = q_t \times A_p \]

ULTIMATE COMPRESSIVE PILE CAPACITY (TONS):
\[ Q_{ult} = Q_s + Q_t \]

ULTIMATE UPLIFT CAPACITY (TONS):
\[ Q_{up} = 0.7 \times Q_s = 0.7 \times q_s \times P_s \times L_s \]

SEISMIC CLE AND DE HAZARD LEVEL EVALUATION

LIQUEFACTION INDUCED CAPACITY REDUCTION (TONS):
\[ Q_{s red} = q_{ss} \times P_s \times L_{ss} \]

ULTIMATE SEISMIC COMPRESSIVE PILE CAPACITY (TONS):
\[ Q_{ult seismic} = Q_{ult} - Q_{s red} \]

ULTIMATE SEISMIC UPLIFT PILE CAPACITY (TONS):
\[ Q_{up seismic} = Q_{up} - Q_{s red} + 0.80 \times D_D \]

IN WHICH:
- \( q_s \) = AVERAGE UNIT SKIN FRICTION MEASURED AT THE PILE EMBEDDED DEPTH (TSF)
- \( q_{ss} \) = AVERAGE UNIT SKIN FRICTION MEASURED AT EMBEDDED DEPTH CORRESPONDING TO ELEVATION -60 FT (TSF)
- \( q_T \) = UNIT TIP BEARING (TSF) – 75 TSF BELOW 30 FT OF EMBEDMENT OR ELEVATION -70 FT, WHICHER IS DEEPEST
- \( A_T \) = PILE TIP AREA (FT²)
- \( P_s \) = PILE PERIMETER (FT)
- \( L_s \) = LENGTH OF PILE BELOW MUDLINE (FT)
- \( L_{ss} \) = EMBEDDED LENGTH OF PILE ABOVE ELEVATION -60 FT (FT)
- \( D_D \) = DOWNDRAG LOAD; SEE TABLE 3 (TONS)

NOTES: 1) PILES SHOULD BE DRIVEN MINIMUM OF 30 FT BELOW MUDLINE OR TO ELEVATION -60 FT, WHICHER IS DEEPEST
2) SEE TEXT FOR SELECTION OF APPROPRIATE SAFETY FACTORS
FIELD EXPLORATIONS

Subsurface materials and conditions at the site of the proposed improvements were investigated by GRI October 29 and November 5, 2002, with four borings, designated B-1 through B-4. The approximate locations of the borings are shown on the Site Plan, Figure 2. The borings were completed using a truck-mounted CME-75 mud-rotary drill rig provided and operated by Geotech Explorations, Inc. of Tualatin, Oregon. Borings B-1 and B-2 were drilled on shore, and borings B-3 and B-4 were drilled over water from a barge owned and operated by Mark Marine Service of Camas, Washington. All drilling and sampling operations were observed by a geotechnical engineer from GRI, who maintained a detailed log of the materials and conditions disclosed during the course of the work.

Borings B-1 and B-2 were advanced to a total depth of 101.5 ft below the ground surface, and borings B-3 and B-4 were advanced to a depth of 101.5 ft below the mudline. Disturbed samples were typically obtained from the borings at 5-ft intervals of depth using a standard split-spoon sampler. At the time of sampling, the Standard Penetration Test was conducted. This test consists of driving a standard split-spoon sampler into the soil a distance of 18 in. using a 140-lb hammer dropped 30 in. The number of blows required to drive the sampler the last 12 in. is known as the standard penetration resistance, or N-value. The N-values provide a measure of the relative density of granular soils, such as sand, and the relative consistency, or stiffness, of cohesive soils, such as silt. In addition to the split-spoon samples, relatively undisturbed 2.85-in.-I.D. Shelby tube samples were also taken at selected depths using an Osterberg piston sampler. The split-spoon samples obtained were carefully examined in the field and representative portions were saved in airtight jars. All samples were returned to our laboratory for further examination and physical testing.

Logs of the borings are shown on Figures 1A through 4A. Each log presents a descriptive summary of the various types of materials encountered in the boring and notes the depth at which the materials and/or characteristics of the materials change. To the right of the descriptive summary, the numbers and types of samples taken during the drilling operation are indicated. Farther to the right, N-values are shown graphically, along with natural moisture content values. The terms and symbols used to describe the soils are defined in Table 1A and the attached legend.

LABORATORY TESTING

General

All samples obtained from the borings were returned to our laboratory where the physical characteristics of the samples were noted and the field classifications were modified where necessary. The laboratory testing program included determinations of natural moisture content, unit weight, fines content (percentage passing than a No. 200 sieve), and Torvane shear strength. The following paragraphs describe the testing program in more detail.

Natural Moisture Content

Natural moisture content determinations were in conformance with ASTM D 2216. The results are summarized on the Boring Logs, Figures 1A through 4A.
Unit Weight

The dry unit weight of undisturbed soil samples was determined in the laboratory in substantial conformance with ASTM D 2937. The test results are summarized below.

<table>
<thead>
<tr>
<th>Boring</th>
<th>Sample</th>
<th>Depth, ft</th>
<th>Natural Moisture Content, %</th>
<th>Dry Unit Weight, pcf</th>
<th>Soil Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>S-4</td>
<td>21.5</td>
<td>44</td>
<td>76</td>
<td>SAND</td>
</tr>
<tr>
<td>S-8</td>
<td></td>
<td>34.0</td>
<td>32</td>
<td>90</td>
<td>SAND</td>
</tr>
<tr>
<td>B-2</td>
<td>S-5</td>
<td>25.5</td>
<td>46</td>
<td>74</td>
<td>SILT</td>
</tr>
<tr>
<td>S-5</td>
<td></td>
<td>26.1</td>
<td>33</td>
<td>87</td>
<td>SAND</td>
</tr>
<tr>
<td>S-9</td>
<td></td>
<td>39.0</td>
<td>43</td>
<td>78</td>
<td>SILT</td>
</tr>
</tbody>
</table>

Grain Size

Washed sieve analyses were performed for selected soil samples to determine the percentage of material passing the No. 200 sieve. These tests were performed to assist in material classification and for use in liquefaction studies. The test is performed by taking a sample of known dry weight and washing it over a No. 200 sieve. The material retained on the sieve is oven dried and weighed. The percentage of material that passed the No. 200 sieve is then calculated. Test results are summarized below.

<table>
<thead>
<tr>
<th>Boring</th>
<th>Sample</th>
<th>Depth, ft</th>
<th>% Passing the No. 200 Sieve</th>
<th>Soil Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1</td>
<td>S-5</td>
<td>22.0</td>
<td>53</td>
<td>Sandy SILT</td>
</tr>
<tr>
<td>S-6</td>
<td></td>
<td>25.0</td>
<td>38</td>
<td>Silty SAND</td>
</tr>
<tr>
<td>S-7</td>
<td></td>
<td>30.0</td>
<td>27</td>
<td>SAND; some silt</td>
</tr>
<tr>
<td>S-9</td>
<td></td>
<td>35.0</td>
<td>69</td>
<td>Sandy SILT</td>
</tr>
<tr>
<td>S-12</td>
<td></td>
<td>50.0</td>
<td>6</td>
<td>SAND; trace silt</td>
</tr>
<tr>
<td>B-2</td>
<td>S-2</td>
<td>10.0</td>
<td>5</td>
<td>SAND; trace silt</td>
</tr>
<tr>
<td>S-3</td>
<td></td>
<td>15.0</td>
<td>73</td>
<td>SILT; some sand</td>
</tr>
<tr>
<td>S-4</td>
<td></td>
<td>20.0</td>
<td>27</td>
<td>SAND; some silt</td>
</tr>
<tr>
<td>S-7</td>
<td></td>
<td>30.0</td>
<td>46</td>
<td>Silty SAND</td>
</tr>
<tr>
<td>S-11</td>
<td></td>
<td>45.0</td>
<td>39</td>
<td>Silty SAND</td>
</tr>
</tbody>
</table>

Torvane Shear Strength

The approximate undrained shear strength of relatively undisturbed, fine-grained, soil samples was determined using a Torvane shear device. The Torvane is a hand-held apparatus with vanes that are inserted into the soil. The torque required to fail the soil in shear around the vanes is measured using a calibrated spring. The results of the Torvane shear strength determinations are shown on the Boring Logs, Figures 1A through 4A.
### Table 1A: GUIDELINES FOR CLASSIFICATION OF SOIL

#### RELATIVE DENSITY FOR GRANULAR SOIL

<table>
<thead>
<tr>
<th>Relative Density</th>
<th>Standard Penetration Resistance (N-values) blows per foot</th>
</tr>
</thead>
<tbody>
<tr>
<td>very loose</td>
<td>0 - 4</td>
</tr>
<tr>
<td>loose</td>
<td>4 - 10</td>
</tr>
<tr>
<td>medium dense</td>
<td>10 - 30</td>
</tr>
<tr>
<td>dense</td>
<td>30 - 50</td>
</tr>
<tr>
<td>very dense</td>
<td>over 50</td>
</tr>
</tbody>
</table>

#### CONSISTENCY FOR FINE-GRAINED (COHESIVE) SOIL

<table>
<thead>
<tr>
<th>Consistency</th>
<th>Standard Penetration Resistance (N-value) blows per foot</th>
<th>Torvane or Undrained Shear Strength, tsf</th>
</tr>
</thead>
<tbody>
<tr>
<td>very soft</td>
<td>0 - 2</td>
<td>less than 0.125</td>
</tr>
<tr>
<td>soft</td>
<td>2 - 4</td>
<td>0.125 - 0.25</td>
</tr>
<tr>
<td>medium stiff</td>
<td>4 - 8</td>
<td>0.25 - 0.50</td>
</tr>
<tr>
<td>stiff</td>
<td>8 - 15</td>
<td>0.50 - 1.0</td>
</tr>
<tr>
<td>very stiff</td>
<td>15 - 30</td>
<td>1.0 - 2.0</td>
</tr>
<tr>
<td>hard</td>
<td>over 30</td>
<td>over 2.0</td>
</tr>
</tbody>
</table>

#### Grain-Size Classification

<table>
<thead>
<tr>
<th>Boulders</th>
<th>Modifier for Subclassification</th>
<th>Percentage of Other Material In Total Sample</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt; 12 in.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cobbles</td>
<td>Adjective</td>
<td></td>
</tr>
<tr>
<td>3 - 12 in.</td>
<td>clean</td>
<td>0 - 2</td>
</tr>
<tr>
<td>Gravel</td>
<td>trace</td>
<td>2 - 10</td>
</tr>
<tr>
<td>1/4 - 3/4 in. (fine)</td>
<td>some</td>
<td>10 - 30</td>
</tr>
<tr>
<td>3/4 - 3 in. (coarse)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sand</td>
<td>sandy, silty, clayey, etc.</td>
<td>30 - 50</td>
</tr>
<tr>
<td>No. 200 - No. 40 sieve (fine)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. 40 - No. 10 sieve (medium)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. 10 - No. 4 sieve (coarse)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Silt/Clay</td>
<td>- pass No. 200 sieve</td>
<td></td>
</tr>
</tbody>
</table>
### Classification of Material

**Depth, Ft**

<table>
<thead>
<tr>
<th>Depth, Ft</th>
<th>Material Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Surface elevation 22 ft (+)</td>
</tr>
<tr>
<td>5</td>
<td>FILL: Very loose, gray SAND; fine to medium grained, scattered fine to medium gravel (white pumice), 3-in.-thick heavily rooted zone at the ground surface</td>
</tr>
<tr>
<td>10</td>
<td>gravel layer between 8.5 and 9.5 ft</td>
</tr>
<tr>
<td>15</td>
<td>medium dense below 10 ft</td>
</tr>
<tr>
<td>17.5</td>
<td>Interbedded layers of loose, gray SAND, fine grained, trace silt to silty, and soft to medium stiff SILT; some sand to sandy, scattered organics</td>
</tr>
<tr>
<td>20</td>
<td>wood debris between 28.5 and 29 ft</td>
</tr>
</tbody>
</table>

**2-IN.-OD Split-Spoon Sampler**

**3-IN.-OD Thin-Walled Sampler**

**Grab Sample of Drill Cuttings**

**SLOTTED PVC Pipe**

**Water Level (date)**

### Groundwater Samples

<table>
<thead>
<tr>
<th>Depth, Ft</th>
<th>Samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>50</td>
<td>50</td>
</tr>
<tr>
<td>100</td>
<td>100</td>
</tr>
</tbody>
</table>

**Boring B-1**

**Surface Elevation 22 ft (+)**

**FEB. 2015 JOB. NO. W1153 FIG. 1A**
Interbedded layers of loose, gray SAND, fine grained, trace silt to silty, and soft to medium stiff SILT; some sand to sandy, scattered organics

Medium dense, gray SAND; fine grained, clean to trace silt, contains thin interbedded layers of coarse-grained sand and fine gravel (white pumice)
CLASSIFICATION OF MATERIAL

Surface Elevation 22 ft (+)

Medium dense, gray SAND; fine to medium grained, trace silt, scattered thin interbedded layers of coarse sand and fine gravel (white pumice)

Boring B-1 (cont.)

FEB. 2015                        JOB. NO. W1153               FIG. 1A
FILL: Medium dense, brown SAND; fine to medium grained, trace silt, 2-in.-thick moderately rooted zone at the ground surface

---

---fine grained below 10 ft

---coarse grained between 13 and 13.5 ft

Interbedded layers of loose, gray SAND, fine grained, trace silt to silty, and soft to medium stiff SILT; some sand to sandy, scattered organics
CLASSIFICATION OF MATERIAL

SURFACE ELEVATION 21 ft (+)

Interbedded layers of loose, gray SAND, fine grained, trace silt to silty, and soft to medium stiff SILT; some sand to sandy, scattered organics

Medium dense, gray SAND; fine grained, trace silt, contains thin interbedded layers of coarse-grained sand and fine gravel (white pumice)

STD PENETRATION RESISTANCE (140-LB WEIGHT, 30-IN. DROP)

BLOWS PER FOOT

MOISTURE CONTENT, %

G  R    I

0 0.5 1.0

(TONS PER FT²)

FEB. 2015                        JOB. NO. W1153               FIG. 2A

BORING B-2 (cont.)
<table>
<thead>
<tr>
<th>Depth, ft</th>
<th>Groundwater Samples</th>
<th>Std Penetration Resistance (140-lb weight, 30-in. drop)</th>
</tr>
</thead>
<tbody>
<tr>
<td>101.5</td>
<td>S-18</td>
<td>Bows per foot</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Moisture content, %</td>
</tr>
<tr>
<td>95</td>
<td>S-19</td>
<td></td>
</tr>
<tr>
<td>90</td>
<td>S-20</td>
<td></td>
</tr>
<tr>
<td>85</td>
<td>S-21</td>
<td></td>
</tr>
<tr>
<td>80</td>
<td>S-22</td>
<td></td>
</tr>
</tbody>
</table>

**CLASSIFICATION OF MATERIAL**

- 2-in.-OD Split-Spoon Sampler
- 3-in.-OD Thin-Walled Sampler
- Grab Sample of Drill Cuttings
- NX Core Run
- Slotted PVC Pipe
- Water Level (date)

**SURFACE ELEVATION** 21 ft (+)

Medium dense, gray SAND; fine to medium grained, trace silt, contains thin interbedded layers of coarse-grained sand and fine gravel (white pumice)

- 6-in.-thick layer of gravel at 88.5 ft
- 1-ft-thick layer of pumice at 100.5 ft

**Liquid Limit**

**Plastic Limit**

**Moisture Content**

**Liquid Limit**

**Plastic Limit**

**Moisture Content, %**

**Torvane Shear Strength, TSF**

**Undrained Shear Strength, TSF**

**No Recovery**

---

**Boring B-2 (cont.)**

FEB. 2015  | JOB. NO. W1153  | FIG. 2A
CLASSIFICATION OF MATERIAL

MUDLINE ELEVATION -43 ft (±)

Very loose, gray SAND; fine grained, trace to some silt

- wood debris between 5 and 7.5 ft
- loose, fine to medium grained, clean to trace silt below 7.5 ft
- wood debris between 12.5 and 13 ft
- medium dense below 17 ft
- thin interbedded layers of coarse-grained sand and fine gravel (white pumice) below 20 ft

BORING B-3

FEB. 2015 JOB. NO. W1153

GRI

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)

- BLOWS PER FOOT
- MOISTURE CONTENT, %

B-3

G

SLOTTED PVC PIPE

Water Level (date)
Medium dense, gray SAND; fine to medium grained, clean to trace silt, contains thin interbedded layers of coarse-grained sand and fine gravel (white pumice)

scattered organics (small wood debris) below 46 ft

medium dense to dense below 49 ft

fine grained below 53 ft

MUDLINE ELEVATION -43 ft

GBORING B-3 (cont.)

FEB. 2015                        JOB. NO. W1153               FIG. 3A
CLASSIFICATION OF MATERIAL

Medium dense to dense, gray SAND; fine to medium grained, clean to trace silt, contains thin interbedded layers of coarse-grained sand and fine gravel (white pumice)

MUDLINE ELEVATION -43 ft (±)

2-IN.-OD SPLIT-SPOON SAMPLER
3-IN.-OD THIN-WALLED SAMPLER
GRAB SAMPLE OF DRILL CUTTINGS

S-16
S-17
S-18
S-19
S-20

(11/1/2002)

Boring B-3 (cont.)

JOB. NO. W1153

FEB. 2015

FIG. 3A
CLASSIFICATION OF MATERIAL

MUDLINE ELEVATION -43 ft (±)

- Very loose, gray SAND; fine grained, trace silt

- Loose, fine to medium grained, clean to trace silt below 7.5 ft

- Medium dense below 15 ft

- Gravel between 18 and 19 ft

- Thin interbedded layers of coarse-grained sand and fine gravel (white pumice) below 20 ft

- 6-in.-thick layer of gravel at 32 ft

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)

- BLOWS PER FOOT
- MOISTURE CONTENT, %

G  R    I

MUDLINE ELEVATION -43 ft (±)

- Very loose, gray SAND; fine grained, trace silt

- Loose, fine to medium grained, clean to trace silt below 7.5 ft

- Medium dense below 15 ft

- Gravel between 18 and 19 ft

- Thin interbedded layers of coarse-grained sand and fine gravel (white pumice) below 20 ft

- 6-in.-thick layer of gravel at 32 ft

BORING B-4

FEB. 2015  JOB. NO. W1153  FIG. 4A
Medium dense to dense, gray SAND; clean to trace silt, contains thin interbedded layers of coarse-grained sand and fine gravel (white pumice), scattered small wood debris

---

MUDLINE ELEVATION -43 ft (±)

---

CLASSIFICATION OF MATERIAL

- 2-IN.-OD SPLIT-SPOON SAMPLER
- 3-IN.-OD THIN-WALLED SAMPLER
- GRAB SAMPLE OF DRILL CUTTINGS
- SLOTTED PVC PIPE
- NX CORE RUN

GROUNDWATER

- BOWS PER FOOT
- MOISTURE CONTENT, %

STD PENETRATION RESISTANCE
(140-LB WEIGHT, 30-IN. DROP)

- MOISTURE CONTENT, %

GRAPHIC LOG

GROUNDWATER

- WATER LEVEL (DATE)

BORING B-4 (cont.)

FEB. 2015  JOG. NO. W1153  FIG. 4A
Medium dense to dense, gray SAND; fine grained, clean to trace silt

---2-in.-thick layer of volcanic ash at 80.5 ft
APPENDIX B
SITE-SPECIFIC SEISMIC HAZARD STUDY

General
GRI has completed a site-specific seismic hazard study for the proposed dock expansion at the North Port terminal at the Port of Kalama, Washington. The purpose of the study was to evaluate the potential seismic hazards associated with regional and local seismicity. The site-specific hazard study is intended to meet the requirements of the American Society of Civil Engineers (ASCE) Seismic Design of Pile-Supported Piers and Wharves in compliance with the requirements of ASCE 7-05 Chapter 21. Our work was based on the potential for regional and local seismic activity, as described in the existing scientific literature, and on the subsurface conditions at the site, as disclosed by the geotechnical explorations completed for the project. Specifically, our work included the following tasks:

1) A detailed review of available literature, including published papers, maps, open-file reports, seismic histories and catalogs, 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.

2) Compilation, examination, and evaluation of existing subsurface data gathered at and in the vicinity of the site, including classification and laboratory analyses of soil samples. This information was used to prepare a generalized subsurface profile for the proposed dock site.

3) Identification of potential seismic sources appropriate for the site and characterization of those sources in terms of magnitude, distance, and acceleration response spectra.

4) Office studies, based on the generalized subsurface profile and the controlling seismic sources, resulting in conclusions and recommendations concerning:
   a) specific seismic events and characteristic earthquakes that might have a significant effect on the site;
   b) the potential for seismic energy amplification at the site; and
   c) site-specific acceleration response spectra for design of the proposed dock.

This appendix describes the work accomplished and summarizes our conclusions and recommendations.

Geologic Setting
General. On a regional scale, the site lies approximately 74 km inland from the down-dip edge of the seismogenic extent of the Cascadia Subduction Zone (CSZ), an active convergent plate boundary along which remnants of the Farallon Plate (the Gorda, Juan de Fuca, and Explorer plates) are being subducted beneath the western edge of the North American continent. The subduction zone is a broad, eastward-dipping zone of contact between the upper portion of the subducting slabs and the over-riding North American Plate as shown on Figure 1B.
On a local scale, the site lies within the Willamette-Puget Sound lowland trough of the Cascadia convergent tectonic system (Blakely et al., 2000). The lowland areas consist of broad north-south-trending basins in the underlying geologic structure between the Coast Range to the west and the Cascade Mountains to the east. The lowland trough is characterized by alluvial plains with areas of buttes and terraces. Within the basin, some faults have been mapped on the basis of stratigraphic offsets and geophysical evidence, but accurate information regarding the precise location and extent of these faults is lacking, due largely to the scale at which geologic mapping in the area has been conducted and the presence of thick, relatively young basin-filling sediments that obscure underlying structural features. The distribution of the crustal faults relative to the site is shown on the Regional Geologic Map and Local Fault Map, Figures 2B and 3B, respectively.

Because of the proximity of the site to the CSZ and its location within the Willamette Valley-Puget Sound Lowland, three seismic sources contribute to the potential for damaging earthquake motions at the site. Two of these sources are associated with tectonic activity related to the Cascadia Subduction Zone; the third is associated with movement on relatively shallow faults.

Subsurface Conditions. The site is mantled with hydraulically placed dredged sand fill that is underlain by a thick sequence of unconsolidated, fine- to coarse-grained Quaternary floodplain deposits. Locally, these alluvial deposits typically consist of silt and clayey silt with lenticular interbeds of sand, sandy silt, and silty sand of limited lateral extent within the Columbia River Channel. Floodplain deposits are underlain by river channel deposits that typically consist of fine- to medium-grained sand with lenses of gravel. Geologic mapping indicates these river channel deposits are underlain by thick accumulations of mainly subaerial basalt lava flows of the Goble Volcanics Formation. The boundary between the overlying sedimentary materials and the underlying basalt is unconformable, indicating that a considerable period of time elapsed between the last of the basalt flows and the deposition of the overlying sedimentary materials. Locally, basalt outcrops of Goble Volcanics are visible on both sides of the river in the Kalama area, one of which can be observed just east of the site between the BNSF rail line and Interstate 5.

A subsurface profile model of the project site was developed based on subsurface information developed by GRI (GRI, 1993; 2003).

Seismicity

General. The geologic and seismologic information available for identifying the potential seismicity at the site is incomplete, and large uncertainties are associated with estimates of the probable magnitude, location, and frequency of occurrence of earthquakes that might affect the site. The available information indicates the potential seismic sources that may affect the site can be grouped into three independent categories: subduction zone events related to sudden slip between the upper surface of the Juan de Fuca plate and the lower surface of the North American plate, subcrustal (intraslab) events related to deformation and volume changes within the deeper portion of the subducted Juan de Fuca plate, and local crustal events associated with movement on shallow, local faults. Based on our review of currently available information, we have developed parameters for each of these potential seismic sources. The seismic sources are characterized by three important parameters: magnitude, distance to the subject site, and the peak horizontal bedrock accelerations produced by the controlling earthquake on the seismic source. The size of an earthquake is commonly defined by its moment magnitude Mw. Distance is measured using the closest horizontal distance to the surface projection of the rupture plane or the closest
distance to the rupture plane, in kilometers. Peak horizontal bedrock accelerations are expressed in units of gravity (1 g = 32.2 ft/sec^2 = 981 cm/sec^2).

Subduction Zone Event. Written Japanese tsunami records suggest that a great CSZ earthquake occurred in January 1700. Geological studies suggest that great megathrust earthquakes have occurred repeatedly in the past 7,000 years (Atwater et al., 1995; Clague, 1997; Goldfinger, 2003; and Kelsey et al., 2005), and geodetic studies (Hyndman and Wang, 1995; Savage et al., 2000) indicate rate of strain accumulation consistent with the assumption that the CSZ is locked beneath offshore northern California, Oregon, Washington, and southern British Columbia (Fluck et al., 1997; Wang et al., 2001). Numerous geological and geophysical studies suggest the CSZ may be segmented (Hughes and Carr, 1980; Weaver and Michaelson, 1985; Guffanti and Weaver, 1988; Goldfinger, 1994; Kelsey and Bockheim, 1994; Mitchell et al., 1994; Personius, 1995; Nelson and Personius, 1996; Witter, 1999), but the most recent studies suggest that for the last great earthquake in 1700, most of the subduction zone ruptured in a single Mw 9 earthquake (Satake et al., 1996; Atwater and Hemphill-Haley, 1997; Clague et al., 2000). Published estimates of the probable maximum size of subduction zone events range from moment magnitude Mw 8.3 to >9.0. Numerous detailed studies of coastal subsidence, tsunamis, and turbidites yield a wide range of recurrence intervals, but the most complete records (>4,000 years) indicate average intervals of 350 to 600 years between great earthquakes on the CSZ (Adams, 1990; Atwater and Hemphill-Haley, 1997; Witter, 1999; Clague et al., 2000; Kelsey et al., 2002; Kelsey et al., 2005; Witter et al., 2003). Tsunami inundation in buried marshes along the Washington and Oregon coast and stratigraphic evidence from the Cascadia margin support these recurrence intervals (Kelsey et al., 2005; Goldfinger, 2003).

The USGS probabilistic analysis assumes four potential locations for the eastern edge of the earthquake rupture zone for the CSZ, as shown in Figure 4B. The 2008 USGS mapping effort indicates two rupture scenarios are assumed to represent these megathrust events: 1) Mw 9.0±0.2 events that rupture the entire CSZ every 500 years and 2) Mw 8.0 to 8.7 events with rupture zones that occur on segments of the CSZ and occur over the entire length of the CSZ during a period of about 500 years (Petersen et al., 2008). The assumed distribution of earthquake magnitudes is shown on Figure 5B. This distribution assumes the larger Mw 9.0 earthquakes likely occur more often than the smaller segmented ruptures. Therefore, for our deterministic analysis, we have chosen to represent the subduction zone event by a design earthquake of Mw 9.0 at a focal depth of 15 km and a rupture distance of 74 km. This corresponds to a sudden rupture of the entire length of the Juan de Fuca-North American plate interface with an assumed rupture zone along the coastline due west of Vancouver.

Subcrustal Event. There is no historic earthquake record of subcrustal, intraslab earthquakes in Southwest Washington. Although both the Puget Sound and Northern California regions have experienced many of these earthquakes in historic times, Wong (2005) hypothesizes that due to subduction zone geometry, geophysical conditions and local geology, Southwest Washington/Oregon may not be subject to intraslab earthquakes. In the Puget Sound area, these moderate to large earthquakes are deep (40 to 60 km) and over 200 km from the deformation front of the subduction zone. Offshore, along the Northern California coast, the earthquakes are shallower (up to 40 km) and located along the deformation front. Estimates of the probable magnitude, distance, and frequency of subcrustal events in Southwest Washington are generally based on comparisons of the CSZ with active convergent plate margins in other parts of the world and on the historical seismic record for the region surrounding Puget Sound, where significant events known to have occurred within the subducting Juan de Fuca plate have been recorded. Published
estimates of the probable maximum size of these events range from moment magnitude $M_w$ 7.0 to 7.5. The 1949, 1965, and 2001 documented subcrustal earthquakes in the Puget Sound area correspond to $M_w$ 7.1, 6.5, and 6.8, respectively. Published information regarding the location and geometry of the subducting zone indicates that a focal depth of 50 km is probable (Weaver and Shedlock, 1989). We have chosen to represent the subcrustal event by a characteristic earthquake of moment magnitude $M_w$ 7.0 at a focal depth of 50 km and a rupture distance of 50 km.

**Local Crustal Event.** Sudden crustal movements along relatively shallow local faults in the southwest Washington area, although rare, have been responsible for local crustal earthquakes. The precise relationship between specific earthquakes and individual faults is not well understood, since few of the faults in the area are expressed at the ground surface, and the foci of the observed earthquakes have not been located with precision. The history of local seismic activity is commonly used as a basis for determining the size and frequency to be expected of local crustal events. Although the historical record of local earthquakes is relatively short (the earliest reported seismic event in the area occurred in 1920), it can serve as a guide for estimating the potential for seismic activity in the area.

Based on the most recent study by Wong and others (2000), the closest mapped fault is the Portland Hills Fault, with a mapped extent of about 60 km and a corresponding characteristic earthquake magnitude ranging from about $M_L$ = 6.5 to about $M_L$ = 7.1, depending somewhat on the geometry of the fault at depth and the degree to which the fault is segmented, neither of which is well understood. Because many faults in the Willamette Valley-Puget Lowlands area, where present, are concealed beneath the thick, widespread sedimentary deposits, it is difficult to assign a specific location to this other possible local crustal earthquakes.

**Other Seismic Hazards.** Based on the presence of loose sands and soft silts below the groundwater surface at the site, there is a high risk of liquefaction and lateral spreading during a design-level earthquake. More detailed discussions regarding liquefaction and lateral spreading are provided in the Seismic Considerations section of the report. Limited tsunami modeling of the Columbia River due to a Cascadia Subduction Zone earthquake has been completed (Yeh et al., 2012). Based on preliminary simulations of tsunami penetration into the Columbia River, we anticipate the risk of upland damage by tsunami is low due to the distance of the site from the coast. River fluctuations may result from a tsunami generated by a CSZ earthquake. Due to the proximity of the Columbia River, there is a risk of seiche. Unless occurring on a previously unmapped fault, it is our opinion the risk of ground rupture at the site is low.

**Probabilistic Seismic Hazard Analysis (PSHA)**

The probability of an earthquake of a specific magnitude occurring at a given location is commonly expressed by its return period, i.e., the average length of time between successive occurrences of an earthquake of that size or larger at that location. The return period of a design earthquake is calculated once a project design life and some measure of the acceptable risk that the design earthquake might occur or be exceeded are specified. These expected earthquake recurrences are expressed as a probability of exceedance during a given time period or design life. The USGS provides updated probabilistic seismic hazard maps every 6 years for various probabilities of exceedance levels across the United States. The results of a PSHA for a given hazard level are commonly called a Uniform Hazard Spectrum (UHS) because all spectral ordinates have a uniform probability of exceedance in a given period of time. The ASCE standard *Seismic Design of Pile-Supported Piers and Wharves* defines three seismic hazard levels: the
Operating Level Earthquake (OLE), the Contingency Level Earthquake (CLE), and the Design Earthquake (DE). The OLE is defined by 50% probability of exceedance in 50 years, which corresponds to an earthquake with an expected recurrence interval of 72 years, and represents a performance level with minimal structural damage. The CLE is defined by 10% probability of exceedance in 50 years, which corresponds to an earthquake with an expected recurrence interval of 475 years, and represents a performance level of controlled and repairable structural damage. The DE is defined per ASCE 7-05, which develops the response spectrum based on ground motions associated with the Maximum Considered Earthquake (MCE). The MCE is represented by a probabilistic earthquake with a 2% probability of exceedance in 50 years (return period of 2,475 years), except where subject to deterministic limitations (Leyendecker and Franke, 2000). The Design Earthquake (DE) response spectrum is obtained by taking two-thirds of the MCE level ground motions.

For the proposed dock expansion site, located at the approximate latitude and longitude coordinates of 46.05°N and 122.88°W, the spectral acceleration values corresponding to return periods of 72, 475 and 2,475 years were obtained for Site Class B (rock) from the 2008 USGS hazard curves and uniform hazard maps. These spectral accelerations for the three hazard levels (return periods) are shown on Figure 6B, and summarized in Table 1B.

<table>
<thead>
<tr>
<th>Period, seconds</th>
<th>72 Years</th>
<th>475 Years</th>
<th>2,475 Years</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.05</td>
<td>0.20</td>
<td>0.42</td>
</tr>
<tr>
<td>0.1</td>
<td>0.10</td>
<td>0.38</td>
<td>0.87</td>
</tr>
<tr>
<td>0.2</td>
<td>0.12</td>
<td>0.44</td>
<td>0.96</td>
</tr>
<tr>
<td>0.3</td>
<td>0.10</td>
<td>0.39</td>
<td>0.84</td>
</tr>
<tr>
<td>0.5</td>
<td>0.08</td>
<td>0.30</td>
<td>0.67</td>
</tr>
<tr>
<td>1</td>
<td>0.04</td>
<td>0.16</td>
<td>0.39</td>
</tr>
<tr>
<td>2</td>
<td>0.01</td>
<td>0.08</td>
<td>0.20</td>
</tr>
<tr>
<td>3</td>
<td>0.01</td>
<td>0.04</td>
<td>0.11</td>
</tr>
<tr>
<td>4</td>
<td>0.00</td>
<td>0.02</td>
<td>0.07</td>
</tr>
<tr>
<td>5</td>
<td>0.00</td>
<td>0.02</td>
<td>0.05</td>
</tr>
</tbody>
</table>

Deaggregation of the 2008 USGS data suggests the Cascadia Subduction Zone, subcrustal events, and local crustal faults all contribute to the seismic hazard at the site. The 2008 USGS deaggregation indicate the seismic hazard for this site is dominated by the CSZ earthquake source for the 2,475-year hazard level and by the local crustal and subcrustal sources for the 72-year hazard level. The deaggregation for the 475-year hazard level indicates local crustal, subcrustal, and CSZ sources all contribute significantly to the site seismicity.

**Deterministic Seismic Hazard Analysis (DSHA)**

A deterministic seismic hazard analysis (DSHA) was performed concurrently with the probabilistic seismic hazard analysis to assist in defining the MCE in accordance with Section 21.2.2 of ASCE 7-05. The deterministic MCE ground motion is defined by the higher of the following: 150% of a 5% damped spectral response from a characteristic earthquake on a known active fault within the region, or the lower
limit response spectrum from Section 21.2.2 of ASCE 7-05 with limiting spectral response factors, \( S_\delta \) and \( S_1 \), of 1.5 and 0.6 g, respectively, and the site coefficient, \( F_s \) and \( F_v \), factors based on the ASCE 7-05 site soil class.

A DSHA is completed by estimating ground motions for characteristic magnitude earthquakes at the location of active seismic sources in the region. Deaggregation of the 2008 USGS PSHA indicates the primary seismic source for the site is a magnitude 9 earthquake on the CSZ with a source-to-site distance of approximately 74 km. Consistent with the site-specific PSHA, the deterministic analysis was based on the following three ground motion prediction equations (GMPEs) used in the 2008 USGS mapping effort for subduction zone earthquakes: Youngs, et al. (1997); Atkinson and Boore (2003), and Zhao, et al. (2006). In accordance with ASCE 7-05, 150% of the median estimates from Youngs, et al. (1997), Atkinson and Boore (2003), and Zhao, et al. (2006) GMPEs were used with weights of 0.25, 0.25 and 0.5, respectively, to create the deterministic response spectrum. This deterministic response spectrum is shown on Figure 7B.

A check was performed to compare the spectral response values from the site-specific 150% deterministic spectrum with the code-based deterministic lower limit spectrum provided in Section 21.2.2 of ASCE 7-05. The code-based deterministic lower limit spectrum was observed to be higher than the 150% deterministic spectrum at all periods. Therefore, the deterministic lower limit response spectrum is selected to represent the bedrock deterministic MCE response spectrum. Figure 7B shows the comparison between the 150% deterministic spectral values and the deterministic lower limit on MCE response spectrum.

**Development of Target Bedrock Spectra**

The site-specific analysis requires developing a bedrock target spectrum prior to selecting and scaling input acceleration time histories. The bedrock target spectra are developed for the three seismic hazard levels previously discussed in the Probabilistic Seismic Hazard Analysis section of this report. The target spectra for the OLE and CLE conditions can be directly represented by the site-specific probabilistic uniform hazard curves, which correspond to the ground motion with 50% probability of exceedance in 50 years and 10% probability of exceedance in 50 years, respectively. The target bedrock spectrum for the DE is developed in accordance with the requirements of ASCE 7-05. According to Chapter 21 of ASCE 7-05, the controlling target spectrum is developed by comparing the deterministic and probabilistic MCE response spectra and taking the lower of the two spectra to represent the site-specific MCE bedrock response spectrum. Comparison of the MCE probabilistic and deterministic response spectrum for the site are shown on Figure 8B. The probabilistic MCE spectrum is lower than the deterministic spectrum, and, therefore, based on the above criterion, the probabilistic spectrum is defined as the MCE bedrock spectrum.

**Site Response Modeling**

The effect of a specific seismic event on the site is related to the type and thickness of soil overlying the bedrock and the type and quantity of seismic energy delivered to the bedrock by the earthquake. Site response analysis was completed to estimate this site-specific behavior in accordance with ASCE Seismic Design of Pile-Supported Piers and Wharves. The site response analysis consisted of three components: 1) selection of target bedrock response spectrum, 2) numerical modeling to analyze the site-specific behavior of the soils using horizontal ground motion acceleration time histories scaled to the approximate level of the target bedrock response spectrum over the periods of interest, and 3) calculation of the ratio of the
surface response spectra values to the bedrock response spectra values, at each spectral period, to develop a recommended ground surface response spectrum.

The site response modeling was completed using the D-MOD2000 program by GeoMotions, LLC. D-MOD2000 is a one-dimensional non-linear, time-domain site response modeling program capable of capturing the nonlinear hysteretic soil behavior during cyclic seismic loading and unloading. The program computes the dynamic response of a layered soil profile to vertically propagating shear waves using total stress or effective stress analyses. The effective stress option provides a means to evaluate the influence of excess pore pressure development and cyclic degradation of soil strength/stiffness (i.e., pore water pressure generation and pore water pressure dissipation and redistribution) on the dynamic response of the soil profile. D-MOD2000 uses the hyperbolic modified Kodner and Zelasko (MKZ) model to characterize the nonlinear stress-strain soil behavior. The MKZ parameters are generally obtained by fitting the hyperbolic model to published empirical curves.

Within the D-MOD2000 program, the user creates a discretized soil profile and inputs a variety of soil modeling parameters derived from field and laboratory testing and established correlations in the geotechnical literature. A suite of scaled earthquake records are input into the program and propagated up through the soil column to the ground surface. From the modeled ground surface response for a particular soil profile, a Spectral Acceleration Ratio (SAR) can be determined for each earthquake record as the ratio of ground surface to bedrock spectral acceleration (S_{\text{surface}}/S_{\text{bedrock}}) at selected periods.

**Input Parameters**

For the proposed dock expansion, D-MOD2000-based total stress analyses were performed for evaluating the seismic response and performance of the soil underlying the site. First, a generalized subsurface profile for the site was developed based on our subsurface explorations. A shear wave velocity profile at the proposed dock expansion site was estimated based on boring B-3 and B-4 drilled near the face of the proposed dock (GRI, 2003) and shear wave velocity measurements in CPT-1 located in the upland area (PBS, 2006). The ratio of the effective overburden stresses to the one-fourth power, \( (\sigma'_{\text{v, Dock}}/\sigma'_{\text{v, CPT-1}})^{0.25} \), was used as an adjustment factor to derive the shear wave velocity profiles from CPT-1 measurements. Table 2B summarizes the assumed soil profile at the proposed dock expansion site and the shear wave velocity profile used in the D-MOD program. Review of logs of water well located in the upland area indicates bedrock is at an approximate depth of 260 ft below the mudline (GRI, 1993).

<table>
<thead>
<tr>
<th>Layer</th>
<th>Soil Type</th>
<th>Depth Below Mudline, ft</th>
<th>Thickness, ft</th>
<th>Shear Wave Velocity, ft/s</th>
<th>Modulus Reduction and Damping Curves</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Loose SAND</td>
<td>2.5</td>
<td>2.5</td>
<td>350</td>
<td>EPRI (1993) Depth = 0 to 20 ft</td>
</tr>
<tr>
<td>2</td>
<td>Loose SAND</td>
<td>5</td>
<td>2.5</td>
<td>350</td>
<td>EPRI (1993) Depth = 0 to 20 ft</td>
</tr>
<tr>
<td>3</td>
<td>Loose SAND</td>
<td>7.5</td>
<td>2.5</td>
<td>350</td>
<td>EPRI (1993) Depth = 0 to 20 ft</td>
</tr>
<tr>
<td>4</td>
<td>Loose SAND</td>
<td>10</td>
<td>2.5</td>
<td>350</td>
<td>EPRI (1993) Depth = 0 to 20 ft</td>
</tr>
<tr>
<td>5</td>
<td>Loose SAND</td>
<td>15</td>
<td>5</td>
<td>500</td>
<td>EPRI (1993) Depth = 0 to 20 ft</td>
</tr>
<tr>
<td>6</td>
<td>Medium dense SAND</td>
<td>20</td>
<td>5</td>
<td>500</td>
<td>EPRI (1993) Depth = 0 to 20 ft</td>
</tr>
<tr>
<td>7</td>
<td>Medium dense SAND</td>
<td>25</td>
<td>5</td>
<td>515</td>
<td>EPRI (1993) Depth = 21 to 50 ft</td>
</tr>
<tr>
<td>8</td>
<td>Medium dense SAND</td>
<td>30</td>
<td>5</td>
<td>515</td>
<td>EPRI (1993) Depth = 21 to 50 ft</td>
</tr>
<tr>
<td>Layer</td>
<td>Soil Type</td>
<td>Depth Below Mudline, ft</td>
<td>Thickness, ft</td>
<td>Shear Wave Velocity, ft/s</td>
<td>Modulus Reduction and Damping Curves</td>
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<tr>
<td>9</td>
<td>Medium dense SAND</td>
<td>35</td>
<td>5</td>
<td>515</td>
<td>EPRI (1993) Depth = 21 to 50 ft</td>
</tr>
<tr>
<td>10</td>
<td>Medium dense SAND</td>
<td>40</td>
<td>5</td>
<td>515</td>
<td>EPRI (1993) Depth = 21 to 50 ft</td>
</tr>
<tr>
<td>11</td>
<td>Medium dense SAND</td>
<td>45</td>
<td>5</td>
<td>700</td>
<td>EPRI (1993) Depth = 21 to 50 ft</td>
</tr>
<tr>
<td>12</td>
<td>Medium dense SAND</td>
<td>50</td>
<td>5</td>
<td>700</td>
<td>EPRI (1993) Depth = 21 to 50 ft</td>
</tr>
<tr>
<td>13</td>
<td>Medium dense SAND</td>
<td>55</td>
<td>5</td>
<td>700</td>
<td>EPRI (1993) Depth = 21 to 50 ft</td>
</tr>
<tr>
<td>14</td>
<td>Medium dense SAND</td>
<td>60</td>
<td>5</td>
<td>700</td>
<td>EPRI (1993) Depth = 21 to 50 ft</td>
</tr>
<tr>
<td>15</td>
<td>Medium dense SAND</td>
<td>65</td>
<td>5</td>
<td>700</td>
<td>EPRI (1993) Depth = 21 to 50 ft</td>
</tr>
<tr>
<td>16</td>
<td>Medium dense SAND</td>
<td>70</td>
<td>5</td>
<td>700</td>
<td>EPRI (1993) Depth = 21 to 50 ft</td>
</tr>
<tr>
<td>17</td>
<td>Medium dense SAND</td>
<td>75</td>
<td>5</td>
<td>950</td>
<td>EPRI (1993) Depth = 51 to 120 ft</td>
</tr>
<tr>
<td>18</td>
<td>Medium dense SAND</td>
<td>80</td>
<td>5</td>
<td>950</td>
<td>EPRI (1993) Depth = 51 to 120 ft</td>
</tr>
<tr>
<td>19</td>
<td>Medium dense SAND</td>
<td>85</td>
<td>5</td>
<td>950</td>
<td>EPRI (1993) Depth = 51 to 120 ft</td>
</tr>
<tr>
<td>20</td>
<td>Medium dense SAND</td>
<td>90</td>
<td>5</td>
<td>950</td>
<td>EPRI (1993) Depth = 51 to 120 ft</td>
</tr>
<tr>
<td>21</td>
<td>Medium dense SAND</td>
<td>95</td>
<td>5</td>
<td>950</td>
<td>EPRI (1993) Depth = 51 to 120 ft</td>
</tr>
<tr>
<td>22</td>
<td>Medium dense SAND</td>
<td>100</td>
<td>5</td>
<td>950</td>
<td>EPRI (1993) Depth = 51 to 120 ft</td>
</tr>
<tr>
<td>23</td>
<td>Dense SAND</td>
<td>110</td>
<td>10</td>
<td>1,090</td>
<td>EPRI (1993) Depth = 51 to 120 ft</td>
</tr>
<tr>
<td>24</td>
<td>Dense SAND</td>
<td>120</td>
<td>10</td>
<td>1,090</td>
<td>EPRI (1993) Depth = 51 to 120 ft</td>
</tr>
<tr>
<td>25</td>
<td>Dense SAND</td>
<td>130</td>
<td>10</td>
<td>1,175</td>
<td>EPRI (1993) Depth = 121 to 250 ft</td>
</tr>
<tr>
<td>26</td>
<td>Dense SAND</td>
<td>140</td>
<td>10</td>
<td>1,265</td>
<td>EPRI (1993) Depth = 121 to 250 ft</td>
</tr>
<tr>
<td>27</td>
<td>Dense SAND</td>
<td>150</td>
<td>10</td>
<td>1,355</td>
<td>EPRI (1993) Depth = 121 to 250 ft</td>
</tr>
<tr>
<td>28</td>
<td>Dense SAND</td>
<td>160</td>
<td>10</td>
<td>1,445</td>
<td>EPRI (1993) Depth = 121 to 250 ft</td>
</tr>
<tr>
<td>29</td>
<td>Dense SAND</td>
<td>170</td>
<td>10</td>
<td>1,535</td>
<td>EPRI (1993) Depth = 121 to 250 ft</td>
</tr>
<tr>
<td>30</td>
<td>Dense SAND</td>
<td>180</td>
<td>10</td>
<td>1,625</td>
<td>EPRI (1993) Depth = 121 to 250 ft</td>
</tr>
<tr>
<td>31</td>
<td>Dense SAND</td>
<td>190</td>
<td>10</td>
<td>1,715</td>
<td>EPRI (1993) Depth = 121 to 250 ft</td>
</tr>
<tr>
<td>32</td>
<td>Dense SAND</td>
<td>200</td>
<td>10</td>
<td>1,805</td>
<td>EPRI (1993) Depth = 121 to 250 ft</td>
</tr>
<tr>
<td>33</td>
<td>Dense SAND</td>
<td>210</td>
<td>10</td>
<td>1,895</td>
<td>EPRI (1993) Depth = 121 to 250 ft</td>
</tr>
<tr>
<td>34</td>
<td>Dense SAND</td>
<td>220</td>
<td>10</td>
<td>1,985</td>
<td>EPRI (1993) Depth = 121 to 250 ft</td>
</tr>
<tr>
<td>35</td>
<td>Dense SAND</td>
<td>230</td>
<td>10</td>
<td>2,075</td>
<td>EPRI (1993) Depth = 121 to 250 ft</td>
</tr>
<tr>
<td>36</td>
<td>Dense SAND</td>
<td>240</td>
<td>10</td>
<td>2,165</td>
<td>EPRI (1993) Depth = 121 to 250 ft</td>
</tr>
<tr>
<td>37</td>
<td>Dense SAND</td>
<td>250</td>
<td>10</td>
<td>2,255</td>
<td>EPRI (1993) Depth = 121 to 250 ft</td>
</tr>
<tr>
<td>38</td>
<td>Dense SAND</td>
<td>260</td>
<td>10</td>
<td>2,345</td>
<td>EPRI (1993) Depth = 251 to 500 ft</td>
</tr>
</tbody>
</table>

The unit weight for the uniform sand layers was estimated to be approximately 120 pounds per cubic foot (pcf). The sand layers encountered throughout the soil profile were assigned the depth-dependent EPRI (1993) deep cohesionless soil modulus and damping curves that account for the effects of confining pressure. The half-space boundary condition at the base of the model was represented by a visco-elastic boundary with a unit weight of 130 pcf and a shear wave velocity of 2,500 ft per second (ft/s).

**Ground Motion Selection and Scaling**

For the site response analyses, a suite of recorded horizontal ground motion acceleration time histories were selected from events having magnitudes and fault rupture distances consistent with those that control the bedrock target spectrum. The bedrock target spectrum for the site was developed for Site Class B, or
rock site, conditions in accordance with the method outlined in the Target Bedrock Spectrum section of this report for OLE, CLE and MCE hazard levels. Since the seismicity of the site is dominated by the CSZ earthquake source at the 2,475-year hazard level, seven subduction zone earthquakes records were selected and utilized for the MCE site response modeling. Additionally, deaggregation of the seismic hazard indicates a significant contribution from local crustal, subcrustal, and CSZ sources at the 475- and 72-year hazard levels. Therefore, ground motion records from crustal, subcrustal, and subduction zone earthquakes were used for the site response modeling of the 475- and 72-year hazard levels. These records were checked for obvious errors, missing data points, and other anomalies and were transformed into a uniform data format. The selected time histories used for the site response modeling are summarized in Table 3B.

### TABLE 3B SUMMARY OF GROUND MOTION RECORDS SELECTED FOR SITE RESPONSE MODELING

<table>
<thead>
<tr>
<th>No.</th>
<th>Earthquake/Year</th>
<th>Magnitude Mw</th>
<th>Station Name</th>
<th>Station ID</th>
<th>Record Source</th>
<th>Rupture Distance, km</th>
<th>PGA, g</th>
<th>Sampling Frequency, Hz</th>
<th>Record Length, sec</th>
</tr>
</thead>
<tbody>
<tr>
<td>MCE (2,475-year return period)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Michoacan/1985</td>
<td>8.1</td>
<td>Caleta deCampos</td>
<td>CaletaDecamposNS</td>
<td>COSMOS</td>
<td>27</td>
<td>0.13</td>
<td>200</td>
<td>51</td>
</tr>
<tr>
<td>2</td>
<td>Tohoku/2011</td>
<td>9</td>
<td>Sawara</td>
<td>CHB004EW</td>
<td>KNET</td>
<td>317</td>
<td>0.29</td>
<td>100</td>
<td>300</td>
</tr>
<tr>
<td>3</td>
<td>Tohoku/2011</td>
<td>9</td>
<td>Ichikawa-Kita</td>
<td>CHB028NS</td>
<td>KNET</td>
<td>360</td>
<td>0.22</td>
<td>100</td>
<td>300</td>
</tr>
<tr>
<td>4</td>
<td>Valparaiso/1985</td>
<td>7.8</td>
<td>Endesa</td>
<td>EndesaNS</td>
<td>COSMOS</td>
<td>122</td>
<td>0.12</td>
<td>200</td>
<td>96</td>
</tr>
<tr>
<td>5</td>
<td>Tohoku/2011</td>
<td>9</td>
<td>Sohma</td>
<td>FKS001NS</td>
<td>KNET</td>
<td>175</td>
<td>0.63</td>
<td>100</td>
<td>300</td>
</tr>
<tr>
<td>6</td>
<td>Tohoku/2011</td>
<td>9</td>
<td>Iwaki</td>
<td>FKS011NS</td>
<td>KNET</td>
<td>203</td>
<td>0.39</td>
<td>100</td>
<td>300</td>
</tr>
<tr>
<td>7</td>
<td>Michoacan/1985</td>
<td>8.1</td>
<td>LaUnion</td>
<td>LaunionEW</td>
<td>COSMOS</td>
<td>79</td>
<td>0.17</td>
<td>200</td>
<td>63</td>
</tr>
</tbody>
</table>

| CLE (475-year return period) |
| 1   | Michoacan/1985        | 8.1          | Caleta deCampos | CaletaDecamposNS | COSMOS         | 27                   | 0.13   | 200                   | 51                 |
| 2   | Chi-Chi Taiwan/1999   | 6.2          | TCU071        | ChiChi71E    | PEER          | 15                   | 0.18   | 200                   | 56                 |
| 3   | Tohoku/2011           | 9            | Sohma        | FKS001NS    | KNET          | 175                  | 0.63   | 100                   | 300                |
| 4   | Michoacan/1985        | 8.1          | LaUnion      | LaUnionEW   | COSMOS         | 79                   | 0.17   | 200                   | 63                 |
| 5   | Mammoth Lakes/1980    | 6.1          | Long Valley Dam | Mammoth90  | PEER          | 15.5                 | 0.27   | 200                   | 30                 |
| 6   | Tohoku/2011           | 9            | Taiwa        | MYG009EW    | KNET          | 183                  | 0.56   | 100                   | 300                |
| 7   | Nisqually/2001        | 6.8          | Seattle      | USGS2181 _360 | USGS          | 63                   | 0.075  | 200                   | 96                 |

| OLE (72-year return period) |
| 1   | Irpinia Italy/1980    | 6.2          | Brienza      | BRZ270      | PEER          | 42                   | 0.041  | 345                   | 44                 |
| 2   | Morgan Hill/1984      | 6.1          | UCSC         | LOB050      | PEER          | 52                   | 0.04   | 200                   | 60                 |
| 3   | Whittier Narrows/1987 | 6.0          | Rancho Palos Verdes - Luconia | LUC276  | PEER          | 42                   | 0.019  | 200                   | 23                 |
| 4   | Chi-Chi Taiwan/1999   | 6.2          | HWA031       | HWA031E     | PEER          | 39                   | 0.024  | 200                   | 58                 |
| 5   | Chi-Chi Taiwan/1999   | 6.2          | HWA038       | HWA038N     | PEER          | 41                   | 0.053  | 200                   | 72                 |
| 6   | Nisqually/2001        | 6.8          | Seattle      | USGS2181   | USGS          | 63                   | 0.075  | 200                   | 96                 |
| 7   | Nisqually/2001        | 6.8          | Aberdeen     | USGS7035    | USGS          | 86                   | 0.066  | 200                   | 121                |
Subsequent to selection of the time histories, the ground motion records were linearly modified using amplitude scaling so that the mean response spectra of the seven recordings reasonably match the target spectrum for each of the hazard levels. The amplitude-scaling process involves selecting a single factor for each time history and multiplying the acceleration time history by this factor so that its response spectrum is compatible with the input bedrock target spectrum over the period of interest. The records were scaled to match a period range of interest of about 0.8 second, or the approximate natural period of the soil column.

Site Response Results
Using the generalized subsurface profile, the target spectra developed at the bedrock, and the strong ground motion records listed in the preceding tables, pseudo acceleration response spectra were computed for the proposed dock site with the D-MOD2000 nonlinear model. The ground surface response spectra were developed at 5% of critical damping. The ground surface spectra were compared with the input rock spectra to quantify amplification and/or attenuation through the soil column at the site. The ratio of ground surface to bedrock spectral accelerations, defined as the spectral amplification ratio (SAR), is shown on Figure 9B for the MCE, CLE, and OLE hazard levels. Review of the spectral amplification ratios indicates the largest soil amplifications for the MCE, CLE, and OLE hazard levels occur at periods ranging from approximately 0.6 to 1.2 seconds. The observed shift in peak amplification between the MCE, CLE, and OLE hazard levels is generally consistent with expected results for higher levels of shaking.

To estimate ground surface site response throughout the range of spectral periods, the target response spectra is multiplied by the SAR to determine the ground surface response spectrum. The results of the site-specific response modeling are shown on Figures 10B, 11B and 12B for the OLE, CLE and MCE hazard levels, respectively. The figures also include the code-based spectrum developed using site amplification factors based on the appropriate Site Class type. The site is generally designated as Site Class E for the proposed dock expansion based on the average shear wave velocity ($V_{S100}$) in the upper 100 ft per Section 20.4 of ASCE 7-05. The code-based spectrum is typically derived based on the 0.2 and 1 second spectral accelerations values at the bedrock and site coefficients (i.e., $F_a$ and $F_v$) provided in Table 11.4-1 and Table 11.4-2 of ASCE 7-05. The code-based Site Class E coefficients and the spectral values corresponding to 0.2 and 1 second periods are provided on Table 4B for the OLE, CLE and MCE hazard levels. The two spectral values are obtained from the map of spectral acceleration parameters provided in Chapter 22 of ACSE 7-05 for the MCE condition while the spectral values corresponding to 72- and 475-year return periods are obtained from USGS data for the OLE and CLE conditions, respectively.

<table>
<thead>
<tr>
<th>Hazard Level</th>
<th>$S_s$, g</th>
<th>$S_s$, g</th>
<th>$F_a$</th>
<th>$F_v$</th>
</tr>
</thead>
<tbody>
<tr>
<td>OLE</td>
<td>0.12</td>
<td>0.04</td>
<td>2.5</td>
<td>3.5</td>
</tr>
<tr>
<td>CLE</td>
<td>0.44</td>
<td>0.16</td>
<td>1.89</td>
<td>3.31</td>
</tr>
<tr>
<td>MCE</td>
<td>0.86</td>
<td>0.34</td>
<td>1.07</td>
<td>2.66</td>
</tr>
</tbody>
</table>

In general, the recommended OLE, CLE, and MCE level response spectra are derived by comparing the results of the site response modeling (site-specific spectra) with the code-based spectra (i.e., Site Class E) that represent the site class at the site. In addition, the code specifies that the site-specific spectral accelerations at the ground surface should not be taken as less than 80% of the of the Site Class response.
The development of the recommended response spectra for each of the hazard levels is discussed below.

**OLE Hazard Level.** Comparisons of the site-specific mean ground surface spectrum with the Site Class E spectrum for the OLE hazard level are shown on Figure 10B. As shown on the figure, the response spectrum from site response modeling is lower than the code-based 80% Site Class E spectrum at all periods. Therefore, the code-based 80% Site Class E spectrum is recommended for estimating the spectral acceleration at all periods for the OLE hazard level.

**CLE Hazard Level.** The site-specific mean ground surface response spectrum is generally less than 80% of Site Class E response spectrum except at periods between 0.9 and 1.4 seconds. At periods between 0.9 and 1.4 seconds, the site-specific response spectrum slightly exceeds 80% of Site Class E response spectrum. Therefore, the recommended response spectrum was derived by encompassing the 80% Site Class E spectrum and the higher site-specific spectral values in the period range between 0.9 and 1.4 seconds. Figure 11B shows the recommended response spectrum for the CLE hazard level.

**DE Hazard Level.** The estimated MCE ground surface response spectra from the D-MOD total stress analysis typically exceed 80% of Site Class E spectra at periods less than 1.9 seconds. At periods greater than approximately 1.9 seconds, the site-specific response spectrum is less than 80% of Site Class E, which is the minimum spectral amplification allowed by ASCE 7-05. Therefore, the recommended MCE spectrum was developed by encompassing 90% of the peak value of the site-specific spectrum (per Section 21.4 of ASCE 7-05) at short periods and the code-based lower limit 80% of Site Class E spectrum at long periods. The recommended MCE response spectrum is shown on Figure 12B. The design earthquake (DE) response spectrum is determined by taking two-thirds of the MCE response spectrum and is shown on Figure 13B.

**Conclusions**

Site-specific response modeling for the proposed dock expansion site was completed using total stress analysis based on the generalized subsurface profile developed from the subsurface explorations. The site response modeling was performed for three seismic hazard levels, i.e., OLE, CLE, and DE, to meet the requirements of ASCE Seismic Design of Pile-Supported Piers and Wharves.

For the OLE hazard level, we recommend 80% of the code-based Site Class E response spectrum, and for the CLE and DE hazard levels, the recommended spectrum is based on a combination of the site-specific spectrum from site response modeling and 80% of the code-based Site Class E response spectrum.

**References**


Electric Power Research Institute (EPRI), 1993, Guidelines for site-specific ground motions, Palo Alto, California, November TR-102293.


GRI, 1993, Preliminary geotechnical investigation, proposed steel mill North Port site, Port of Kalama, Washington.


A) TECTONIC MAP OF PACIFIC NORTHWEST, SHOWING ORIENTATION AND EXTENT OF CASCADIA SUBDUCTION ZONE (MODIFIED FROM DRAGERT AND OTHERS, 1994)

B) EAST-WEST CROSS-SECTION THROUGH WESTERN OREGON AT THE LATITUDE OF KALAMA, WASHINGTON, SHOWING THE SEISMIC SOURCES CONSIDERED IN THE SITE-SPECIFIC SEISMIC HAZARD STUDY (MODIFIED FROM GEOMATRIX, 1995)
TIME OF MOST RECENT SURFACE RUPTURE
- Holocene (<10,000 years) or post last glaciation (<15,000 years; 15 kya)
- No historic ruptures in Oregon to date
- Lake Quaternary (~10,000–100,000 years; post-glacial and pre-late Holocene)
- Late and middle Quaternary (~70,000–10,000 years; 75 kya)
- Quaternary, undifferentiated (~400,000 years; < 4-Ma)
- Class B structure (age or origin uncertain)

SLIP RATE
- >5 mm/year
- 1.0-5.0 mm/year
- 0.2-1.0 mm/year
- <0.2 mm/year

TRACE
- Mostly continuous at map scale
- Mostly discontinuous at map scale
- Inferred or concealed

STRUCTURE TYPE AND RELATED FEATURES
- Normal or high-angle reverse fault
- Strike-slip fault
- Thrust fault
- Anticline fold
- Syncline fold
- Monocline fold
- Plunge direction of fold
- Fault scarp or marker

DETAILED STUDY SITES
- Trench site
- Subduction zone study site

CULTURAL AND GEOGRAPHIC FEATURES
- Divided highway
- Primary or secondary road
- Permanent or intermittent stream
- Permanent or intermittent lake

FAULT NUMBER | NAME OF STRUCTURE
--- | ---
714 | HELVETIA FAULT
715 | BEAVERTON FAULT
716 | CANYON CREEK FAULT
717 | NEWBERG FAULT
718 | GALES CREEK FAULT ZONE
867 | EAGLE CREEK THRUST FAULT
868 | BULL RUN THRUST FAULT
871 | MOUNT ANGELO FAULT
874 | BILON FAULT
875 | OATFIELD FAULT
876 | EAST BANK FAULT
877 | PORTLAND HILLS FAULT
878 | GRANT BUTTE FAULT
879 | DAMASCUS-TICKLE CREEK FAULT ZONE
880 | LACAMAS LAKE FAULT
881 | TILLAMOOK BAY FAULT ZONE
882 | HAPPY CAMP FAULT

From: PERSONIUS, S.F., AND OTHERS, 2003, MAP OF QUATERNARY FAULTS AND FOLDS IN OREGON, USGS OPEN FILE REPORT OFR-03-095.
Figure 21. Location of the eastern edge of earthquake-rupture zones on the Cascadia subduction zone for the various models used in this study relative to the surficial expression of the trench: top, base of the elastic zone; mid, midpoint of the transition zone; bottom, base of the transition zones; base, base of the model that assumes ruptures extend to about 30-kilometers depth. Figure provided by Ray Weldon.
Magnitude-frequency distribution of the Cascadia subduction zone.

BEDROCK RESPONSE SPECTRA FOR DIFFERENT PROBABILISTIC HAZARD LEVELS (5% DAMPING)

FEB. 2015
JOB NO. W1153
FIG. 6B
Deterministic Lower Limit (Fig. 21.2-1 in ASCE 7-05)

150% Deterministic Spectrum
DETERMINISTIC AND PROBABILISTIC MCE BEDROCK SPECTRA COMPARISON
(5% DAMPING)
SPECTRAL AMPLIFICATION RATIO (SAR) FOR DIFFERENT PROBABILISTIC HAZARD LEVELS
(5% DAMPING)

SAR for OLE (72-year) Hazard Level
SAR for CLE (475-year) Hazard Level
SAR for MCE (2,475-year) Hazard Level
OLE RESPONSE SPECTRA
AT GROUND SURFACE
(5% DAMPING)

Period, T, seconds

Spectral Acceleration, g
MCE RESPONSE SPECTRA
AT GROUND SURFACE
(5% DAMPING)
Recommended Design Earthquake (DE) Spectrum at Ground Surface

DE (2/3 MCE) RESPONSE SPECTRUM AT GROUND SURFACE (5% DAMPING)