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Summary of Available Data and Report of Expected Earthquake Risk

Oregon Critical Energy Infrastructure Hub

Portland, Oregon

Prepared for
Multnomah County

April 23, 2021
Revised May 11, 2021
2nd Revision June 29, 2021
Job No. 154-035-019

SALUS RESILIENCE

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1.0 INTRODUCTION AND PROJECT UNDERSTANDING

The Cascadia Subduction Zone (CSZ) reaches from Vancouver, Canada to Cape Mendocino, California and has the capacity to produce earthquakes with a magnitude of 8.0 or higher. Geologists previously believed that these large earthquakes from the CSZ have a recurrence interval of 400 to 600 years; however, research done by a team of scientists at Oregon State University proved the recurrence interval is closer to 350 years. The most recent major earthquake was on January 26, 1700, a little over 300 years ago, with an estimated magnitude of 9.0 on the CSZ. Research by Oregon State University indicates that Oregon has a 37 percent chance of a large earthquake (> M8) from the CSZ within the next 50 years. Based on our understanding of these earthquakes and a recent study by the Oregon Department of Geology and Mineral Industries (DOGAMI), such an earthquake will cause significant damage to infrastructure throughout Oregon, the Portland Metro Region, and Multnomah County.

Part of Oregon's critical infrastructure includes the Oregon Critical Energy Infrastructure (CEI) Hub, which is located on a 6-mile stretch of the west shore of the lower Willamette River, as shown on Figure 1.1. The CEI Hub houses approximately 90 percent of the liquid fuel needed to support the state of Oregon and all of the jet fuel used by the Portland International Airport, as well as other hazardous materials (DOGAMI 2012). New technology, data, and mapping have greatly expanded our understanding of the effects of seismic hazards in our region, including the effects of earthquakes to soft and loose fill and alluvial soils, such as those mapped at the location of the CEI Hub site. These soils are prone to seismically induced strength loss, settlement, and slope failure or lateral spread. The 2017 DOGAMI data indicate that significant displacement will occur in this area during a 9.0 CSZ event. In addition to the hazards related to the soils at the site, a large portion of the existing infrastructure at the CEI Hub was constructed prior to our understanding of Oregon's seismic risk, including tanks constructed over 100 years ago that are still being used for hazardous material storage. The age of the tanks and infrastructure and the soil vulnerabilities result in significant risk to the CEI Hub infrastructure and the materials that are stored there.

The purpose of this study is to evaluate the effects of an anticipated seismic event for the region on the CEI Hub and its infrastructure in order to support an evaluation of the economic ramifications for Multnomah County (County). Based on the scenarios developed by DOGAMI for emergency planning, the goals for this project, and our understanding of the geology in the area, the 9.0 CSZ earthquake scenario will be used for this evaluation. This earthquake scenario is the most likely to occur in the next 50 years and will be the most difficult for emergency response and long-term recovery because it will affect the entire Pacific Northwest.

This report summarizes the first phase in our evaluation and includes a bibliography of the data and reports used in our evaluation as well as a detailed summary of the earthquake scenario and geotechnical risk evaluation for the project. The impacts of the earthquake scenario outlined herein on the CEI Hub are not addressed in this report; however, this information will be used in the next phase of the project to evaluate the CEI Hub impacts.

1.1 Geologic Setting of the CEI Hub

The CEI Hub is within the city of Portland, Oregon and lies within the Portland Basin, one of several basins that form the Puget-Willamette forearc trough of the Cascadia subduction system (Evarts et al 2009). This trough extends from the Washington-Canada border to approximately Eugene, Oregon, includes the Puget Sound and Willamette River Valley, a distance of nearly 350 miles. Contractional tectonic stresses from the convergent CSZ also create a series of north- to northwest-trending folds that extend from the Pacific coast east to the Cascade Mountains. These folds form the valleys, hills, and mountains characteristic of northwest Oregon and the Pacific Northwest in general. The Portland Basin has also been receiving sediments from the continental-scale Columbia River system for over 20 million years (Evarts et al 2009), of which the Willamette River is a tributary and the source of the near surface sediments at the CEI Hub.

The oldest deposits in the basin form the uplands that surround the valley and are composed of 30- to 40-million-year old volcanic and marine rocks and 15- to 16-million-year old basalt flows of the Columbia River Basalt Group. These rocks were folded and uplifted along faults at the southwest and northeast margins of the Portland Basin, which include the adjacent Portland Hills. The basin itself began to form approximately 20 million years ago and is filled with a thick accumulation of river sediments, including the Troutdale Formation, a gravel to cobble conglomerate found widely throughout the Portland Basin (Evarts et al 2009).

Near the end of the last ice age, a series of cataclysmic floods flowing down the Columba River Gorge repeatedly inundated the Portland Basin up to 400 feet above sea level (Evarts et al 2009). These floods originated from the repeated failing of a glacial ice dam in northwestern Montana between 16,000 and 12,000 years ago and are collectively called the Missoula Floods. While massive gravel bars were formed in the eastern Portland Basin closest to the river, these floodwaters slowed and ponded behind the narrower Columbia River valley downstream, dropping slack water deposits of sand and silt across the entire Willamette River valley. Since the end of the last ice age 13,000 years ago, sea levels have risen over 370 feet, causing the Columbia and Willamette rivers to rapidly deposit sediments across the basin, typically through overbank deposition during yearly snowmelt floods (Evarts et al 2009). These loose sand and silt deposits have been overlain by fill in places where floodplains and wetlands were developed along the banks of the Willamette and Columbia rivers.

1.2 Seismic Setting of the CSZ

Oregon sits near the contact between two large crustal tectonic plates. The Juan de Fuca Plate forms the floor of the Pacific Ocean off the coast of the northwestern United States and moves northeastward from its spreading ridge boundary with the Pacific Plate at an average rate of approximately 1.5 inches per year. As it converges with continental North America, the Juan de Fuca Plate dips below (or “subducts”) beneath the North American Plate, forming a shallow, eastward-dipping contact interface. This boundary is known as the CSZ and is responsible for the seismicity in the Pacific Northwest, producing earthquakes associated with three types of source zones: subduction interface, subduction intraslab, and shallow crustal.

Based on geologic and historical evidence, CSZ interface earthquakes occur an average of every 350 years in the form of magnitude 8 to 9.2 earthquakes. Interface earthquakes (such as the 2011 magnitude M9.0 Tohoku earthquake in northeastern Japan) are some of the largest magnitude earthquakes on record. Characteristics of this type of earthquake may include very large ground accelerations, shaking durations in excess of 3 minutes, and strong long-period ground motions that may particularly affect tall or long-period structures and deep soft soils.

Shallow crustal faults are caused by cracking of the continental crust resulting from the stress that builds as the subduction zone plates remain locked together. The Portland Hills, Oatfield, and East Bank faults run approximately in a northwest-southeast direction through downtown Portland and are generally believed to be capable of producing earthquake events in the study area. However, earthquake events on these crustal faults are less likely than the 9.0 CSZ earthquake.

Based on our discussions with the County and the project team, the scenario that will be used for this project is a M9.0 on the CSZ. This event has been widely used for evaluation and emergency planning in the Portland Metro area and Oregon because of the higher probability of its occurrence and greater area that will experience damage. Damage to the entire Pacific Northwest is expected during this scenario resulting in a much larger challenge for emergency response and recovery. DOGAMI has completed a comprehensive damage estimate based on shaking data for a 9.0 CSZ event. Based on their mapping, the CEI Hub is expected to experience very strong to severe shaking from aggregated earthquake sources, with severe shaking and moderate to heavy damage potential during a magnitude 9.0 CSZ earthquake as shown on Figure 1.2.

The anticipated ground shaking will also cause weaknesses within the subsurface soils. Liquefaction is a phenomenon where ground shaking in saturated granular (sand or silt) soils creates a rapid increase in pore water pressure that results in the sudden loss of shear strength in the soil. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water. Liquefaction can result in settlement and strength loss, which can impact foundations. DOGAMI has mapped generalized liquefaction hazard at the site as moderate to high as shown on Figure 1.3. Additionally, liquefaction can cause global instability and may result in lateral spread towards water bodies and other low areas. DOGAMI has mapped the potential permanent ground deformation due to lateral spreading at the site as being between 39 and 173 inches, as shown on Figure 1.4.

1.3 History of the Oregon CEI Hub

The CEI Hub development began in the early 1900s, with the first tanks constructed in approximately 1907 at the Phillips 66 property. Since the beginning of development, the CEI Hub has expanded to five distinct areas, with 11 owners and 31 properties as indicated in Table 1.1 below. For the purposes of our evaluation, we have separated the CEI Hub into five distinct geographic areas for geotechnical evaluation. The property ownership and designated areas are shown on Figure 1.5. Closer views of each area are show in Figure 1.6 through Figure 1.10. We reviewed data collected from the State Fire Marshall, City of Portland, Portland State University (PSU), and historical aerial and satellite imagery to aid in the evaluation.

Table 1.1 – CEI Hub Areas

Area 1 - Kinder Morgan North					
Property Name	Address	City	State	Zip	Property ID
Kinder Morgan - North	11400 NW ST HELENS RD	PORTLAND	OR	97231	R323828
Area 2 – Linnton					
Property Name	Address	City	State	Zip	Property ID
BP West Coast	9930 WI/ NW ST HELENS RD	PORTLAND	OR	97231	R323779
BP West Coast	9930 NW ST HELENS RD	PORTLAND	OR	97231	R498331
BP West Coast	9900 WI/ NW ST HELENS RD	PORTLAND	OR	97231	R323771
BP West Coast	9930 WI/ NW ST HELENS RD	PORTLAND	OR	97231	R323758
Shore Terminals / Nustar	9420 WI/ NW ST HELENS RD	PORTLAND	OR	97231	R518296
Shore Terminals / Nustar	9420 WI/ NW ST HELENS RD	PORTLAND	OR	97231	R491070
Shore Terminals / Nustar	9400 S/ NW ST HELENS RD	PORTLAND	OR	97231	R324088
Shore Terminals / Nustar	9420 NW ST HELENS RD	PORTLAND	OR	97231	R518295
Shore Terminals / Nustar	9420 WI/ NW ST HELENS RD	PORTLAND	OR	97231	R518294
Area 3 - NW Natural					
Property Name	Address	City	State	Zip	Property ID
Pacific Terminal Services	7900 NW ST HELENS RD	PORTLAND	OR	97210	R324159
NW Natural	7900 WI/ NW ST HELENS RD	PORTLAND	OR	97210	R324171
NW Natural	7900 WI/ NW ST HELENS RD	PORTLAND	OR	97210	R324170
NW Natural	7598 NW ST HELENS RD	PORTLAND	OR	97210	R324113
NW Natural	7900 WI/ NW ST HELENS RD	PORTLAND	OR	97210	R324172
NW Natural	7441 SW/ NW ST HELENS RD	PORTLAND	OR	97210	R324165
NW Natural	7441 NW ST HELENS RD	PORTLAND	OR	97210	R324160
NW Natural	7540 NW ST HELENS RD	PORTLAND	OR	97210	R502592
NW Natural	7540 WI/ NW ST HELENS RD	PORTLAND	OR	97210	R324213
Area 4 – Willbridge					
Property Name	Address	City	State	Zip	Property ID
Kinder Morgan - South	5800 WI/ NW ST HELENS RD	PORTLAND	OR	97210	R324222
Kinder Morgan - South	5800 NW ST HELENS RD	PORTLAND	OR	97210	R121076
Kinder Morgan - South	6080 WI/ NW FRONT AVE	PORTLAND	OR	97210	R315782
Chevron	5533 NW DOANE AVE	PORTLAND	OR	97210	R315798
Chevron	5533 WI/ NW DOANE AVE	PORTLAND	OR	97210	R315771
Conoco Phillips	5528 WI/ NW DOANE AVE	PORTLAND	OR	97210	R315810
Conoco Phillips	5528 NW DOANE AVE	PORTLAND	OR	97210	R315769
Zenith Energy Terminals	5501 NW FRONT AVE	PORTLAND	OR	97210	R315845
Zenith Energy Terminals	5501 NW FRONT AVE	PORTLAND	OR	97201	R315777
McCall Oil	5700 NW FRONT AVE	PORTLAND	OR	97210	R315872
McCall Oil	5480 WI/ NW FRONT AVE	PORTLAND	OR	97210	R315786
Area 5 - Equilon					
Property Name	Address	City	State	Zip	Property ID
Equilon	3610-3640 NW ST HELENS RD	PORTLAND	OR	97210	R315819

The earliest available aerial photographs of the study area were taken by the U.S. Army Corp of Engineers (USACOE) in 1923 with coverage limited to Area 4 and Area 5. Tanks associated with Kinder Morgan and Chevron are visible on the 1923 aerial photograph, which displays approximately 30 percent of the tanks present today.

1.3.1 Area 1 – Kinder Morgan North

Area 1 includes one property owned by Kinder Morgan and is located at 11400 NW St. Helens Road on the north end of the Linnton neighborhood and includes riverfront as shown on Figure 1.5. The earliest available photograph of Area 1 is from 1936. At that time, 12 tanks are visible on the southwest portion of the property, and the northeast portion of the property is a combination of industrial land and the Willamette River. Extensive in-river filling of the northeast portion of the property occurred through 1941 when five additional tanks were constructed on the new land. Between 1954 and 1955, three additional tanks were added to the northeast portion of the property. Additional land was added along the shoreline of the property between 1956 and 1961. Based on available data, the oldest tank remaining at this property was constructed in 1914 and is currently out of service. Of the original tanks present in 1936, three were replaced in 1944, 1958, and 2011. Two of the original tanks have been removed permanently. Based on data provided by the City of Portland (City), PSU, and satellite imagery, there are currently 33 tanks present (Cone 2020 and Dusicka 2019). Additional details are provided in *Section 4.0 Geologic Risk of the CEI Hub in a CSZ Earthquake*.

1.3.2 Area 2 – Linnton

Area 2 includes nine properties owned by BP West Coast at 9900 and 9930 NW St Helens Road and Shore Terminals/Nustar at 9400 and 9420 NW St Helens Road. All nine properties are located north of the St. Johns Bridge and include riverfront.

1.3.2.1 BP West Coast

BP West Coast includes four properties. Three located on the west side of NW St Helens Road with no tank infrastructure and one property with tanks located on the east side of NW St Helens Road along the Willamette River. The earliest available photograph of the BP West Coast property is a 1940 aerial photograph that shows eight tanks present on the southern portion of the property, and two on the northern portion of the property. Between 1948 and 1957, the shoreline of BP West Coast was filled to add approximately 30 feet of land between the existing tanks and the Willamette River. By 1962, the additional tanks present today were constructed on the northern portion of the property. Based on data provided by the City, PSU, and satellite imagery, there are currently 30 tanks present (Cone 2020 and Dusicka 2019).

1.3.2.2 Shore Terminals/Nustar

Shore Terminals/Nustar includes five properties. Two properties on the west side of NW St. Helens Road include vacant land, small office buildings, and four small tanks that appear to have been installed between 1968 and 1977. Two properties located on the east side of NW St Helens Road include extensive tank infrastructure along the Willamette River. The earliest available photograph of the Shore Terminals/Nustar property is a 1939 aerial photograph that shows that the majority of the tank infrastructure is located on the northern portion of the northern property. That photograph also shows the southern portion of the property as well as the adjoining southern property are partially vegetated with filling activity visible. Additional filling continued on both properties through 1962, and the number of tanks approximately doubled. A large expansion of tanks on the southern property occurred between 1977 and 1984 and included additional shoreline filling. Two additional tanks were constructed on the southern portion of the southern property in 2007. The third property located on the east side of NW St Helens Road is a small, vacant piece of land on the northwest corner of the main Shore Terminals/Nustar property. Based on data provided by the City, PSU, and satellite imagery, there are currently 39 tanks present (Cone 2020 and Dusicka 2019).

1.3.3 Area 3 – NW Natural

Area 3 includes nine properties owned by Pacific Terminal Services and NW Natural at 7900, 7598, 7441, and 7540 NW St Helens Road. All nine properties are located between the St. Johns Bridge and the Burlington Northern Santa Fe railroad bridge, and include riverfront. The earliest available aerial photograph of this property is from 1936, and much of the southern portion of the property is wetland and an inlet of the Willamette River. Over 30 tanks are present on the northern portion and western property. Two large tanks are present on what appears to be a filled area of land adjacent to the Willamette River forming a partial island for the tanks. Additional filling occurred through 1944 on the southern portion of the property, and additional infrastructure was constructed, including tanks. By the late 1990s and into the 2000s, significant infrastructure was removed from the property. Based on data provided by the City, PSU, and satellite imagery, there are currently eight tanks present (Cone 2020 and Dusicka 2019).

1.3.4 Area 4 – Willbridge

Area 4 includes 11 properties owned by Kinder Morgan (5800 and 6080 NW St Helens Road), Chevron (5533 NW Doane Avenue), Conoco Phillips (5528 Doane Avenue), Zenith Energy Terminals (5501 NW Front Avenue), and McCall Oil (5700 and 5480 NW Front Avenue). All 11 properties are located south of the Burlington Northern Santa Fe railroad bridge and includes some riverfront properties.

1.3.4.1 Kinder Morgan South

Kinder Morgan South includes three properties. One property is located on the east side of NW St Helens Road, along the Willamette River with no tank infrastructure. The other two properties with tanks are located on the west side of NW St Helens Road and do not include riverfront. The earliest aerial photograph from 1923 depicts limited tank infrastructure constructed on the southern property. By 1936 the northern property remained vacant, undeveloped land and the southern property has been developed with approximately 15 tanks. Additional tanks were added to the southern property by 1944, and additional roads were constructed around the northern and southern properties. By 1956, approximately 20 tanks had been constructed on the northern property. Infrastructure continued to be added or removed over the next 50 years. Based on data provided by the City, PSU, and satellite imagery, there are currently 134 tanks present (Cone 2020 and Dusicka 2019).

1.3.4.2 Chevron

Chevron includes two properties. One property is located on the east side of NW St Helens Road along the Willamette River and appears to have one tank which was installed between 1944 and 1956. The larger property with the majority of the tank infrastructure is located on the west side of NW St Helens Road and does not include riverfront. Minor development of the property was visible in the earliest available aerial photograph from 1923. Major development of this property continued through 1936, when 12 tanks were visible on the property. Significant development of the property continued through the early 1960s, with larger tanks constructed on the eastern portion of the property and smaller volume tanks constructed on the west portion of the property. Based on data provided by the City, PSU, and satellite imagery, there are currently 146 tanks present (Cone 2020 and Dusicka 2019).

1.3.4.3 Conoco Phillips

Conoco Phillips includes two properties. One property is located on the east side of NW St Helens Road along the Willamette River and does not have any tank infrastructure based on satellite imagery. The larger property located on the west side of NW St. Helens Road was first developed prior to 1936. Approximately 20 tanks are visible on the westernmost portion of the property in 1936. The remaining property appears undeveloped, with a small water body noted east of the existing tanks. By 1944, the water body and been filled, and new tank infrastructure was installed to the east and south. By 1970, the majority of the tank infrastructure had been constructed on the site. Based on available records, the tanks all appear to be the original structures. Based on data provided by the City, PSU, and satellite imagery, there are currently 93 tanks present (Cone 2020 and Dusicka 2019). Zenith Energy Terminals.

Zenith Energy Terminals (formerly Arc Logistics) includes two properties. Both properties are located on the west side of NW Front Avenue and share a property line with Conoco Phillips. The smaller of the two properties, which is approximately 3 acres, was undeveloped until at least 1944 when buildings were constructed on the property. By 1964, one tank was constructed on the western portion of the property. A second tank was constructed by 1980, and all preexisting buildings had been removed. The larger property was first developed as housing in the early 1940s. Limited tank infrastructure development was present by 1948, on the northwest corner of the property, adjacent to the housing. By 1959, the housing had been removed, and additional tanks were constructed. Between 1964 and 1968, the former housing area had been filled and graded for additional tank infrastructure, which continued to expand through the mid-1980s. Based on data provided by the City, PSU, and satellite imagery, there are currently 97 tanks present (Cone 2020 and Dusicka 2019).

1.3.4.4 McCall Oil

McCall Oil includes two properties, both located on the east side of NW St. Helens Road, along the shore of the Willamette River. Both properties were part of the Willamette River prior to 1968. Significant filling of the site and surrounding properties continued through the 1980s. The earliest available aerial photograph of the area shows the present-day tank infrastructure had been constructed by 1986. Based on data provided by the City, PSU, and satellite imagery, there are currently 26 tanks present (Cone 2020 and Dusicka 2019).

1.3.4.5 Zenith Energy

Zenith Energy includes two properties, both located on the west side of NW St Helens Road and are not located on the riverfront. Development of the larger property to the south was noted in the 1956 aerial photograph, and one of the two tanks on the smaller property to the north was noted in the 1964 aerial photograph. By 1990, all tanks currently present were visible on the aerial photographs. Tank decommissioning's appeared as early as the 1998 aerial photograph. Based on data provided by satellite imagery and Portland Fire & Rescue (PF&R 2021), there are currently 86 tanks present.

1.3.5 Area 5 – Equilon

Area 5 includes one property owned by Equilon. The property is located on the west side of NW St Helens Road. The earliest available aerial photograph indicates that tank infrastructure was present prior to 1936 on the southeast portion of the property. Three additional tanks were constructed on the northwest portion of the property between 1944 and 1956, and a fourth tank was added in the 1990s. Based on data provided by satellite imagery, there are currently 14 tanks present.

2.0 DATA REVIEW

As part of this evaluation, we reviewed multiple technical documents, including construction reports, geotechnical reports, previous studies of the CEI Hub, and previous studies of the CSZ expected earthquake. Our document review included both publicly available data and confidential data necessary for the completion of this evaluation. Publicly available data included updated data from DOGAMI, the City, Oregon Solutions, PSU, and private contractors who have completed work at the CEI Hub. Confidential data were provided by the Oregon Office of State Fire Marshal (OSFM) in the form of a data table (Appendix A). Confidential data will be removed from the report prior to publishing. Detailed review included review of boring logs, permit applications, aerial photographs, and detailed infrastructure data provided by both OSFM and the City.

A detailed bibliography of the resource documents reviewed is provided in Table 2.1 (attached). Specific properties for which documents were reviewed as part of the geologic risk evaluation in *Section 4.0 Geologic Risk of the CEI Hub in a CSZ Earthquake* are highlighted on Figure 1.5 through Figure 1.9.

Using the technical documents provided by the City and other sources, a detailed analysis of the geologic risk to the CEI Hub in a CSZ earthquake was conducted. This included the use of local boring logs as well as the updated DOGAMI data to evaluate the ground shaking, liquefaction, and lateral displacement expected at the CEI Hub during a CSZ earthquake. Details of this evaluation are provided in *Section 4.0 Geologic Risk of the CEI Hub in a CSZ Earthquake*.

2.1 Tank Data Collection and Review

During the initial data gathering process, it became clear that the data available from the OSFM would likely not include all data necessary to construct a complete inventory of tanks and supporting infrastructure at the CEI Hub. A critical part of this evaluation was to include an inventory of the tanks and supporting infrastructure at the CEI Hub, which would later be used to evaluate the impacts of a

CSZ earthquake on the CEI Hub. Data necessary to do this would include exact location of tanks and supporting infrastructure and the age of the tanks and supporting infrastructure. During a phone call with Mark Johnston, Assistant Chief Deputy at OSFM, (Johnston 2020), Mr. Johnston indicated that tank owners are not required to report the exact location of the tanks, rather, only the quadrant of the property in which the stored material is located is required. Additionally, OSFM does not keep records of supporting infrastructure, and tank owners are not required to report the age of the tanks. Mr. Johnston indicated that the information on tank age would likely need to be requested directly from the property owners; however, he expects doing so would involve a lengthy legal process. Publicly available data collected regarding the infrastructure at the CEI Hub are provided in *Section 3.0 Tanks and Infrastructure of the CEI Hub*.

Another key aspect of the data collection was to include the contents of each tank at the CEI. As discussed with Mr. Johnston, property owners are only required to report the amount of hazardous substances on their property once a year, and that report only needs to include the maximum daily amount at any given point during the year. Therefore, the OSFM data were supplemented with data compiled by the City and PSU (see discussion below). Data collected regarding the contents of the tanks at CEI hub are provided in *Section 3.0 Tanks and Infrastructure of the CEI Hub*.

3.0 TANKS AND INFRASTRUCTURE OF THE CEI HUB

Salus received two main datasets regarding the tanks present at the CEI Hub, both of which were incomplete. The first dataset was provided by the City in the form of a web map (Cone 2020) and feature layer (Appendix A). The web map and feature layer were created from data collected during the PSU study of the CEI Hub (Dusicka 2019). This feature layer was compared to available satellite photographs of the CEI Hub to obtain an inventory of the number of tanks present in each area and each property. Approximately 122 tanks observed during a review of satellite imagery were not included in the web map; therefore, we had no information on tanks or their contents. The majority of these 122 tanks observed in satellite imagery coincide to tanks located at Zenith Energy and Equilon, which are not listed in the COP dataset feature layer. Table 3.1 (attached) provides an abridged summary of the data provided in the feature layer and the additional tanks at Zenith Energy (107 tanks),Equilon (14 tanks), and NW Natural (1 tank)identified from satellite photographs.

The second dataset was a confidential data table provided from the OSFM's office (Appendix A). This dataset was obtained through a Freedom of Information Act (FOIA) request submitted by John Wasiutynski from the City on behalf of Salus. The data received from the OSFM are data collected by the OSFM as part of the Community Right to Know (CR2K) program. The OSFM maintains the records associated with the Oregon Community Right to Know and Protection Act of 1985 (ORS 453.307-414), which requires Oregon employers to report their hazardous substances to OSFM, including where they are stored and the hazards associated with them (OSP 2021). Employers reporting hazardous substances are required to follow specific survey instructions but are only required to report substances once per calendar year, or if a substantive change occurs (OSFM 2020).

Following receipt of the OSFM data, Salus compared the dataset to that previously received from the City. Limited redundancies were noted that allowed for merging of the data. In a follow-up conversation with OSFM, it was noted that employers are only required to report the maximum daily amount of any substance present at their entire property and the general quadrant of their property it is stored at (Johnston 2020). For example, a property may have four above ground storage tanks (ASTs) that each hold 25 gallons of gasoline, four ASTs that each hold 20 gallons of diesel, and four ASTs that each hold 10 gallons of oil. This property will report 100 gallons of gasoline, 80 gallons of diesel, and 40 gallons of oil during their yearly submittal to OSFM. Due to the amalgamation of substances in the OSFM records, this dataset is not useful for identification of contents of individual tanks. The confidential dataset is provided in Appendix A.

Additional information was collected from City (Portland Fire & Rescue) resources and permit applications to cover the Zenith, Equilon, and NW Natural properties. This information was compared with the above data sets and incorporated into our tank database.

In addition to the inventory of tanks present at the CEI Hub, Salus made efforts to create an inventory of supporting infrastructure present at the CEI Hub. No existing datasets were found inventorying supporting infrastructure; therefore, Salus relied on satellite imagery, the City web map, and Portland Maps to identify buildings present at the CEI Hub (Portland Maps 2020). A summary of this inventory is provided in Table 3.2 (attached).

4.0 GEOLOGIC RISK OF THE CEI HUB IN A CSZ EARTHQUAKE

This section presents estimates of site and soil behavior of the CEI Hub areas during a magnitude 9.0 CSZ earthquake. Estimates for the level of ground motion shaking were evaluated, the soil at each of the areas was characterized based on the existing data provided by the City, and estimates of liquefaction settlement and lateral spread were developed for each location.

4.1 CSZ Earthquake Ground Motion Shaking Intensity

Since the publication of the 2017 DOGAMI report, several additional resources have been published that can estimate the intensity of the ground motion shaking in the project areas. The resources are in the form of ground motion models published as a part of the Next Generation Attenuation-Subduction (NGA-Subduction) (Bozorgnia and Stewart 2020) research effort and simulations published in Frankel et al. (2018). The ground motion models are developed from recordings and simulations of subduction zone events around the world and developed for compatibility with probabilistic assessments of ground motion shaking, such as those used in building design and, as such, include model features to address uncertainty. The simulations represent the synthetic modeled ground surface response of 30 magnitude 9.0 events occurring in the CSZ using a large-scale numerical model of the Pacific Northwest.

The shaking of a site at the ground surface is influenced by the stiffness of the surface soil. Softer soil will typically amplify ground motion shaking more than stiff soils. While the DOGAMI report includes these soil effects and the NGA-Subduction ground motion models (GMMs) can account for these effects, the Frankel et al. (2018) simulation dataset does not. For a more direct comparison, the two new data sources (the NGA-Subduction and Frankel et al. 2018 simulations) are evaluated in the following sections for a hard soil or rock-like site condition so a consistent basis of comparison between the models can be used. Where ground motion intensity values in this study are evaluated at the ground surface, the site classes and factors commonly used in the National Earthquake Hazards Reduction Program (NEHRP) are used to adjust the earthquake intensity hard-soil and rock condition to a surface condition in order to reflect the soft site soils. The NEHRP site factors are a simplified intensity-dependent ratio of ground motion intensity between stiff and soft sites, and they are widely adopted in design standards, such as the Oregon Structural Specialty Code, International Building Code, and American Association of State and Highway and Transportation Officials seismic design standards.

4.1.1 NGA-Subduction Ground Motion Models

The NGA-Subduction project is one of a series of research projects created to facilitate the development of ground motion models for use in seismic hazard assessments. Previous NGA projects were done for shallow crustal earthquakes (NGA-West1 and NGA-West2) and for stable continental regions (NGA-East) and the resulting models are widely used in the International Building Code (IBC) and in other design and research applications. The NGA-Subduction project is focused on the development of ground motion models for subduction zones and results from this project are in the process of being published.

Two ground motion models have been produced from the NGA-Subduction project, the Kuehn et al. (2020) model (KBCG20), and the Parker et al. (2020) model (PSHAB20). These models use information about a specified earthquake scenario to estimate the intensity of ground shaking at a site. Typical inputs for these models include the earthquake magnitude, rupture distance from the site to the epicenter, site soil stiffness, and depth to the rupture. Because of the variability and uncertainty of the ground motion shaking for a specified earthquake scenario, the models are used to develop percentiles of the ground motion intensity response. For example, for a given earthquake scenario, the ground motion models are commonly used to estimate a median, 50th percentile ground motion intensity response, in which half of the modeled ground motions values are greater than and half less than the median response. Instead of only evaluating the median (50th percentile) ground motion, it is standard practice to also consider the 84th percentile intensity response, which represents the median response plus a standard deviation (or “sigma”) of the response values.

Ground motion models, such as the KBCG20 and PSHAB20, which consider the effects of uncertainty on the level of ground motion shaking are commonly adapted for use in seismic hazard assessments that depend on the likelihood of a certain level of ground motion shaking occurring, such as in the seismic design of new buildings.

4.1.2 Frankel et al. (2019) Simulations

A series of simulations of ruptures of the CSZ interface were conducted and published in Frankel et al. (2019). Thirty ruptures of magnitude 9.0 and greater of the CSZ were modeled for a variety of rupture parameters and locations along the CSZ interface zone. One of the products of these simulations are synthetic ground motion recordings at locations throughout the Pacific Northwest. The synthetic seismograms are representative of individual earthquake events and are not comprehensive or representative of the full range of uncertainty of ground motions due to a CSZ interface event.

For this study, the ground motions were selected for the model grid point nearest 45.57 degrees N, -122.76 degrees E, the closest model grid point to the project study area. The synthetic ground motions are two-component (north-south and east-west) synthetic acceleration time histories for a stiff soil condition. The soil condition used at the ground surface in the Frankel et al. (2019) model is a site with time averaged shear wave velocity in the top 100 feet (30 meters) of approximately 2,000 feet per second (600 meters per second). Figure 4.1 below shows the response spectrum for the 60 acceleration time series selected from the Frankel et al. (2019) model in blue with the median in red.

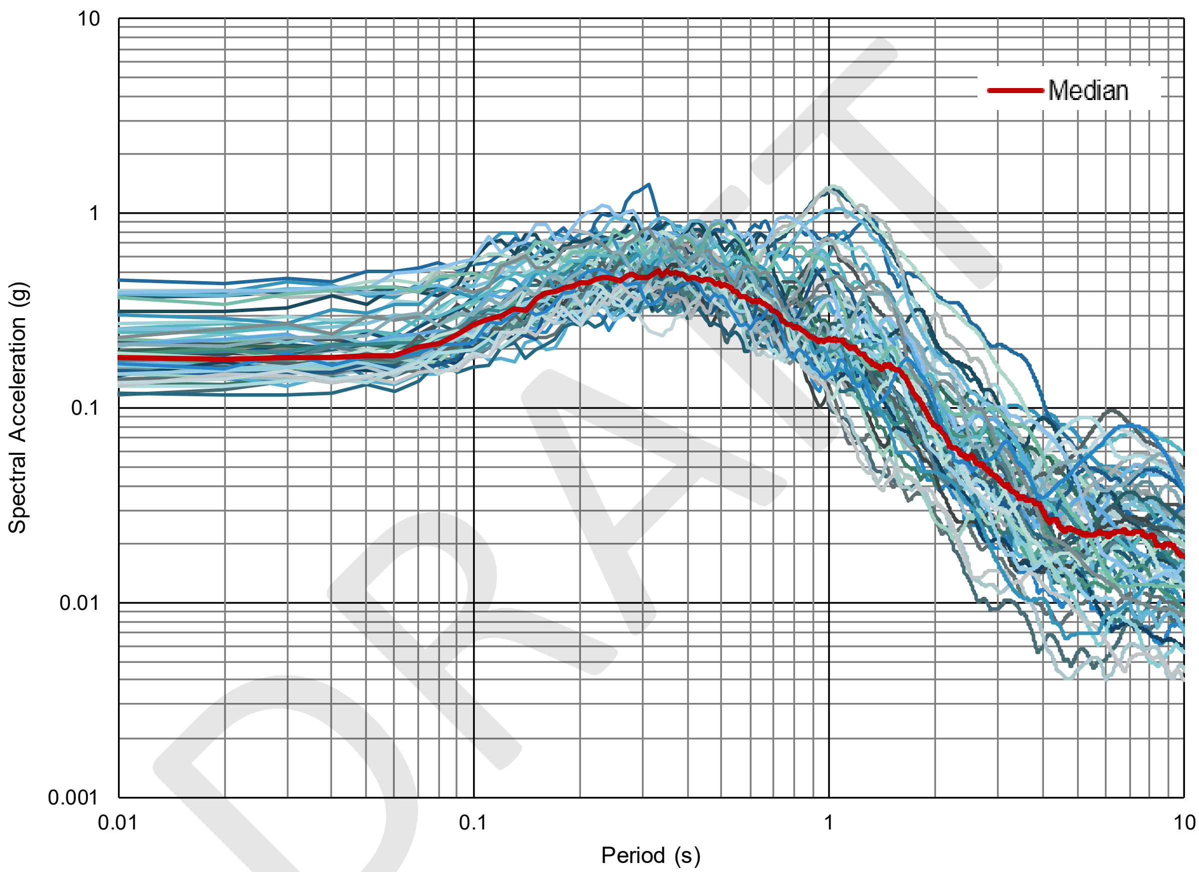

Figure 4.1 CSZ Ground Motion Components (Frankel et al. 2019)

4.1.3 Ground Motion Intensity Comparison

This study evaluates the level of shaking at the project sites of interest for a magnitude 9.0 rupture of the CSZ. This is commonly referred to as a "deterministic" event; the computed level of ground motion shaking is computed for a specific event and the likelihood of that event occurring is not considered. In analyses where the likelihood of a seismic event occurring is considered, the seismic assessment is referred to as "probabilistic." Structures designed using the IBC are typically designed considering the lesser of an 84th percentile deterministic event and a probabilistic hazard assessment for a probability of exceedance of 2 percent in 50 years.

In engineering design, the ground motions due to seismic shaking are commonly transformed to a spectral acceleration response spectrum that can be used to model how an earthquake is experienced by a building/structure, Spectral acceleration values for the available calculation methods are shown on Figure 4.2 below for a stiff soil or rock-like Site Class B/C condition. The black line represents a probabilistic geometric mean spectrum from the 2014 USGS hazard maps commonly used in IBC design for new construction. This probabilistic curve includes the effects of both subduction events and shallow crustal events, represents the hazard of a 2 percent probability of exceedance in 50 years (equivalent to a 2,475-year return period), and is shown for comparison only. The red and blue lines are computed from the PSHAB20 and KBCG20 GMMs, respectively, with the solid lines representing the median and dashed lines representing the 84th-percentile ground motion (median plus one standard deviation, sigma). The PSHAB20 and KBCG20 GMMs were computed using the earthquake characteristics shown in Table 4.1 below. The green line is the median of the Frankel et al. (2019) simulations. The gray points are the surface intensity values from the DOGAMI (2018) report decreased by a factor of 1.2 to remove the effects of soft soil amplification and approximate a stiff soil or rock-like condition similar to the condition used for the other lines plotted on the figure. The 1.2 factor is consistent with the NEHRP amplification ratio between the site class used in the DOGAMI (2018) map near the project site (Class D, representative of the surface soil condition) and the site class used in this study for the Frankel et al. (2018) simulations and NGA-Sub GMMs (Class C, representative of a stiff soil or soft rock condition).

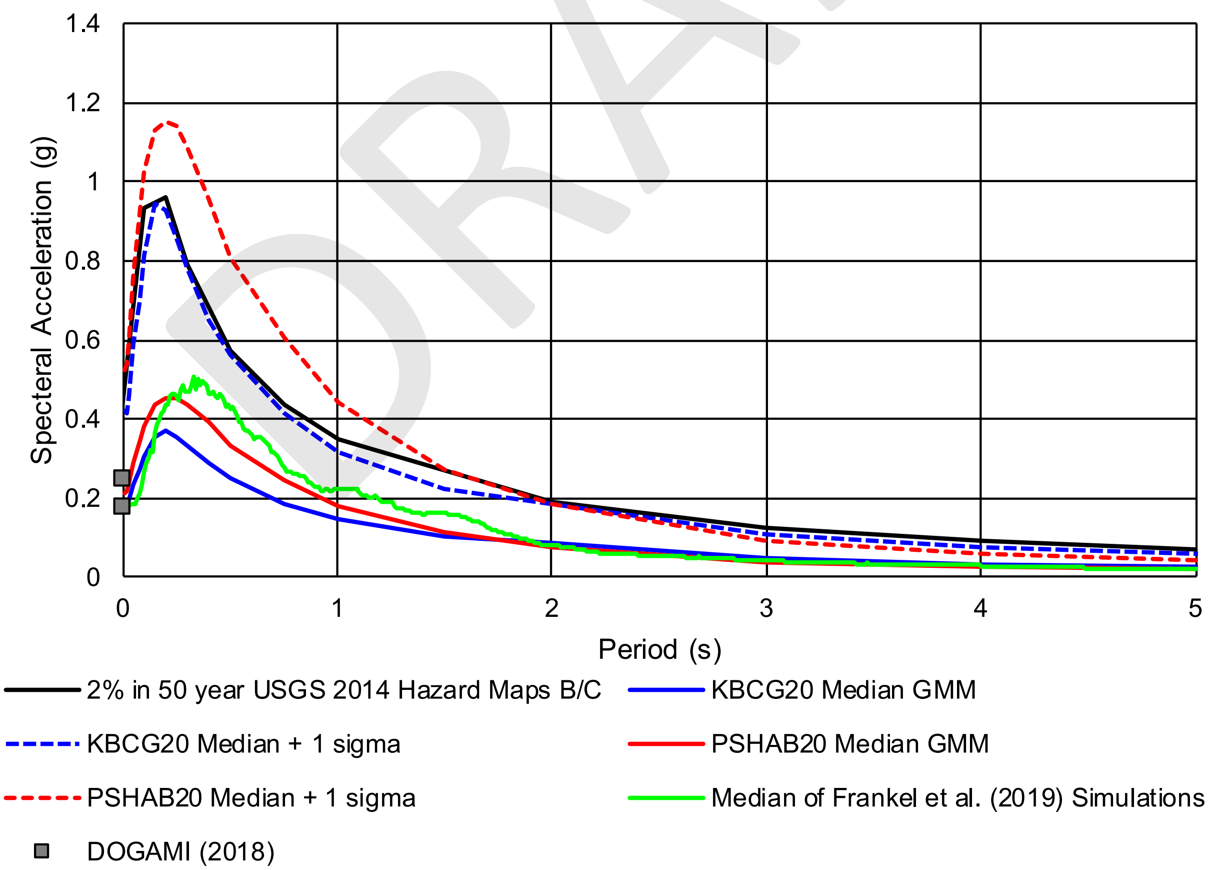

Figure 4.2 CSZ Spectral Response

Table 4.1 - KBCG20 and PSHAB20 Earthquake Parameters

Parameter	Value
Region and Type	Cascadia Interface
Moment Magnitude	9.0
V_{S30}	760 meters per second
Rupture Distance	72 kilometers
Rupture Depth	10 kilometers

Figure 4.2 indicates that the level of ground motion shaking shown in the DOGAMI hazard maps is similar to the intensity estimated from the most recent ground motion models for a CSZ rupture. However, the uncertainty range of the GMMs indicates that an 84th percentile event represents a significantly higher level of ground motion shaking than the median earthquake event; specifically, the peak ground acceleration (equivalent to the spectral response at a period of zero seconds) is approximately 100 percent higher for the 84th percentile event than the median event.

The median of the Frankel et al. (2019) simulations have a similar PGA as both the DOGAMI hazard maps and the KBCG20 and PSHAB20 ground motion models. The PGA values of the simulated ground motions range from 0.12 to 0.45. The simulations represent a range of rupture scenarios for the CSZ, not just a worst-case scenario. The similarity of the simulations to the other estimates of ground motion shaking indicate that the site is susceptible to strong shaking from interface CSZ events anywhere along the fault.

The scope of the liquefaction and lateral spread analyses presented later in this section only considered the median event at the ground surface and not also the 84th percentile event (sigma event).

4.2 Representative Soil Information and Liquefaction Analysis

Available geotechnical subsurface soil information was collected for the areas of interest of this study. This section presents the generalized subsurface conditions of the soil at each of the locations. The characterization of the soil at these sites is representative only and not intended to replace a more detailed geotechnical design study at each location, and the values provided in this report are not intended for use in geotechnical design. The references for the geotechnical reports and other subsurface information cited in this section can be found in the attached Table 2.1.

Key information from the geotechnical reports used to characterize subsurface conditions at the sites primarily included logs of mechanically drilled borings and cone penetration test (CPT) soundings. From the borings, we evaluated standard penetration test (SPT) blow count (N_{SPT}) data, which is a standardized soil sampling method used throughout the geotechnical industry. The CPT soundings include advancing a steel probe equipped with electronic instrumentation to measure resistance, friction, and other soil parameters. Equivalent N_{SPT} values can be obtained from CPT soundings to help compare data to the drilled borings.

The soil at each area was generalized into stratigraphic units that were evaluated for their potential for immediate liquefaction settlement, an approximate upper and lower bound of N_{SPT} values and a representative fines content in the soil layer. The upper and lower bound N_{SPT} values are used to provide a range of anticipated liquefaction settlement at each site, lower N_{SPT} values indicate larger amounts of potential surface settlement during an earthquake. For the lateral spread analyses, only the lower-bound N_{SPT} profile was used.

The subsurface soil information in this section considers fine-grained soils as generally "non-liquefiable" as the focus of this study is on immediate ground surface settlements that will occur following an earthquake event. While fine-grained soils, such as silt and clay, may experience strength loss during an earthquake that results in failure of foundations and structures at the ground surface, these soils generally contribute less to ground surface settlement than coarse-grained sand and gravel. A detailed design study for each of the project areas, including further review of soil laboratory testing data may be required to characterize the likelihood of strength loss in the fine-grained soil deposits.

4.2.1 Area 1 – Kinder Morgan North

Geotechnical soil information for Area 1 is documented in a report by GeoEngineers (2011). The soil at the site generally consists of a dense layer of gravel and coarse-grained fill over layers of layers of silt and clay that appear to be generally non-liquefiable. Approximately 38 to 40 feet below the ground surface (bgs) is a unit of potentially liquefiable coarse-grained sandy silt and silt with sand that may have beds of fine-grained clayey silt and silty clay. The groundwater table appears to be approximately 4 feet bgs. The representative stratigraphy of the area is shown in Table 4.2 (below).

The range of N_{SPT} values for the stratigraphic units are equivalent corrected blow counts from CPT soundings in the area as provided in the GeoEngineers (2011) report.

Table 4.2 - Kinder Morgan North Area Soil Stratigraphy

Stratigraphic Unit	Potentially Liquefiable	Upper Bound N_{SPT} (blows/foot)	Lower Bound N_{SPT} (blows/foot)	Fines Content (percent)	Thickness (feet)
Gravel and Silty Sand Fill	Yes	50	50	10	4
Clayey Silt to Silty Clay	No	11-22	5	60	34
Silty Sand	Yes	27	16	50	6
Clayey Silt to Silty Clay	No	50	13	60	2
Sand with Silt	Yes	35	21	40	4
Basalt	Top of bedrock encountered at approximately 50 feet below ground surface				

4.2.2 Area 2 – Linnton

The Linnton Area has the most available subsurface information of the areas reviewed in this study. Therefore, there was enough information for Area 2 information to characterize the northern and southern parcels separately.

4.2.2.1 North Area 2 – Linnton

Geotechnical soil information for the north region of Area 2 is documented in a series of reports from URS Corporation (2006, 2007a, 2007b), Professional Service Industries, Inc. (PSI) (2015) and Hart Crowser (1992). The stratigraphy generally consists of liquefiable coarse-grained fill and stream deposits overlying a layer of non-liquefiable fine-grained deposits, which overlies a deeper layer of liquefiable coarse-grained alluvial deposits. The ordinary high-water elevation was considered the top of the groundwater table at this site and is approximately 14 feet bgs. The representative stratigraphy of the area is shown in Table 4.3.

Table 4.3 - Linnton Northern Area Soil Stratigraphy

Stratigraphic Unit	Potentially Liquefiable	Upper Bound N_{SPT}	Lower Bound N_{SPT}	Fines Content (percent)	Thickness (feet)
Sandy Fill with Silt	Yes	22	8	10	20
Coarse-Grained Stream Deposits	Yes	N/A	10	10	0-10
Fine-Grained Alluvial Deposits	No	20	12	70	10-20
Sandy Alluvial Deposits	Yes	22	14	10	30
Basalt	Top of bedrock encountered at approximately 70 feet below ground surface				

In the series of URS reports the average N_{SPT} values for each of the stratigraphic units is reported and plotted with all available N_{SPT} measurements. The upper- and lower-bound N_{SPT} values were selected to represent reasonable upper and lower bounds of the available N_{SPT} data. These values are generally consistent with the noted subsurface information in the PSI and Hart Crowser reports.

The liquefiable coarse-grained stream deposits do not appear to be present throughout the site. However, because these soils represent a significant contribution to the potential for liquefaction settlement and lateral spread in the area of the site, they were considered to be 10 feet thick in the analysis of the lower-bound N_{SPT} values only and not in the upper-bound N_{SPT} value analysis.

4.2.2.2 South Area 2 – Linnton

Geotechnical soil information for the southern region of Area 2 is documented in a series of reports and technical memoranda by CH2MHILL (2006a, b, c, and d) and a report by Dames and Moore (1981). The soil generally consists of coarse-grained liquefiable gravel fill and silty sand overlying non-liquefiable fine-grained silt and clay. The groundwater table is indicated to be at approximately 18 feet bgs.

In the CH2MHILL reports, N_{SPT} values of the stratigraphic units are reported as a range. The upper and lower N_{SPT} values are taken as the middle of the range plus and minus 25 percent of the range.

Table 4.4 - Linnton Southern Area Soil Stratigraphy

Stratigraphic Unit	Potentially Liquefiable	Upper Bound N_{SPT}	Lower Bound N_{SPT}	Fines Content (percent)	Thickness (feet)
Gravel Fill	Yes	17	7	5	10
Silty Sand	Yes	9	5	45	20
Silt and Clay	No	20	9	75	35
Basalt	Top of bedrock encountered at approximately 65 feet below ground surface				

4.2.3 Area 3 – NW Natural

The subsurface soil information of Area 3 is characterized in a series of geotechnical reports by GeoEngineers (2005, 2012, 2015, 2016. 2018). Soil in this area generally consists of a unit of liquefiable coarse-grained sandy silt and fill over a thicker layer of non-liquefiable fine-grained alluvial silt. The groundwater table appears to be approximately 10 feet bgs from soil borings at the site. Soil stratigraphy information is provided in Table 4.5.

The N_{SPT} values for each of the stratigraphic units were approximated as the average of N_{SPT} values from the stratigraphic units as measured in four soil borings at the site plus and minus one half of the standard deviation.

Table 4.5 - NW Natural Northern Area Soil Stratigraphy

Stratigraphic Unit	Potentially Liquefiable	Upper Bound N_{SPT}	Lower Bound N_{SPT}	Fines Content (percent)	Thickness (feet)
Sandy Silt and Poorly Graded Sand Fill	Yes	17	7	10	20
Fine-Grained Alluvial Silt	No	8	5	80	55
Basalt	Top of bedrock encountered at approximately 80 feet below ground surface				

4.2.4 Area 4 – Willbridge

The subsurface soil information of Area 4 is characterized in reports by GeoEngineers (1998, 2000a, 2000b), PSI (2015), AMEC Earth and Environmental (2004), URS Corporation (2001) and the City of Portland (1968). However, much of the soil information in these reports only extends to depths of 20 to 40 feet bgs and does not extend to the top of the basalt bedrock. The GeoEngineers (1998) and PSI (2015) reports were the reports most significantly used to develop the generalized stratigraphy profile in Table 4.6 for Area 4.

The stratigraphy in Area 4 generally consists of liquefiable sandy fill and loose sand overlying a layer of fine-grained non-liquefiable stiff silt. Below the silt is a layer of liquefiable loose sand deposits. The groundwater table appears to be approximately 10 feet bgs.

Upper and lower bounds for the N_{SPT} values were computed from soil borings in the GeoEngineers (1998) and PSI (2015) reports that extended to the basalt. The N_{SPT} values were approximated as the average of N_{SPT} values from the stratigraphic units as measured in three soil borings at the site plus and minus one half of the standard deviation. The N_{SPT} values from this subset of the soil information available for the site are generally representative of the soil conditions documented in the other subsurface information reports.

Table 4.6 - Willbridge Area Soil Stratigraphy

Stratigraphic Unit	Potentially Liquefiable	Upper Bound N_{SPT}	Lower Bound N_{SPT}	Fines Content (percent)	Thickness (feet)
Sandy Fill and Loose Sand	Yes	19	9	5	25
Stiff Silt	No	9	9	75	15
Loose Sand	Yes	8	8	5	10
Basalt	Top of bedrock encountered at approximately 50 feet below ground surface				

4.2.5 Area 5 – Equilon

The subsurface soil information of Area 5 is characterized in reports by GeoDesign Inc. (2006), Rittenhouse-Zeman and Associates, Inc. (1990) and Shannon and Wilson, Inc. (1965). The soil at the site generally consists of a layer of liquefiable loose sand and sandy fill over a layer of stiff silt overlying a layer of liquefiable loose sand. The groundwater table appears to be at a depth of approximately 10 feet bgs. The stratigraphy information for Area 5 is shown in Table 4.7 (below).

Area 5 has generally lower N_{SPT} values for similar stratigraphic units than the other areas. The deep layer of loose sand did not have any N_{SPT} values at this location and so the N_{SPT} values of Area 4 were assumed. The Upper and Lower bound N_{SPT} Values in table 4.7 represent the range of N_{SPT} values measured in each stratigraphic layer. However, because there is so little variability in these values relative to the mean, the standard deviation of N_{SPT} was not considered for this site as it was for Areas 3 and 4.

Table 4.7 - Equilon Area Soil Stratigraphy

Stratigraphic Unit	Potentially Liquefiable	Upper Bound N_{SPT}	Lower Bound N_{SPT}	Fines Content (percent)	Thickness (feet)
Sandy Fill and Loose Sand	Yes	7	4	5	25
Stiff Silt	No	6	4	75	20
Loose Sand	Yes	8	8	10	10
Basalt	Top of bedrock encountered at approximately 50 feet below ground surface				

4.3 Surface Settlement Due to Liquefaction of Coarse-Grained Soil

Each of the characteristic soil profiles in the five areas were evaluated for estimated surface settlement due to liquefaction. The simplified Idriss and Boulanger (2008) procedure for estimating liquefaction effects during an earthquake was used. This calculation method uses the soil information provided in Tables 4.2 through 4.7 above and parameters for a characteristic earthquake. The earthquake used in this analysis was a magnitude 9.0 earthquake with a ground surface PGA of 0.3 g, which is approximately equal to the median surface response of a deterministic event as discussed in *Section 4.1 CSZ Earthquake Ground Motion Shaking Intensity*. The estimated surface settlement at each area is shown in Table 4.8.

Table 4.8 - Estimated Surface Settlement due to Liquefaction

Area	Estimated Settlement (inches)	
	Upper Bound N_{SPT} Profile	Lower Bound N_{SPT} Profile
Area 1 – Kinder Morgan North	0	2
Area 2 – Linnton North	8	19
Area 2 – Linnton South	7	8
Area 3 – NW Natural	3	9
Area 4 – Willbridge	9	14
Area 5 – Equilon	15	17

Additional estimates of surface settlement are included for some of the areas in the geotechnical reports reviewed in this study. These surface estimates are generally not evaluated for a deterministic CSZ event and use a probabilistic earthquake hazard level. A summary of the available estimates of surface settlement from these reports is in Table 4.9 below. The estimates in Area 2 North and Area 4 are based on shallow exploration data and do not consider settlement of the soil from the ground surface to the bedrock, including the deep liquefiable sand layer observed in some of the areas. The more detailed estimate of surface settlement for Area 2 South in the CH2MHILL (2006) report computed with the Ishihara and Yoshimine (1992) simplified method generally agrees with the estimate from this study in Table 4.8.

Table 4.9 - Reported Surface Settlement in Reviewed Historical Reports

Area	Reported Surface Settlement	Report	Method
Area 2 – Linnton North	1.5 to 1.75 inches	PSI (2015)	CPT
Area 2 – Linnton South	6 to 9 inches	CH2MHILL (2006)	Ishihara and Yoshimine (1992)
Area 4 – Willbridge	3 to 4.25 inches	GeoEngineers (1998)	CPT

4.4 Lateral Spread Potential

The estimated lateral spread at each site was evaluated for the five areas using the Youd, Hansen, and Bartlett (2002) simplified procedure. The Youd, Hansen, and Bartlett (2002) procedure estimates the amount of horizontal movement at a location on a slope or some distance away from a free-standing soil face due to earthquake-induced liquefaction of coarse-grained soil.

The inputs to the Youd, Hansen, and Bartlett (2002) simplified procedure include earthquake magnitude and distance, the cumulative thickness of liquefiable soil units at the site, the average mean grain size of the granular layers (D_{50}), the average fines content of the granular layers, and information about the geometry of the slope. The Youd, Hansen, and Bartlett (2002) procedure is limited to earthquake magnitudes 6 to 8, and a magnitude 8 earthquake was considered for this study. If the procedure is extrapolated to a magnitude 9 earthquake, the estimated lateral spread increases by a factor of 7. The earthquake distance used was 70 kilometers and is consistent with the deterministic seismic hazard analyses discussed in *Section 4.1 CSZ Earthquake Ground Motion Shaking Intensity*. The thickness of the liquefiable soil layers and fines content of the soil layers used in this analysis is consistent with the stratigraphy profiles given in *Section 4.2 Representative Soil Information*. A single representative D_{50} of 0.25 millimeters for all granular soil was estimated from the laboratory testing results provided in the historical subsurface information documents discussed in *Section 4.2 Representative Soil Information*. The range of the D_{50} for both the shallow and deep granular materials was fairly consistent and ranged from 0.1 to 0.7 millimeters.

The Youd, Hansen, and Bartlett (2002) correlations depend on the geometry of the site investigated and consider either a sloping ground condition or a free-face condition. For this study, we evaluated the surface profile at each area on the cross-section lines shown on Figures 1.5 to 1.9 using LiDAR data (DOGAMI 2014) for upland topography and bathymetry data (2005) for offshore slopes. Generally, the areas at each of the sites where tanks are located are flat and has little to no slope. However, along the Willamette River, there is a consistent elevation change from the ground surface down to the edge of the river. Under the surface of the river, the slope of this elevation change generally becomes more gradual and the submerged slope ends at approximately the same elevation as the basalt encountered in the reviewed borings. In this preliminary study, we considered the elevation change from the upland ground surface to the approximate bottom of the submerged slope as a free-face soil condition that ranged from 50 to 70 feet tall for most locations. For Area 3, we considered the height of the free face only to include the surficial liquefiable sand as the free face condition that has a height of 20 feet. Horizontal lateral spread displacement estimates are provided in Table 4.10 below as a function of distance from the soil free-face.

Table 4.10 - Estimated Lateral Spread at Each Area Varied by Distance to Free Face

Area	Estimated Lateral Spread (feet) Distance to Free Face of Soil				
	50 Feet	100 Feet	250 Feet	500 Feet	1000 Feet
Area 1 – Kinder Morgan North	8	5	3	2	1
Area 2 – Linnton North	20	13	8	5	3
Area 2 – Linnton South	13	9	5	3	2
Area 3 – NW Natural	6	4	2	2	1
Area 4 – Willbridge	14	9	5	4	2
Area 5 – Equilon	15	10	6	4	2

Geotechnical reports for locations in some of the areas reviewed for this study included estimates of lateral spread as shown in Table 4.11. As with the liquefaction settlement analyses discussed in Section 4.3, these reports evaluate the lateral spread potential for a probabilistic design condition and not a deterministic condition representative of a magnitude 9 subduction event. The CPT analyses in PSI (2015) and Geoengineers (1998) do not consider surface geometry, are of limited depth, and are simplified procedures similar to the Youd, Hansen, and Bartlett (2002) analysis conducted for this study.

The CH2MHILL (2006) analysis was a 2-dimensional finite difference model run with the software FLAC for the edge slope of the soil along the Willamette River, the same slope considered a free-face in this study. The FLAC analyses were conducted with detailed soil models for a series of earthquake time histories to model the behavior of the slope during an earthquake. While there have been several advancements in numerical modeling and understanding subduction zone earthquake hazards in the Portland area, the analyses conducted in the CH2MHILL report are generally representative of detailed, high-quality analyses and result in a similar maximum displacement as estimated with the Youd, Hansen, and Bartlett (2002) analysis above.

Table 4.11 - Reported Surface Settlement in Reviewed Historical Reports

Area	Reported Lateral Spread	Report	Method
Area 2 – Linnton North	1.3 to 1.8 feet	PSI (2015)	CPT
Area 2 – Linnton South	1.2 to 12.7 feet	CH2MHILL (2006)	2D FLAC Nonlinear Analysis
Area 4 – Willbridge	4.6 to 6.7 feet	GeoEngineers (1998)	CPT

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References for project-specific documents reviewed are included in Table 2.1.

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