Jordan Cove Energy Project L.P.

Resource Report No. 6

Geological Resources

Jordan Cove Energy Project

September 2017

Responses to FERC's August 11th Comment List

Agency	Agency Comment #	Agency Comment	Response Summary
NMFS	6.1	The report indicates that there is a high susceptibility for landslides on slopes surrounding Kentuck Slough. A description of that risk to the mitigation site and any waterbodies potentially affected needs to be included as part of the BA.	See RR6 for landslide risk description..
NMFS	6.2	Design of the rip rap slopes and vegetation system for the Marine Basin and the MOF should be included in the BA.	Will be included in BA
FERC	1	Include supporting literature to justify the statement regarding the time period of "low historical seismicity".	Added reference to RR6 - 6.4.1.1 - Earthquakes.
FERC	2	Include a discussion and supporting literature regarding the historical evidence of earthquake activity related to the Cascadia Subduction Zone (CSZ), and for the several earthquakes that took place in 1873 within 75 miles of Coos Bay.	Added narrative to RR6 - 6.4.1.1 - Earthquakes.
FERC	3	Include definitions for the terms Maximum Considered Earthquake, Operating Basis Earthquake, and Safe Shutdown Earthquake (as used in RR 6).	Added narrative to RR6 - 6.4.1.1 – Earthquakes
FERC	4	Indicate what the "design level earthquake" is for the soil liquefaction discussion. Also include a discussion of what the "cyclic shear stresses of a sufficient magnitude and duration" entails. Include specific references to studies of soils and liquefaction hazard for the Project.	Added narrative to RR6 - 6.4.1.3 – Soil Liquefaction.
FERC	5	Include a statement regarding how the Oregon Department of Transportation seismic design requirements would address the specific seismic risks at the Trans Pacific Parkway/US-101 intersection. Confirm these requirements would address a Cascadia Subduction Zone (CSZ) earthquake scenario.	Added narrative to section 6.4.1.3 - Soil Liquefaction

Agency	Agency Comment #	Agency Comment	Response Summary
FERC	6	Include references and additional justification, including site-specific rationale, to support the statements indicating that subsidence would not be a hazard for the Project.	Added narrative to RR6 - 6.4.3.1 – Co-seismic Subsidence.
FERC	7	Although paragraph 4 of Section 6.4.1.1 discusses in general the FERC Guidelines and design standard categories, there is currently no discussion of how these standards would specifically address seismic hazards for this particular Project. Furthermore, the assertions made in this section regarding these hazards cannot be verified as the studies they rely on have not yet been provided (i.e., Appendix B.6). Therefore, include a discussion of the design standards in light of the specific site conditions and specific Project risks, and include Appendix B.6 in subsequent filings.	Added narrative to RR6 - 6.4.1.1 – Earthquakes. The seismic hazard report will also be included with RR13.
FERC	8	Include a discussion of how peak ground accelerations (PGA) relate to earthquake magnitudes and earthquake return periods. Also include sufficient discussion to clarify how seismic risk (in terms of a potential worst-case seismic scenario) is addressed by the design values presented in Table 6.4-1.	Added narrative to RR6 - 6.4.1.1 – Earthquakes
FERC	9	Include specific Project design return periods and magnitudes that are applicable to the PGAs found in Table 6.4-1. Include footnotes in the table to define the abbreviations and terminology used in this table. Also include references for the design value columns of this table.	Added narrative to RR6 - 6.4.1.1 – Earthquakes

JCEP LNG TERMINAL PROJECT

Resource Report 6 – Geological Resources

To Verify Compliance with this Minimum FERC Filing Requirement:	See the Following Resource Report Section:
1. Identify the location (by milepost) of mineral resources and any planned or active surface mines crossed by the proposed facilities – Title 18 Code of Federal Regulations (CFR) part (§) 380.12 (h)(1 & 2)	Section 6.3
2. Identify any geologic hazards to the proposed facilities – 18 CFR § 380.12 (h)(2)	Section 6.4
3. Discuss the need for and locations where blasting may be necessary in order to construct the proposed facilities – 18 CFR § 380.12 (h)(3)	Section 6.2
4. For liquefied natural gas (LNG) projects in seismic areas, the materials required by "Data Requirements for the Seismic Review of LNG Facilities," National Bureau of Standards Information Report 84-2833 – 18 CFR § 380.12 (h)(5)	Section 6.5 RR 13 Appendix J.13.4, I.13.1, and J.13.4
5. For underground storage facilities, how drilling activity by others within or adjacent to the facilities would be monitored, and how old wells would be located and monitored within the facility boundaries.	Not Applicable

INFORMATION RECOMMENDED OR OFTEN MISSING	See the Following Resource Report Section:
1. Identify any sensitive paleontological resource areas crossed by the proposed facilities. (Usually only if raised in scoping or if the project affects federal lands.)	Section 6.6
2. Briefly summarize the physiography and bedrock geology of the project.	Section 6.1
3. If proposed pipeline crosses active drilling areas, describe plan for coordinating with drillers to ensure early identification of other companies' planned new wells, gathering lines, and above-ground facilities.	N/A
4. If the application is for underground storage facilities:	N/A
Describe monitoring of potential effects of the operation of adjacent storage or production facilities on the proposed facility, and vice versa;	
Describe measures taken to locate and determine the condition of old wells within the field and buffer zone and how the applicant would reduce risk from failure of known and undiscovered wells; and	
Identify and discuss safety and environmental safeguards required by state and federal drilling regulations.	

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OTHER REFERENCED APPENDICES (INCLUDED IN RESOURCE REPORT 13)

ACRONYMS

ASCE	American Society of Civil Engineers
CFR	Code of Federal Regulations
CPT	Cone Penetration Test
CSZ	Cascadia Subduction Zone
DOGAMI	Oregon Department of Geology and Mineral Industries
FERC	Federal Energy Regulatory Commission
JCEP	Jordan Cove Energy Project, L.P.
km	kilometer
LNG	liquefied natural gas
MOF	Material Offloading Facility
M_W	moment magnitude
NGVD 29	National Geodetic Vertical Datum
NSHM	National Seismic Hazard Map
PCGP	Pacific Connector Gas Pipeline, LP
PGA	Peak Ground Accelerations
RFP	Roseburg Forest Products Company
SLIDO	Statewide Landslide Information Database for Oregon
SPT	Standard Penetration Test
U.S.	United States
USGS	U.S. Geological Survey

6.0 INTRODUCTION

Jordan Cove Energy Project, L.P. ("JCEP") is seeking authorization from the Federal Energy Regulatory Commission ("FERC" or "Commission") under Section 3 of the Natural Gas Act to site, construct, and operate a natural gas liquefaction and liquefied natural gas ("LNG") export facility ("LNG Terminal"), located on the bay side of the North Spit of Coos Bay, Oregon. JCEP will design the LNG Terminal to receive a maximum of 1,200,000 dekatherms per day ("Dth/d") of natural gas and produce a maximum of 7.8 million metric tons per annum ("mtpa") LNG for export. The LNG Terminal will turn natural gas into its liquid form via cooling to about -260ºF, and in doing so it will reduce in volume to approximately 1/600th of its original volume, making it easier and more efficient to transport.

In order to supply the LNG Terminal with natural gas, Pacific Connector Gas Pipeline, LP ("PCGP") is proposing to contemporaneously construct and operate a new, approximately 229-mile-long, 36-inch-diameter natural gas transmission pipeline from a point of origin near the intersection of the Ruby Pipeline LLC ("Ruby") and Gas Transmission Northwest LLC ("GTN") systems to the LNG Terminal ("Pipeline," and collectively with the LNG Terminal, the "Project"). . PCGP will submit a contemporaneous application to FERC that will include its own set of resource reports with references to certain materials in the LNG Terminal resource reports.

This Resource Report 6 contains a discussion of, and an evaluation of, the potential impacts to geologic resources within the JCEP Project Area.

6.1 GEOLOGIC SETTING

The LNG Terminal site (which includes Ingram Yard, the Access and Utility Corridor, and the South Dunes site) is located within the Pacific Border Physiographic province at the western edge of the coastal headlands of the Central Coast Mountain Range, on the North Spit of Coos Bay. The North Spit of Coos Bay marks the southern edge of the Holocene-age Coos Bay dune sheet (Peterson, et al. 2005). The Kentuck Project site is located east of Coos Bay near the confluence of Kentuck Slough and Mettman Creek.

The LNG Terminal is located near the eastern edge of the Cascadia Subduction Zone ("CSZ"), an active convergent plate boundary between the segmented subducting Explorer, Juan de Fuca, and Gorda Plates and the overriding North America Plate (Wells, et al. 2000). The LNG Terminal site is within Juan de-Fuca Plate segment of the CSZ. The converging tectonic plates have resulted in the accretion of marine deltaic sediments and volcanic seamounts, referred to as the Siletzia terrane, to the western edge of the North American Plate (Heller and Ryberg 1983).

The active converging tectonic plates create a deformation zone along the western edge of the accretionary wedge complex, strike-slip faulting in the North American Plate, and a zone of bedrock folding that extends from the coast eastward. The major tectonic elements associated with the subduction zone include the accretionary wedge complex, a deformed forearc basin (the Coast Range and Willamette Valley), a volcanic arc complex (the Cascade Mountain Range), and a backarc (eastern Oregon and Washington).

The project area lies at the junction of the accretionary wedge complex and the forearc basin. Local bedrock structures reflect east-west compressional deformation resulting from ongoing oblique subduction on the CSZ that has occurred since the late middle Miocene epoch (Wells and Peck 1961), and includes the megathrust itself, north-south-trending folds, north-south-trending reverse and thrust faults, and west-northwest-trending oblique strike-slip faults (Black and Madin 1995; Madin et al. 1995; Goldfinger et al. 1992). The location and extent of local fold and fault structures have been inferred from stratigraphic, geomorphic, and geophysical evidence. Exposed geologic structures south of the site include the South Slough Syncline, the Westport Arc (anticline), and the eastern and western forks of the Westport Arc (Allen and Baldwin 1944).

Sedimentary rocks exposed in the Coos Bay area were deposited during periods of sea level change (regressions and transgressions) in the Coos Basin. Convergent tectonism uplifted these sedimentary rocks and caused the initial faulting and folding now observed in bedrock units south and east of Coos Bay. Bedrock units exposed south of the LNG Terminal site have been folded into a series of north-trending anticlines and synclines primarily due to east-west tectonic compression. The anticlines and synclines plunge slightly to the north and may be present at depth beneath the LNG Terminal site. Faults are generally either north-south-trending reverse faults or thrust faults, bedding plane reverse faults, or west-northwest reverse faults (Madin et al. 1995).

Neogene age bedrock exposed in the Coos Bay region includes Eocene to Pliocene marine interbedded siltstones and sandstones of the Coaledo Formation, the Bastendorff Formation, and the Empire Formation (Baldwin et al. 1973; Beaulieu and Hughes 1975). The Coaledo Formation is composed of coarse to fine-grained, hard, deltaic sandstone with interbeds of softer siltstone, conglomerate, and coal beds. The upper member of the Coaledo Formation is composed of gray, coarse to fine-grained weakly cemented sandstone, mudstone, and minor amounts of coal. The upper Coaledo Formation also underlies alluvial deposits in the Kentuck Slough. The Bastendorff Formation consists of thinly laminated gray siltstones and mudstones that weather to light brown. The Empire Formation is medium-grained sandstone with minor siltstone, conglomerate, and water laid tuff (Madin et al. 1995). The lower, fossiliferous portion of the Empire Formation has been described informally as the Miocene Beds exposed at Fossil Point southwest of the LNG Terminal (Ehlen 1967).

The LNG Terminal site is located within the Coos Bay dune sheet that features lowland areas generally underlain by Quaternary age unconsolidated wind-blown sediments. Geologic mapping in the area shows stable (vegetated) and unstable sand dunes (Beaulieu and Hughes 1975) underlain by sedimentary rock. Recent dating of dune sand in the Florence and Coos Bay dune sheets indicates that the dunes in the immediate proximity of the project area were deposited during the late Holocene epoch; however, late-Pleistocene dune deposits may also be exposed (Peterson et al. 2005).

Surface soils have been disturbed by the operations of the Roseburg Forest Products Company ("RFP"), the Weyerhaeuser Company, and the former Kentuck Golf Course, and from the placement of fill material. The U.S. Army Corps of Engineers spread materials dredged during maintenance of the Coos Bay navigation channel on the LNG Terminal site. The historical fill materials placed on portions of the LNG Terminal site are predominantly sand with a small percentage of silt.

The site topography varies within Ingram Yard and ranges from approximately elevation 20 feet in the western portion of the site, which is relatively flat, up to approximately elevation 125 feet in the northeastern portion of the site, which is covered by a north-south oriented longitudinal sand dune. Along the Access and Utility Corridor, ground surface elevations are variable, ranging from a low of approximately 20 feet up to 135 feet in the area of a dune. Adjacent to wetlands within the Access and Utility Corridor, the ground surface elevations are approximately 15 feet. At the South Dunes site, the elevation is less variable and is approximately 15 feet. In the western portion of the South Dunes Site, the elevation increases up to approximately 60 feet in the area of a landfill. For all areas of the LNG Terminal adjacent to Coos Bay, the ground surface slopes down to meet the water level.

Geotechnical studies for the LNG Terminal facilities on the North Spit indicate that the LNG Terminal site is typically mantled with relatively clean, fine-grained sand. The geotechnical investigations for all of the sites that will be utilized for construction and operation of the LNG Terminal (“JCEP Project Area”) are provided in the Geotechnical Data Report (J1-000-GEO-RPT-GRI-00033-00) included in Appendix J.13.4 of Resource Report 13. Historical records and aerial photography indicate that the flatter, upper portion of the LNG Terminal site on the North Spit is typically sand fill to depths of about 10 feet. The sand fill ranges from loose to very dense, and is typically underlain by dense to very dense sand with a trace of silt. Organics are present in portions of the site where the site was not stripped prior to fill placement. The very dense sand is underlain by refusal blow count silt and sand at depths of about 120 feet in the vicinity of the proposed tanks. A geotechnical boring completed on Ingram Yard encountered hard clayey silt that was classified as poorly indurated silty shale at a depth of about 252 feet. Another boring drilled about 480 feet north, did not encounter the poorly indurated silty shale when terminated at a depth of about 280 feet; rather, the soil was classified as silt and sand at this depth. Outcrops of weathered sandstone and siltstone of the Coaledo Formation can be observed north and south of the Trans Pacific Parkway/US-101 Intersection. At the South Dunes site, bedrock was not encountered in a boring, which terminated at a depth of approximately 240 feet in clayey silt that was first encountered at a depth of approximately 168 feet.

Two overwater geophysical surveys have been performed between the LNG Terminal and the Southwest Oregon Regional Airport, which is located on the south side of the Federal Navigation Channel (J1-000-GEO-RPT-GRI-00033-00). The overwater surveys indicated that bedrock is present at a depth of about elevation -120 feet near the south edge of the proposed slip.

Sand fill is also present to a depth of about 15 feet at the location of the Trans Pacific Parkway/US-101 Intersection. An approximate 5-foot-thick layer of alluvial silt to clayey silt was encountered beneath the sand fill at this location. Similar to the other locations on the North Spit, this area is underlain by sand until reaching the underlying Coaledo Formation. The sand at the intersection locations is typically loose to a depth of about 30 feet and dense to very dense below 30 feet (J1-000-GEO-RPT-GRI-00033-00). Although the Coaledo Formation was not encountered in the explorations for the intersection, this formation can be seen in roadside cuts along US-101 north and south of the LNG Terminal site.

Geotechnical information at the Kentuck Project site indicates that the lower-lying portions of the site are mantled and underlain by soft alluvial deposits to depths of more than 100 feet in some areas. The alluvium consists of interbedded layers of sand, silt, and clay that can contain a relatively high organic content. Dense sands were encountered beneath the soft alluvial deposits at depths of about 100 feet on the western portion of the site. Sandstone and siltstone of the Coaledo Formation underlie the alluvial deposits, and form the hillsides north and south of the site. The depth of the Coaledo Formation varies with the distance from the hillsides.

The known or interpreted surface and bedrock geology of the JCEP sites is summarized in Table 6.1.1.

Table 6.1-1 Surface and Bedrock Geology Summary

	Jordan Cove Site	Surface Geology	Bedrock Geology
Jordan Cove Energy Project Areas	Myrtlewood Offsite Park & Ride	Fills, Quaternary surficial deposits	Coaledo Formation
	Trans Pacific Parkway/US-101 Intersection Widening	Sand	Coaledo Formation
	Mill Casino Offsite Park & Ride	Fills	Upper Coaledo Formation
	APCO Site	Fills, Quaternary alluvium	Potentially Coaledo Formation
	Boxcar Hill	Dune deposits	Potentially Coaledo Formation
	South Dunes	Fills, Dune deposits	Potentially Coaledo Formation
	Ingram Yard	Fills, Unstable and stable dune sand	Potentially Coaledo Formation
	Industrial Wastewater Pipeline	Unstable and stable dune sand, deflation plain and beach sand	Coaledo Formation
	Meteorological Station	Beach Sand	Coaledo Formation
	Port Laydown Site	Beach Sand	Bastendorff Formation
	Dredge Area 1	Coos Bay Channel Sand	Empire Formation
	Dredge Area 2	Coos Bay Channel Sand	Bastendorff Formation
	Dredge Area 3	Coos Bay Channel Sand	Coaledo Formation
	Dredge Area 4	Coos Bay Channel Sand	Coaledo Formation
Mitigation Sites	Eelgrass Mitigation Site	Coos Bay Channel Sand	Potentially Coaledo Formation
	Kentuck Project	Quaternary alluvium (sand, silt, clay, organics)	Upper Coaledo Formation
	Lagoon	Beach Sand	Coaledo Formation

Impacts to surface geology will be limited primarily to the construction phase of the LNG Terminal, when the topographic features at specific locations on the site will be altered by clearing, mechanical excavation, dredging, and fill placement. All soil material cut at Ingram Yard, including material dredged for the marine slip, will be used for regrading purposes at LNG Terminal site, on the RFP property, and at the Kentuck Project site. The majority of the slip dredging/excavation will be completed in an upland environment to minimize in-water work. The in-water dredging work will occur only as the last phase of work during an in-water work window.

All open-water marine construction will be completed in accordance with permitted in-water work windows.

Site elevations for the LNG Terminal are provided in Resource Report 1, Table 1.3-1. The elevations of the portion of the RFP property to be filled will be aligned with the elevations of Ingram Yard and the Access and Utility Corridor. Final grading and landscaping consists of gravel surfacing, asphalt surfacing, concrete paved surfaces, and grassed areas.

6.2 BLASTING

Geotechnical investigations completed for the LNG Terminal have not identified hard rock within the development boundary that requires blasting. No blasting would be required during any phase of construction of the LNG Terminal, as geotechnical investigations indicate that the site consists of unconsolidated granular material. Therefore, no impacts from blasting are anticipated.

6.3 MINERAL RESOURCES

The principal mineral production of Oregon in order of value was crushed stone, and construction sand and gravel (USGS 2013). Mineral resources identified in Coos County, Oregon, include gold, platinum, chromium, clay, manganese, sand, gravel, silica, stone, and titanium (Baldwin et al. 1973). There are three permitted sand and gravel mines within 0.25 mile of the LNG Terminal site. All three of these mines are closed and are not producing material (DOGAMI 2017).

Based on studies of coal and natural gas resources in the Coos Bay area (Newton 1980; Mason and Hughes 1975), coal and gas resources may occur at depths within or below sandstone at the LNG Terminal site. Coal deposits are present in the upper and lower members of the Coaledo Formation. However, the depth of burial (paleotemperature) was not sufficient to generate large quantities of hydrocarbons (Newton 1980). The Steva coal seam and the Hardy coal seam have been identified within the vicinity of the Kentuck Project site (Diller 1914). Based on the State of Oregon Mineral Information Layer for Oregon-Release 2, there are no permitted coal mines or oil and gas wells within 0.25 mile of the LNG Terminal site (DOGAMI 2017).

Construction and development of the LNG Terminal site will not affect any known or potential mineral resources or the recovery of any mineral resources and would not preclude future utilization of potential leasable resources in deeper, underlying geologic formations.

6.4 GEOLOGIC AND OTHER NATURAL HAZARDS

Geologic and other natural hazards in the vicinity of the LNG Terminal include seismic hazards, landslides, ground subsidence, and other natural hazards and these are discussed further below.

6.4.1 Seismic Hazards

Seismic hazards include earthquakes, faults, soil liquefaction, and tsunami and are discussed below.

6.4.1.1 Earthquakes

As discussed in the Seismic Ground Motion Hazard Study (J1-000-GEO-RPT-KBJ-50002-00) provided in Appendix I.13.1 of Resource Report 13, the Cascadia Subduction Zone (CSZ) is the dominant tectonic feature in western Oregon, and various lines of geologic evidence indicate the CSZ has produced numerous megathrust earthquakes over the past 7,000 years or more

(Atwater et al. 1995; Goldfinger et al. 2012). These lines of geologic evidence include sudden coastal subsidence (Atwater 1992; Atwater et al. 1995), onshore tsunami deposits (Atwater et al. 1995; Kelsey et al. 2002; Kelsey et al. 2005; Williams et al. 2005; Nelson et al. 2006), offshore turbiditie deposits (Goldfinger et al. 2012), and soil liquefaction features (Atwater et al. 1995; Kelsey et al. 2002). Historical observations and modeling also indicate the most recent megathrust earthquake in 1700 likely ruptured the whole CSZ and produced a tsunami that inundated harbors in Japan (Satake et al. 1996; 2003). Megathrust earthquakes occur when the fault between the tectonic oceanic plate subducting beneath the continental North American plate suddenly slips (Audet et al. 2010). Offshore, subduction causes a deformation zone along the western edge of the accretionary wedge complex, strike-slip faulting in the North America plate, and a zone of folding that extends from the coast westward. Onshore, the major tectonic elements associated with the subduction zone include the accretionary wedge complex of the Oregon Coast Range, deformed forearc basin (Willamette Valley), a volcanic arc complex (the Cascade Range), and a backarc region east of the Cascade Mountains (Lewis et al. 2003).

The Coos Bay area is located in a zone of low historical seismicity, and records of earthquakes that could be felt in the vicinity of the LNG Terminal site are limited (Goldfinger et al. 1992; Wells et al. 1998; Wong 2005). There is one historical record of an earthquake with a magnitude greater than 3.0 within a 50 kilometer ("km") radius of the site in the Advanced National Seismic System database and the U.S. Geological Survey ("USGS") 2014 National Seismic Hazard Map ("NSHM") earthquake database. This earthquake was a local magnitude 3.3 event located off the coast of Barview, Oregon (22 km away) in 2016. Other notable earthquakes felt in the vicinity of the LNG Terminal site include a moment magnitude ("M_w") earthquake rated 7 off the coast of Crescent City, California (200 km away) in 1991, a M_w 4.9 off the coast of Waldport, Oregon (105 km away) in 2004, a M_w 6 approximately 280 km off the coast of Coos Bay in 2012, and a M_W 5.5 approximately 225 km off the coast of Coos Bay in 2016. Additionally, an earthquake with an estimated M_w of 6.9 occurred near Brookings, Oregon (about 155 km away from the LNG Terminal site) in 1873 (Petersen et al. 2014). This earthquake was reportedly felt as far away as Portland, Oregon, and isoseismal maps indicate a Modified Mercalli (MM) intensity of V was likely felt in the Coos Bay area (Wong and Bott 1995; Niewendorp and Neuhaus 2003). A MM of V suggests ground motions were felt by most people in Coos Bay, but that slight or no damage occurred.

Ground motion results were developed in accordance with 49 CFR Part 193 and the FERC Guidelines. 49 CFR Part 193 specifies the 2006 edition of the National Fire Protection Association 59A Standard as the basis for seismic design of LNG terminals. The FERC Guidelines require application of portions of the American Society of Civil Engineers ("ASCE") 7-05 standard. Because the LNG Terminal will use the 2014 Oregon Structural Specialty Code that references ASCE 7-10, development of the ground motions also considers ASCE 7-10.

The FERC Guidelines categorize LNG facility structures and ancillary components to define which design standard is applicable for each structure. The guidelines provide different seismic recommendations for the following classes of structures: (1) Category I structures, (2) Category II structures, and (3) Category III structures. The FERC Guidelines define waterfront structures as a specific subset of Category II structures with their own definitions. The seismic guidelines for these groups of structures are outlined in the Seismic Ground Motion Hazard Study (J1-000-GEO-RPT-KBJ-50002-00) provided in Appendix I.13.1 of Resource Report 13. The Seismic Design Philosophy for FERC Facilities (J1-000-STR-PHL-KBJ-50001-00) and associated seismic load criteria for Category I, II and III structures, systems and components are also included in Appendix I.13.1 of Resource Report 13. Seismic categorization of structures, equipment and piping is provided in the Seismic Design Criteria (J1-000-STR-BOD-KBJ-50001-00) included in Appendix I.13.1 of Resource Report 13.

In Table 6.4-1, the peak ground accelerations ("PGA") and the spectral acceleration values at 0.2 and 1.0 second periods from the USGS NSHM are compared to the site-specific values for the same return periods for the Maximum Considered Earthquake ("MCE"), Operating Basis Earthquake (OBE), and Safe Shutdown Earthquake (SSE) ground motions. The SSE and OBE are defined in accordance with the FERC Guidelines, which indicate the SSE is taken as the ground motion with a 2% probability of exceedance within a 50 year period with appropriate deterministic limits and the OBE is taken as the ground motion with a 10% probability of exceedance within a 50 year period. The MCE is defined in accordance with the 2014 Oregon Structural Specialty Code, which indicates the MCE is a risk-target ground motion (identified as MCE_R) intended to achieve a 1% probability of collapse within a 50 year period with appropriate deterministic limits (ASCE 7, 2010). Deaggregation of the seismic hazard results indicate the primary source of hazard for the SSE, MCE and OBE levels of ground motion is a magnitude 8.9 to 9.1 event on the CSZ. The CSZ, which is the largest magnitude seismic source in western Oregon, contributes more than 88 percent of the seismic hazard to the OBE ground motions and more than 99 percent to the SSE and MCE ground motion as detailed in the Seismic Ground Motion Hazard Study (J1-000-GEO-RPT-KBJ-50002-00) provided in Appendix I.13.1 of Resource Report 13. Similarly, a magnitude 9 event on the CSZ also controls the deterministic ground motions at the site. The comparison in Table 6.4-1 includes values for soft rock site conditions as well as the anticipated site soil conditions after construction. Site specific nonlinear site response modeling was utilized to estimate the ground surface values.

Table 6.4-1 Ingram Yard Seismic Ground Motion Values

Parameter (as defined in the 2014 OSSC and ASCE 7)	Site-Specific Design Values	ASCE 7-05 Design Values[1]	ASCE 7-10 Design Values[1]
S_S – Mapped short period spectral response accelerations [2] (5 percent damped) (g)	1.692[#]	1.500	1.409
S_1 – Mapped 1 second period spectral response accelerations [2] (5 percent damped) (g)	0.737[#]	0.718	0.717
Site Class (After mitigation of liquefiable soils)	D	D	D
Site Coefficient F_a	Not Applicable	1.0	1.0
Site Coefficient F_v	Not Applicable	1.5	1.5
S_{DS} - Design spectral response accelerations at short periods (5 percent damped) (g)	1.025	1.000	0.939
S_{D1} - Design spectral response accelerations at 1 second period (5 percent damped) (g)	1.002	0.718	0.717
PGA – Mapped geometric mean peak ground acceleration [2] (5 percent damped) (g)	0.816[3]	Not Applicable	0.614
Site Coefficient F_{PGA}	Not Applicable	Not Applicable	1.000
PGA_M – Design geometric mean peak ground acceleration (5 percent damped) (g)	0.490	0.600 (S_S/2.5)	0.614

Notes:

[1] Ground motion values obtained from the USGS ground motion parameter calculator (http://earthquake.usgs.gov/designmaps/us/application.php)

[2] Deaggregation of the site-specific seismic hazard indicates 99 percent of the hazard is from a magnitude 8.9 to 9.1 event on the CSZ

[3] Geometric mean ground motion for 2 percent probability of exceedance in 50 year period

g = standard acceleration of gravity

6.4.1.2 Faults

Twelve faults that are considered seismogenic in the USGS NSHM are located within 150 km of the LNG Terminal site. Table 6.4-2 provides information on the faults, and Figure 6.4-1 shows the fault locations. These 12 faults include the CSZ, a megathrust fault, and 11 crustal faults. The three closest faults are the CSZ (15 km), the South Slough (16 km), and the Coquille Anticline (27 km). There are three additional crustal faults within 65 km of the LNG Terminal site and six additional crustal faults within 132 km of the LNG Terminal site. The CSZ has the greatest potential for earthquakes with a M_w of 8.6 to 9.3 during a rupture of the entire length of the CSZ (Petersen et al. 2014). The maximum M_w of the crustal faults range from 6.1 to 7.3 (Petersen et al. 2014). Additional information on these faults can be found in the Seismic Ground Motion Hazard Study (J1-000-GEO-RPT-KBJ-50002-00) provided in Appendix I.13.1 of Resource Report 13.

McInelly and Kelsey (1990) identified possible Holocene fault activity on the Barview fault based on radiocarbon dating of tree stumps hypothesized to have died about 220 years before, when displacement on the fault placed them in contact with more saline or brackish water. However, observations of living and dead trees just above the tide zone by Madin et al. (1995) suggest that normal wave erosion rather than fault movement may have killed the trees. Given the uncertainty in the potential activity of the Barview fault, this fault is added to the 12 other faults in Table 6.4-2 that are considered seismogenic in the USGS NSHM for consideration at the LNG Terminal site. A maximum M_W of 6.3 is used for the Barview fault, because it may be part of the South Slough faults identified in the USGS NSHM.

In addition to the faults discussed and identified in the USGS database and the Barview fault, Briggs (1994) indicated the possibility of a Holocene-active fault located in Pony Slough, immediately southeast of the LNG Terminal site. The potential for a fa`ult was based on vertical displacements measured in stratigraphy of marsh core samples obtained from across Pony Slough. An overwater geophysical seismic reflection survey was conducted to further explore the possible existence of an active fault at this location. The geophysical survey was located across the mouth of Pony Slough and along the east-west aligned portion of Coos Bay, immediately south of the South Dunes site, to identify depth to bedrock, bedrock structure, and the presence of any unmapped faults (RR13 Appendix J.13.4). Review of the results of the seismic reflection survey at Pony Slough did not indicate the presence of a fault across Pony Slough or across the bay at the location of the LNG Terminal site.

Table 6.4-2 Fault Sources within 100 miles (161 Kilometers) of the LNG Terminal Site

Fault Name	Fault Type	Maximum Magnitude	Distance form Site Miles (km)
Cascadia Subduction Zone	Megathrust	8.6 - 9.3	9.3 (15)
Barview fault	Thrust	6.3	4.5 (7.3)
South Slough faults	Thrust	6.3	9.9 (16)
Coquille anticline	Reverse	6.8	17 (27)
Battle Rock fault zone	Normal	7.0	39 (63)
Beaver Creek fault zone	Normal	6.5	40 (64)
Cape Blanco anticline	Thrust	7.1	40 (65)

Fault Name	Fault Type	Maximum Magnitude	Distance form Site Miles (km)
Waldport faults	Reverse	6.4	63.4 (102)
Whaleshead fault zone	Strike Slip	7.0	64.0 (103)
Alvin Canyon fault	Strike Slip	7.2	68.4 (110)
Stonewall anticline	Reverse	6.8	73.9 (119)
Daisy Bank fault	Strike Slip	7.3	80.2 (129)
Yaquina faults	Reverse	6.1	82.0 (132)

6.4.1.3 Soil Liquefaction

Field and laboratory studies have demonstrated that if saturated, loose to medium dense sands and some softer, low plasticity, fine-grained soils, such as sandy silts, are subject to cyclic shear stresses of a sufficient magnitude and duration, an increase in pore water pressure can result. As pore water pressure increases, the effective stress in the soil mass below the groundwater level decreases, which results in a corresponding loss of shear strength in the saturated material. If the pore water pressure ratio approaches 100 percent, the material will lose most of its shear strength and deform as a viscous fluid (complete liquefaction) when a shear stress greater than the shear strength is applied. Additional information on liquefaction can be found in the Geotechnical Report (J1-000-GEO-RPT-KBJ-50001-00) provided in Appendix J.13.4 of Resource Report 13.

The site specific ground motions used for liquefaction analyses correspond to cyclic stresses consistent with a geometric mean PGA_M of 0.49g and a M_W 9.0 earthquake.

A quantitative liquefaction potential evaluation was conducted for the LNG Terminal site using the site specific ground motions and in situ and laboratory testing information available for the LNG Terminal site. The methods of Youd et al. (2001), Moss et al. (2007), and Boulanger and Idriss (2016) were used to assess the liquefaction potential.

The majority of the sand soils encountered at Ingram Yard are dense enough to resist liquefaction during design-level earthquakes; however, liquefiable soils are present throughout the Ingram Yard, with depths varying with location. Liquefaction at Ingram Yard and along the Access and Utility Corridor is estimated to be in distinct soil layers from the groundwater table to a maximum of approximately elevation -30 feet (NAVD 88). At the Ingram Yard and Access and Utility Corridor, the liquefiable layers could extend below the dunes. At the South Dunes site, liquefaction is estimated in a soil zone that starts at the groundwater table and extends to variable depths from elevation 0 feet to approximately elevation -25 feet.

Ground improvement methods will be used to mitigate the risk of liquefaction. Ground improvement methods being considered includes vibro-compaction and deep soil mixing. During vibro compaction, clean sand flows from shallow to greater depths penetrated by a vibrator (vibroflot) that is lowered into the ground to impart vibrations to the soil. As the vibroflot is withdrawn, compaction of the sand that flows around the vibroflot and the surrounding soil is performed. The impacts to the site of vibro-compaction will be an increased density of the sand, and there are no anticipated adverse consequences to the geology. Sand densification confirmation would be performed using cone penetrometer soundings or borings with standard penetration testing. Deep soil mixing would consist of installing overlapping (secant) soil mixed columns to create shear walls that reinforce the liquefiable soil mass. The deep soil mixed shear walls would be installed using multiple overlapping augers. Binder would be injected as slurry while the augers advance to the target depth to produce columns of soil-cement mix. Confirmation of deep soil mixing is evaluated with the use of strength testing of the mixed material and confirmation of the design geometry. The impact of deep soil mixing will be reinforcement of the soils, and there are no anticipated adverse consequences to the geology.

Similar to the other facilities on the North Spit, the risk of liquefaction at the Trans Pacific Parkway/US-101 Intersection is most significant to depths of about 25 feet, and denser materials were encountered below this depth. The proposed improvements at the Trans Pacific Parkway/US-101 Intersection will be constructed to meet the Oregon Department of Transportation seismic design requirements. These requirements include designing the proposed retaining walls to the 1,000 year seismic hazard level and addressing liquefaction risks during a M9 Cascadia Subduction Zone earthquake.

The explorations completed at the Kentuck Project site encountered soft and loose alluvial soils to depths of about 100 feet in some areas. The loose sand and low plasticity silts encountered are susceptible to liquefaction, while higher plasticity silts and clays encountered are not liquefiable. The design for the bridge structure planned at the Kentuck Project site will consider liquefaction effects.

Lateral spreading involves lateral displacement of large, surficial blocks of soil as a result of liquefaction of a saturated surface layer, and can develop in gentle slopes and move toward a free face, such as a river channel. Displacement occurs in response to the combination of gravitational forces and inertial forces generated by an earthquake. Where a free face exists, liquefiable soils may be susceptible to lateral spreading. Where soil susceptible to lateral spreading is present at the LNG Terminal, ground improvement consisting of vibro-compaction and/or deep soil mixing will be completed to mitigate the lateral spreading hazard.

6.4.1.4 Tsunamis

The west coast of the United States has historically been subject to minor inundation from tsunamis generated by distant earthquakes in South America, Alaska, and Japan. Kelsey et al. (2005) note that tsunamis generated from these distant subduction zone earthquakes have minor inundation effects because of the long diagonal approach of tsunami waves to the west coast from these sources. Based on this explanation, observations made around the Indian Ocean following the 2004 megathrust Sumatran earthquake, and recent modeling (DOGAMI 2012) indicate a tsunami generated by a megathrust earthquake on the Cascadia Subduction Zone (CSZ) will present the greatest tsunami inundation risk at the LNG Terminal site.

The impacts and hazards of tsunamis to an industrialized area were well illustrated during the 2011 Tohoku, Japan earthquake. This tsunami was generated by an offshore subduction zone earthquake; subsidence occurred and increased the tsunami impacts significantly in some areas. Because similar earthquake and subsidence are of concern off the Oregon coast, the lessons learned from the 2011 Tohoku earthquake regarding subsidence, run-up, scour, and foundation performance provide a useful case history for evaluating hazards at the LNG Terminal site.

The Oregon Department of Geology and Mineral Industries ("DOGAMI") produced tsunami hazard maps for a tsunami generated by a megathrust earthquake on the CSZ, for most of the Oregon coast in 1995 (Priest 1995), 2002 (Priest et al. 2002), and more recently in 2012 (DOGAMI 2012). All studies include scenarios that vary wave height and co-seismic subsidence. It is noted that the 1995 maps are not considered to be accurate in accordance with the most recent research, but are referred to here since they are still officially referred to in Oregon Revised Statutes 455.466 and 455.447, which are referenced in the building code.

Using the latest hydrodynamic modelling methods and knowledge, site-specific tsunami run-up modelling was completed by Coast and Harbor Engineering (CHE) in 2013; these results have been included in the Tsunami Hydrodynamic Modeling (J1-000-MAR-TNT-CHE-00001-00) report provided in Appendix I.13.2 of Resource Report 13). Subsequently, the outcome of the Coast and Harbor Engineering modelling work was verified via a separate modelling effort performed by Moffatt & Nichol (M&N), these results are detailed in the Tsunami Modeling Report (J1-000-MAR-RPT-MON-00001-00) included in Appendix I.13.2 of Resource Report 13. Modelling efforts by each consultant demonstrated good agreement and results within typical bounds of accuracy for such modelling efforts.

Modelling efforts and maximum run-up elevations scenarios evaluated by each consultant were established using the following criteria and assumptions:

- Cascadia Subduction Zone rupture consistent with the DOGAMI L1 rupture scenario.
- 2,475 year rupture return period consistent with the FERC design tsunami event
- The post-construction geometry for the LNG Terminal and South Dunes
- The current bathymetry adjacent to the LNG Terminal and South Dunes
- The current topography adjacent to the LNG Terminal and South Dunes
- 1.3 factor of safety applied to the modelled maximum surface elevation relative to the initial pre-tsunami water elevation.
- A pre-tsunami water elevation equal to Mean High Water (MHW)
- A predicted subsidence of 7.6ft at the LNG Terminal site for the L1 event.

For comparison purposes, M&N completed studies using the DOGAMI L1, XL1 and XXL1 rupture scenarios, representing the 2,475 yr., 10,000 yr. and >10,000 yr. events, respectively. These studies are detailed in the Tsunami Maximum Run-up Modelling (J1-000-MAR-RPT-MON-00002-00) provided in Appendix I.13.2 of Resource Report 13.

In order to ensure modeled run-up elevations were not limited by site elevations, modelling efforts used post construction site elevations well in excess of those required to mitigate the design tsunami. This strategy allowed maximum run-up elevations to be determined without the need for multiple modelling iterations.

The LNG Terminal will be designed to mitigate inundation due to the design tsunami. Mitigation shall take the form of elevating the site or protecting those parts of the LNG Terminal considered critical to the safety and integrity of the facility. Where practical, the site shall be raised to a height equal to or greater than the maximum run-up elevation. Exceptions include non-critical water dependent facilities such as the Emergency Lay Berth, Tug Boat Berth and Materials Offloading Facilities. For the LNG Tanks, these shall be provided with protective berms extending to or above the maximum run-up elevation. The maximum tsunami run-up elevation was determined in accordance with the assumptions and modelling strategy described above. The maximum run-up determined by comparison of the CHE Tsunami Hydrodynamic Modeling (J1-000-MAR-TNT-CHE-00001-00) and M&N Tsunami Modelling (J1-000-MAR-RPT-MON-00001-00) studies, included in Appendix I.13.2 of Resource Report 13, was established to be 34.5ft (NAVD88). The maximum run-up occurred within the marine basin in both cases.

The tsunami modelling studies established that the worst case run-up occurs within the marine basin. In order to understand the impact that the basin configuration might have on run-up elevations, further sensitivity modelling was performed by M&N, the modeling results are included in the Tsunami Maximum Run-up Modelling (J1-000-MAR-RPT-MON-00002-00) study provided in Appendix I.13.2 of Resource Report 13. The study used three different basin configurations modeling a combination of vertical walls and sloping revetments. The study demonstrated that the marine basin configuration didn't have a significant impact on the run-up, achieving a maximum differential of 1.6ft between configurations and a maximum run-up of 34.7ft for a configuration that is not being proposed. Recognizing that the greatest differential came about due to a configuration not being proposed, the maximum run-up of 34.5ft (NAVD88) has been adopted in line with the proposed marine basin configuration.

Further to the site specific modelling work performed by CHE and M&N, NOAA recently developed and issued tsunami run-up mapping showing the run-up elevations for the undeveloped LNG Terminal site. The NOAA maps are the basis for the new tsunami provisions referenced in the recently released ASCE7-16 document. A recent study completed by M&N, Tsunami Wave Run-up Comparison (J1-000-MAR-RPT-MON-00006-00) provided in Appendix I.13.2 of Resource Report 13, compared the run-up elevations for the existing site topography, using the NOAA mapped values and the JCEP project specific model at the points shown in Figure 6.4-4. These points represent the location of maximum run-up (inundation limit) predicted by NOAA in the proximity of the LNG Terminal site. The NOAA mapped values were processed accounting for subsidence (In accordance with ASCE7-16 Geo-Database) and a factor of 1.3 in the same manner as the project specific model results. For the comparison, the project specific model run-up was extracted not at the exact NOAA mapped value locations but at the location of maximum run-up (inundation limit), which across most of the site occurred further inland than the NOAA mapped values, as shown in Figure 6.4-4.

The comparison between the factored NOAA run-up and the project specific model is presented in Figure 6.4-5 for 140 location points. Table 6.4.-3 presents the comparison for a subset of location points representative of the spatial variation of differences in run-up. The graphic shows that the maximum run-up elevations established by the project specific modelling efforts exceed those provided by the factored NOAA run-up except at Points 14 – 15 (Ingram Yard), Point 33 (Southeast of marine slip), and Points 96 – 116 (Northeast corner of South Dunes). Points 14-15 and Point 33 are the result of localized differences in the ground elevations and grid resolution between NOAA and the project specific model and not representative of a trend in the area. The factored NOAA run-up at these points is 1 – 1.9 ft. higher than the project specific model. Along Points 96 – 116, the higher factored NOAA run-up is also attributed to differences in ground elevations, particularly regarding the resolution of specific local features (e.g. Jordan Cove Road).

Furthermore, the project specific model grid resolution is finer than that of the NOAA model, which allows it to resolve the ground elevations in more detail. Ground elevations at the LNG Terminal site in the project specific model are from a high resolution topographic survey. On average the project specific maximum run-up is 2.73 ft. higher than the factored NOAA mapped run-up across the LNG Terminal site. Given that differences between the models are due to differences in ground elevations and model resolution, it is concluded that the tsunami source and wave amplitudes driving the project specific model are generally consistent with the NOAA modeling. Therefore, this demonstrates that the project specific model used to establish maximum run-up elevations is an appropriate tool for deriving the design crest elevations for the proposed LNG Terminal.

Table 6.4-3 Wave Run-up Comparison

Point	Location	Project Specific Modelling Results Tsunami Run-up (ft, NAVD88)	NOAA Mapping Results Tsunami Run-up (ft, NAVD88)
1	North of Ingram Yard	28.5	25.9
10	Ingram Yard	29.8	24.8
20	Inside Marine Basin	32.6	28.4
30	Southeast Corner of Marine Basin	32.8	23.9
40	Roseburg Terminal Yard	24.4	18.8
50	Roseburg Terminal Yard	24.4	19
60	Northeast of Roseburg Terminal Yard	22.8	19.7
70	Southwest of South Dunes	22.7	18.7
80	South Dunes	25.8	18.8
90	South Dunes	23.9	20.3
100	North South Dunes	19.5	22.8
110	North South Dunes	16	22.8
120	Southeast Corner of South Dunes	20.6	18.2
130	East of South Dunes	17.5	15.3
140	Northeast of South Dunes	17.3	15.1

JCEP has demonstrated through a number of different studies that the result of site specific modelling is consistent with the latest NOAA mapping efforts, and that maximum design tsunami run-up elevation for the proposed site topography is no greater than 34.5ft (NAVD88). Therefore, design will ensure minimum grade elevations are typically at or above 34.5 ft. (NAVD88). Furthermore tsunami protection berms, safety critical elements of the facility, point of support elevations, invert levels and underside of essential equipment, will be at least 1ft above the estimated maximum run-up elevation.

Hydrodynamic modelling results showed that the maximum inundation due to the design tsunami occurred at the north end of the marine slip. As such, a subsequent transient seepage analysis was focused on the southernmost LNG tank protective berm at the north end of the marine slip. Given the tsunami elevation, period and the hydraulic conductivity of the tertiary impoundment berms, it was shown that tsunami inundation will not infiltrate through the LNG tank berms into the containment area, refer to the Geotechnical Report (J1-000-GEO-RPT-KBJ-50001-00) provided in Appendix J.13.4 of Resource Report 13.

The previously described tsunami modeling was completed to solicit the elevations required for asset protection. In addition, further modeling was completed to support the design of vertical tsunami evacuation areas in key locations to provide operations personnel the opportunity to evacuate to higher ground. The elevation of vertical evacuation areas was determined by comparing modelled values established using the DOGAMI XXL1 (~10,000 year return period) rupture scenario and the FEMA P646 tsunami refuge height criteria. FEMA P646 determines the run-up elevation given the run-up due a 2475 yr. event +10 feet. The DOGAMI XXL1 rupture scenario was chosen to be consistent with the largest deterministic event considered by DOGAMI in development of the Tsunami Evacuation Maps for Coos Bay and North Bend. As shown in Table 6.4-4 below, the DOGAMI XXL1 elevations are greater than the elevations established in accordance with the FEMA P646 elevations and as such the JCEP project will use the more conservative tsunami refuge height established in accordance with the DOGAMI XXL1 rupture scenario.

Table 6.4-4 Tsunami Evacuation Heights

Refuge Area	FEMA P646 Tsunami Refuge Height (ft. NAVD88)	DOGAMI XXL1 Tsunami Refuge Height (ft. NAVD88)	Minimum Tsunami Refuge Height Adopted by Project (ft. NAVD88)
Access and Utility Corridor	38.5	60.0	60.0
South Dunes	42.0	52.0	52.0

The Kentuck Project site is frequently underwater during wet portions of the year, and the DOGAMI mapping (DOGAMI 2012) confirms that it is located within the estimated tsunami inundation zone. Because no permanent LNG Terminal facilities are planned at the Kentuck Project site, there are no appreciable impacts to the LNG Terminal from tsunami inundation at that site. The existing Trans Pacific Parkway/US-101 Intersection is also located in the tsunami inundation zone. To maintain grades, improvements to the intersection will not remove the intersection from the tsunami inundation zone.

6.4.2 Landslides

The type and occurrence of landslides in the vicinity of the LNG Terminal were evaluated using information gathered though review of geologic maps, literature, aerial photography, Light Detection and Ranging ("LiDAR"), and Statewide Landslide Information Database for Oregon ("SLIDO") (Burns and Watzig. 2014; Burns et al. 2016). Landslides were identified on the hillslopes above the town of Glasgow, Oregon, and on hillslopes above Kentuck Slough. Maps of landslide occurrence and susceptibility are provided on Figures 6.4-2 and 6.4-3 (Burns and Watzig 2014; Burns et al. 2016). No landslide deposits were identified within the LNG Terminal site. On Figure 6.4-3, a moderate to high landslide susceptibility hazard is mapped on the dune ridges at the LNG Terminal site; however, as indicated in Figure 6.4-2, active landslides have not been identified on the sand dunes. The high susceptibility indicated on Figure 6.4-3 at the

LNG Terminal site is primarily based on the steep slopes of the dune deposits. JCEP will regrade the steep dunes thereby eliminating potential landslide hazards related to dune sand stability.

At Kentuck Slough, as shown in Figure 6.4-2, landslides currently exist in isolated areas on the slopes surrounding the area, and Figure 6.4-3 shows a high susceptibility for landslides on the slopes surrounding Kentuck Slough. Because no LNG Terminal structures are planned for this area, the existence of these landslides is not considered significant to the LNG Terminal.

Constructed slopes planned for the LNG Terminal were evaluated for static and seismic conditions, with results and any required mitigation measures presented in the Geotechnical Report (J1-000-GEO-RPT-KBJ-50001-00) provided in Appendix J.13.4 of Resource Report 13.

6.4.3 Ground Subsidence

Ground subsidence can result from co-seismic coastal subsidence, karst terrain, fluid extraction, or underground mining, each of which is described in this section.

6.4.3.1 Co-seismic Coastal Subsidence

Modeling of megathrust earthquake ruptures on the CSZ indicates sequences of interseismic uplift and co-seismic coastal subsidence. The predictions for coastal subsidence are locally constrained by features such as submerged trees and buried intertidal marshes interpreted to be associated with the 1700 CSZ earthquake. This repeated coastal subsidence pattern has been documented along the length of the CSZ (Atwater et al. 1995; Clague 1997; Goldfinger 2003). Leonard et al. (2004) presents profiles of coastal deformations from northern California to southern Canada based on this geologic information. The subsidence information indicates that the largest coastal subsidence, of 3 to 6 feet, occurred in northern Oregon and southern Washington, with subsidence ranging from 0 to 3 feet elsewhere. Leonard et al. (2004) estimated an average of 2 feet of co-seismic subsidence occurred in the Coos Bay area during the 1700 earthquake. Leonard et al. (2004) also estimated that the co-seismic subsidence in the Coos Bay area will range from 0 to about 5 feet during a future magnitude M_w 8 to M_w 9 megathrust earthquake located along this portion of the CSZ.

For the LNG Terminal location and in accordance with more recent tsunami modeling completed for the Southern Oregon Coast (Witter et al. 2011), the estimated subsidence at the LNG Terminal for DOGAMI scenario L1 is of the order of 7.6 feet and for DOGAMI scenario XXL1 of the order of 13.1 feet. More recently, subsidence estimates have been included in the latest release of the ASCE7 (2016). For the LNG Terminal location, subsidence for a design tsunami given a return period of 2475 years and using the parameters given in ASCE7 is in the range of 5.3 to 6.0 feet.

The LNG Terminal will be designed to mitigate inundation due to the design tsunami. For design purposes, design tsunami run-up elevations are established including an allowance for subsidence. As established in Section 6.4.1.4 above, the worst case design tsunami run-up was established using the DOGAMI rupture scenarios and co-seismic subsidence values.

Subsidence estimates reflect deformations over much of coastal Oregon, and will vary over large areas and distances. Over the extent of the Project site, the regional subsidence differential in accordance with DOGAMI (Witter et al. 2011) is estimated to be in the range of 0.01 percent (1.2 foot over 11,600 feet). This level of differential subsidence is less than the 0.1 percent limit (U.S. Army Corp of Engineers, 1990, Fang, H., 1991) given in the Geotechnical Report (J1-000-GEO-RPT-KBJ-50001-00) provided in Appendix J.13.4 of Resource Report 13, and is not anticipated to be damaging to the LNG Terminal site.

6.4.3.2 Karst Terrain

Karst terrain describes a distinctive topography that results from the dissolution of soluble carbonate and evaporite rocks by slightly acidic surface water or groundwater. Karst terrain is characterized by the presence of sinkholes, caverns, and disappearing streams.

Karst terrain has not been identified within the LNG Terminal site on the National Karst Map (Weary and Doctor 2014). Carbonate or evaporate rocks have not been identified within the LNG Terminal site (Beaulieu and Hughes 1975). The hazards associated with karst terrain are not anticipated for the LNG Terminal.

6.4.3.3 Fluid Extraction

Ground subsidence due to fluid extraction can occur when large quantities of subsurface fluids (such as oil, gas, or groundwater) have been withdrawn from certain types of rocks and sediments. Rock and sediments compress as fluid pressure in the material is reduced by pumping activities and effective stress on the rock or sediment increases. As a result of the increased stress, the porosity and volume of porous material is reduced. The reduction in volume decreases the thickness of the unit, which results in subsidence at the ground surface.

Ground subsidence due to extraction for oil and gas is not likely given the lack of commercially viable oil and gas deposits within the LNG Terminal site.

The LNG Terminal site is located on the North Spit of Coos Bay at the southern end of the Dune-Sand Aquifer. Groundwater extraction wells are present on the Siuslaw National Forest Oregon Dunes Recreation Area north of the LNG Terminal site. Three of the Roseburg Forest Product Company (RFP) wells are within the footprint of a proposed construction laydown area on the RFP property. The laydown area will revert back to use by RFP following construction.

The Coos Bay-North Bend Water Board will supply the LNG Terminal site with water during construction and operation. Additional water will be supplied during construction using temporary wells installed at Ingram Yard. Additionally, localized dewatering wells may be used to lower groundwater levels to facilitate soil improvement techniques during site preparation. All on-site construction wells will be operated only for the period of construction; therefore, any impacts to groundwater levels from pumping on-site during construction will be temporary, and water levels will recover when the pumping is terminated.

The Coos Bay-North Bend Water Board uses a portion of the Dune-Sand Aquifer for public water supply; the closest well is approximately 3,500 feet north of the LNG Terminal site (Groundwater Solutions Inc. 2006). Model simulations for the Dune-Sand Aquifer indicate that a maximum of 10 million gallons per day could be pumped with little risk of inducing seawater to flow into the wells (Jones 1992). With no permanent water supply wells planned for the LNG Terminal, there will be no reduction in the groundwater level from pumping at the LNG Terminal site during plant operations. The low risk of inducing seawater from pumping of the Dune-Sand Aquifer at high rates indicates that the aquifer has a high capacity. Therefore, reduction of the groundwater level at the LNG Terminal site is considered unlikely; therefore, ground subsidence is not anticipated to occur.

6.4.3.4 Underground Mining

Ground subsidence may occur in areas where abandoned underground mines that could collapse are located. Abandoned underground mines have not been identified in the vicinity, and therefore, the LNG Terminal will be unaffected by mine subsidence. The hazards associated with underground mines collapse are not anticipated.

6.4.4 Other Natural Hazards

In addition to the seismic hazards, landslides, and ground subsidence hazards, this section discusses other natural hazards, including biogenic gas, volcanoes, extreme wind and hurricanes, and flooding and scour.

6.4.4.1 Biogenic Gas

The borings, soil and rock samples, and laboratory testing associated with the extensive geotechnical investigation of the LNG Terminal site have not identified additional evidence of potential significant biogenic gas sources or potential hazards with natural soil or rock units. The peat layers identified in borings and test pits completed on the South Dunes site and Ingram Yard are relatively thin, have a high percentage of inorganic sand and silts, and are not considered to be a significant potential source of biogenic gas.

6.4.4.2 Volcanoes

The Cascade Mountain Range is the volcanic arc complex of the CSZ and is located approximately 100 miles east of the LNG Terminal site. Rising molten rock from the subducting Juan de-Fuca tectonic slab that erupts at the surface as a volcano can pose a variety of eruptive hazards in the region. Volcanoes of the Cascade Mountains are found from northern California to British Columbia. The eruptive hazards of Cascade Volcanoes include ash fall, pyroclastic flows, lava flows, debris avalanche, and lahars. The nearest Cascade Volcano is the Crater Lake caldera that was formed during the eruption and collapse of Mount Mazama approximately 7,700 years ago. River valleys 44 miles from the summit were destroyed by pyroclastic flows, and ash was deposited to the northeast of the volcano as far as southern Canada (Bacon et al. 1997).

The LNG Terminal site is over 100 miles west of the nearest volcanic hazard area along the Cascade Mountains. At this distance, the pyroclastic flows, lava flows, debris avalanche, and lahars eruptive hazards would not reach the LNG Terminal site. Volcanic ash (tephra) consists of small pulverized pieces of rock and glass ejected during an eruption that could travel by wind in the atmosphere to the LNG Terminal site. Ash is hard, abrasive, and mildly corrosive. Ash has a low density and small particle size and therefore is able to spread over broad areas by wind. The ash begins to fall when the energy needed to keep the particles in the air diminishes. The size of ash particles that fall to the ground generally decreases exponentially with increasing distance from the volcanic vent in the prevailing wind direction (Wolfe and Pierson 1995). Tephra fragments larger than a few centimeters typically do not fall more than a few miles from the vent and are not likely to impact the project area.

Although a future eruption of the Mount Mazama volcano is possible, a large pyroclastic eruption is not considered likely for many thousands of years in the future, because the magma reservoir that fed the eruption of Mount Mazama has not had sufficient time to regenerate a large volume of gas-rich magma (Bacon et al. 1997).

The LNG Terminal site would not be directly affected by the various types of volcanic eruption hazards due to the distance of the hazard, the upwind location of the LNG Terminal site from the volcanic hazard, and the low likelihood of volcanic eruption during the lifetime of the LNG Terminal.

6.4.4.3 Extreme Wind/Hurricanes

Tropical cyclones (i.e., hurricanes) do not occur near the LNG Terminal site. Tropical and subtropical cyclones are by nature barotropic, warm core systems that require sea surface temperatures of 80°F or higher. Sea surface temperatures off of the Oregon coast near the LNG Terminal site average between 50°F and 55°F year-round (National Oceanographic Data Center), which is far too cool to support tropical cyclone formation and/or sustenance. Additionally, tropical cyclones require low deep-layer wind shear environments characteristic of the weak upper-level flow regimes found in tropical latitudes. Given the location of the LNG Terminal site in the mid-latitudes, moderate to strong upper-level westerly winds generally prevail. These upper-level winds contribute to deep-layer wind shear environments that are not conducive to tropical cyclones.

Strong extratropical cyclones (baroclinic, cold core systems) are a common occurrence along the Oregon coast during the late fall and winter, and even into the early spring. Several times every year, winds can reach hurricane force at different locations along the Oregon coast (WRCC 2017). Strong winds that reach hurricane force can and do occur occasionally at the LNG Terminal site.

ASCE 7-05 dictates inputs for structural analysis and design for locations and territories of the United States, and Section 6.5.4.2 states the following:

> In areas outside hurricane-prone regions, regional climatic data shall only be used in lieu of the basic wind speeds given in Fig. 6-1 when (1) approved extreme-value statistical-analysis procedures have been employed in reducing the data; and (2) the length of record, sampling error, averaging time, anemometer height, data quality, and terrain exposure of the anemometer have been taken into account. Reduction in basic wind speed below that of Fig. 6-1 shall be permitted.

This statement remains unchanged in ASCE 7-10, Section 26.5.3.

The LNG Terminal will be designed to maintain structural integrity in the most critical combination of wind velocity and duration having a probability of exceedance in a 50-year period of 0.5 percent (10,000-year mean return interval) per 49 CFR 193.2067 paragraph (b)(2)(ii). This mean return interval exceeds the requirements of ASCE 7. For the LNG Terminal site location, the strength level 10,000-year design wind speed was determined to be 127 miles per hour (3 second gust wind speed, 33 feet, Exposure Category C) (CPP 2016).

6.4.4.4 Flooding and Scour

6.4.4.4.1 Flooding

The LNG Terminal will be designed to mitigate inundation due to flooding by elevating those parts of the LNG Terminal considered critical to the safety and integrity of the facility above the maximum flood elevation. The maximum flood elevation is established considering; estuarine flooding and tsunami. Flash flooding is not considered applicable to the Terminal Site and as such is disregarded. Comparison of the flooding phenomenon applicable to the LNG Terminal shows that the most critical flood elevation is due to the design tsunami inundation.

6.4.4.4.2 Estuarine Flooding

Estuarine flood elevations are established using the Federal Emergency Management Agency (FEMA) flood hazard maps and given consideration to the impact that project specific fill activities could have on these elevations.

By reference to the Flood Insurance Rate Map, Panel 0167E, the existing LNG Terminal site is outside the special flood hazard area (SFHA) and no floodway has been designated for the portion of the estuary adjacent to the JCEP project area. Proposed development to the very southern extremes of the LNG Terminal falls within a special flood hazard area designated AE. A study carried out by SHN (SHN2017) (RR13, Appendix I.13.2) demonstrates that project specific fill activities within this area will add ~0.004 feet to the FEMA mapped flood elevations. The added elevation will have no measurable effect on the estuary nor will it affect flooding elsewhere within the estuary.

In development of the flood hazard maps, FEMA conducted flood insurance studies which included; statistical data for river flow, storm surges, tides, hydrologic/hydraulic analyses, rainfall, and topography. The FEMA developed maps provide the 100-year base flood elevation (BFE) as shown in Table 6.4-5. The BFE provided by FEMA is provided for the still water levels (SWL) and do not include contributions from wind setup or wave action in the estuary. According to the Flood Insurance Study (FIS) for Coos Bay, Oregon dated March 17, 2014, calculations for wind setup suggest the contribution is insignificant, while the increase from wave action will be less than 1 foot.

Table 6.4-5 100 yr. BFE for Ingram Yard and South Dunes (NAVD88)

Flood Type	Elevation (ft) Ingram Yard	Elevation (ft) South Dunes
100 yr. FEMA BFE (Excluding Wind Wave)	11.0	12.0
100 yr. FEMA BFE (Including Wind Wave)	<12.0	<13.0

Further study by SHN (SHN 2017)(RR13, Appendix I.13.2), developed for comparison purposes estimated flooding at the LNG Terminal to be approximately 12.6 feet for the 500-year event, 12.8 feet for the 1000-year event and 13.4 feet for the 10,000-year event. For South Dunes the flood elevations are typically 0.2 feet higher than at Ingram Yard.

6.4.4.4.3 Storm Surge

Although tropical cyclones do not occur at the LNG Terminal site, extreme storms offshore sometimes cause the elevation of the water level along the coastline to raise significantly beyond the normal (predicted) tide levels. This phenomenon is referred to as storm surge.

Extreme water levels are evaluated based on the gauge at Charleston, OR (NOAA/NOS tide gauge number 9432780). The period of record for this gauge is 1978 to present, with data gaps from January 2005 to December 2007. The highest two storm surges (i.e., the greatest super elevations of measured over predicted water levels) were both observed in strong El Niño years: 3.13 feet on 1/26/1983 and 3.06 feet on 12/16/2002. The return period of these two events is estimated at approximately 50 years.

Storm surge is not considered additive to the tsunami inundation height as both storm surge and tsunami are low frequency events. In terms of estuarine flooding, FEMA BFEs already include an allowance for storm surge and as such no modifications of the FEMA BFEs are required.

6.4.4.4.4 Sea Level Rise

Sea level rise associated with global climate change has been observed in various coastal locations. Climate change induced sea level fluctuations are expected to continue in the future. NOAA maintains a website that tracks the potential sea level rise. Based on sea level data from

1970 to 2015 the trend is for sea level to increase at 0.0386 in. / yr. (0.98 mm/year) with a 95 percent confidence interval of ±0.0327 in. / yr. (0.83 mm/year) for Charleston, Oregon, located in lower Coos Bay. Given the worst case of 0.0713 in. / yr. (0.0386 + 0.0327), the equivalent sea level raise over a 50-year period would be 3.565 in. or 0.3 feet.

Safety factors used during tsunami modelling (J1-000-MAR-RPT-MON-00001-00, Appendix I.13.2 of Resource Report 13) cater for potential sea level rise and as such no further allowance for the sea level rise is required. Estuarine flooding heights given above do not include an allowance for sea level rise and as such, the sea level rise as shown here will be added to the BFE as determined in accordance with the FEMA maps.

6.4.4.4.5 Scour

Wave and currents along the coastline can produce forces that scour and undermine coastal soils or structures. The areas of the LNG Terminal with the high potential for scour are identified as follows; access channel, marine slip, Material Offloading Facility (MOF), access route between Jordan Cove Road and South Dunes, and fill material placed on project boundaries (including protective berms).

Where required, the extent of scour protection for the FERC jurisdictional facilities is determined considering tsunami inundation, estuarine flooding, highest measured tide (HMT), sea level rise (SLR), wind wave effects, prop wash and passing ship effect. Areas of the LNG Terminal within the FEMA flood hazard zone are typically at a higher risk of scour. Within the range of the HMT (including an allowance for SLR), a cement based rip-rap or rocks with the durability and mass to resist tidal loads will be used. HMT is given as 10.26ft (NAVD88) in Resource Report 13, Table 13.2-8. Above HMT+SLR, scour protection for less frequent events may be used. The overall height of protection is determined as a function of the FEMA 100-year base flood elevation, an allowance for wind/wave effects, a 1-foot freeboard allowance for critical areas, and the sea level rise over a 50-year period. For the purpose of scour protection, critical areas are defined as areas with infrastructure which is vital to the operation of the LNG Terminal. Table 6.4-6 provides the minimum height established for protection.

Table 6.4-6 Protection Height (NAVD88)

Parameter	Elevation (ft) Ingram Yard	Elevation (ft) South Dunes
100 yr. FEMA BFE (Including Wind Wave) See Table 6.4-5	<12.0	<13.0
Freeboard (For Critical Areas) See Section 6.4.4.4.5	1.0	1.0
Sea Level Rise (SLR) See Section 6.4.4.4.4	0.3	0.3
Minimum Protection Height for Critical Areas	13.3	14.3
Minimum Protection Height for Non-Critical Areas	12.3	13.3

*Note: Scour protection provided to at least HMT+SLR.

Above the protection height established in Table 6.4-6 and at least to a height equal to or greater than the design tsunami run-up, a wind/rain erosion protection system for less frequent events will be used. It is intended that the system used for less frequent events will consist of gravel or an anchored reinforced vegetation system if shown to be suitable. For the anchored reinforced vegetation system, seed beds will use existing site top soil where possible with American Dune Grass or similar native species in order to stabilize the soil.

Effects due to prop wash and passing ship wave will be studied ensuring additional scour requirements over and above those discussed here are provided. For the access channel, given safety and environmental concerns, slope protection is not planned below the mean low water elevation.

6.5 FACILITIES IN SEISMIC RISK AREAS

The Seismic Ground Motion Hazard Study (J1-000-GEO-RPT-KBJ-50002-00) for the LNG Terminal site is provided in Appendix I.13.1 of Resource Report 13.

6.6 PALEONTOLOGY

Fossils are the preserved remains or traces of organisms from the past that are generally older than Holocene age deposits. Vertebrate fossils have been identified at Fossil Point, located 5 miles southwest of the LNG Terminal site. Fossil types encountered include clams, gastropods, and sand dollars. Pieces of whale and sea lion fossils have also been found in this area. Sedimentary rock units identified in the vicinity of the LNG Terminal have a potential for paleontological resources. However, the LNG Terminal will be sited on Holocene and Pleistocene unconsolidated sedimentary deposits with little paleontological resources. Shell fragments have been identified in the unconsolidated deposits, but these fragments have little paleontological value. There are no known paleontological resources that will be impacted by the LNG Terminal.

6.7 GEOTECHNICAL INVESTIGATIONS

Numerous geotechnical investigations have been completed to support the design for the LNG Terminal since 2005. These investigations have been conducted at the LNG Terminal site, the Trans Pacific Parkway/US-101 Intersection, and the Kentuck Project site. The investigations were conducted to evaluate subsurface characteristics and conditions, with investigation techniques including the following:

- Mud-rotary borings with Standard Penetration Tests ("SPT") using hammer energy measurements
- Cone Penetration Test ("CPT") probes with pore pressure measurements
- Test pits
- Soil electrical resistivity testing
- P-S suspension logging, crosshole testing, downhole testing, seismic reflection surveys, and refraction microtremor to measure shear and compression wave velocities
- Borehole pressuremeter testing
- Pump testing
- Laboratory testing

The borings and shear wave velocity logging on the LNG Terminal site were completed to depths of approximately 300 feet. The data from the geotechnical explorations completed to date for the LNG Terminal is provided in the Geotechnical Data Report (J1-000-GEO-RPT-GRI-00033-00) included in Appendix J.13.4 of Resource Report 13. Additional subsurface investigation is planned, including borings, CPTs, PMTs, and geophysical testing to support final design. The Geotechnical Report (J1-000-GEO-RPT-KBJ-50001-00) included Appendix J.13.4 of Resource Report 13 details the proposed investigation to support final design.

The results of the geotechnical investigation to date indicate that subsurface conditions at the site are suitable for the LNG Terminal, provided that adequate site preparation and foundation design and construction methods are implemented.

Liquefaction, subsidence, and tsunami hazards will be addressed. Proposed ground improvement methods include vibro-compaction and deep soil mixing. Either of these methods or a combination of these methods will be used to mitigate the liquefaction, lateral spreading, and seismic slope stability. Seismic slope stability following liquefaction mitigation is addressed in the Geotechnical Report (J1-000-GEO-RPT-KBJ-50001-00) provided in Appendix J.13.4 of Resource Report 13. Permanent slopes require ground improvement to provide stability during the design seismic event. Analyses were performed that considered seismic inertial forces combined with soil strengths following ground improvement. The analysis methodology, acceptance criteria and results are presented in the Geotechnical Report (J1-000-GEO-RPT-KBJ-50001-00) provided in Appendix J.13.4 of Resource Report 13.

Driven pipe piles and drilled shafts are being proposed to support much of the critical equipment and structures. The following discussion summarizes the subsurface conditions and foundation systems planned, with more detailed discussions provided in the Geotechnical Report (J1-000-GEO-RPT-KBJ-50001-00) provided in Appendix J.13.4 of Resource Report 13.

Construction QA/QC procedures will be developed and provided to FERC during detail engineering.

6.7.1 Ingram Yard Subsurface Condition Summary

A number of design profiles were developed for the LNG Terminal site. The design profiles were differentiated based on differences in the amount of fill to be placed or sand dune to be excavated and minor differences in the existing subsurface conditions. These conditions are summarized here, and the design profiles and geologic cross sections are presented in detail in the Geotechnical Report (J1-000-GEO-RPT-KBJ-50001-00) provided in Appendix J.13.4 of Resource Report 13.

Throughout the LNG Terminal site, the subsurface conditions are relatively consistent below elevation -30 feet. The existing sands above elevation -30 feet consist of either existing sand fill or native dune or estuary sand deposits. Historical records indicate that the upper approximately 10 feet of the sand west of the dunes is likely fill. The existing sand fill and underlying native sand are physically indiscernible; however, in some areas, the transition to the native sand can be identified based on the presence of an organic/peat layer at the interface that occurs between elevation 11 feet and elevation 8 feet. The existing sand fill and organics/peat were only identified west of the existing dune. The existing sand fill generally had N_{60}-values (which are the standard penetration test N-values or the number of blows to drive a standard 2-inch split barrel sampler the last 12 inches of an 18-inch sample interval using a 140-pound hammer falling 30 inches, normalized to 60 percent hammer energy) that were less than 10 blows per foot with design N_{60}-values of 7 to 8 blows per foot. The native sand typically had N_{60}-values greater than 20 blows per foot. The fines content of the existing sand fill and native sand generally ranged from 5 to 10 percent.

In the area of the dune on the eastern portion of the Ingram Yard site, the sands are native starting at the ground surface.

Below elevation -30 feet, the native sand is predominantly fine-grained, with occasional shells and silt zones. The fines content of the sand is generally less than 10 percent. Below elevation -30 feet, the native sand density is markedly greater than the overlying sand and has N_{60}-values greater than 50 blows per foot.

A sand-silt unit is present beneath the native sand at elevations ranging from -110 feet to -140 feet. The sand-silt unit density is generally significantly greater than the overlying sand and generally has field N-values of 50 or more in the first 6 inches of the SPT. The sand-silt unit has a fines content ranging from 5 to 85 percent and is non-plastic to low plasticity, with occasional high plasticity.

Borings completed near the south LNG tank for the geotechnical investigation at a depth of about 252 feet on the Ingram Yard site encountered hard clayey silt that was classified as poorly indurated silty shale. Another boring drilled about 480 feet north, did not encounter the poorly indurated silty shale when terminated at a depth of about 280 feet.

6.7.2 Access and Utility Corridor Subsurface Condition Summary

A number of design profiles were developed within the Access and Utility Corridor. The design profiles were developed based on differences in the amount of fill or excavation required for site development and minor differences in the existing subsurface conditions identified above elevation -30 feet. A summary of conditions is provided here, and the design profiles and geologic cross sections are presented in the Geotechnical Report (J1-000-GEO-RPT-KBJ-50001-00) provided in Appendix J.13.4 of Resource Report 13.

The sands consisted of both fill and native sand. Organics and peat were encountered only in the western end of the Access and Utility Corridor between elevation -11 feet and elevation -10.5 feet. The majority of the existing sand fill had N_{60}-values of less than 10 blows per foot. The majority of the native sand N_{60}-values were greater than 15 blows per foot. The fines content for most of the sands ranged from 5 to 10 percent.

Below elevation -30 feet, the conditions for the Access and Utility Corridor are similar to those described for the LNG Terminal site.

6.7.3 South Dunes Site Subsurface Condition Summary

A number of design profiles were developed for the South Dunes site. The design profiles were developed based on minor differences in the existing subsurface conditions identified above elevation -30 feet. The design profiles and geologic cross sections are presented in the Geotechnical Report (J1-000-GEO-RPT-KBJ-50001-00) provided in Appendix J.13.4 of Resource Report 13, and a summary of the subsurface conditions are provided here.

As at Ingram Yard and along the Access and Utility Corridor, the subsurface conditions at the South Dunes site are relatively constant below elevation -30 feet. The conditions above elevation -30 feet vary between design profiles mainly due to variation in the sands and the presence or absence of peat/organics. Peat/organics were encountered in several areas of the South Dunes site at elevations ranging from 4 to 9 feet.

The existing sand above elevation -30 feet consists of fill and native dune and estuary sand deposits. The existing sand fill and underlying native sand were basically indistinguishable; however, the transition between the existing sand fill and native sand was inferred based on the presence of peat/organics, where encountered. The existing sand fill generally has lower N_{60}-values than other areas of the LNG Terminal site. The native sand typically has N_{60}-values greater than 15 blows per foot. The fines content of the sands typically ranged from 5 to 10 percent.

In the northeast quadrant of the South Dunes site, a layer of clay was encountered between elevation 6 feet and elevation 3.5 feet. The clay thickness varies from 0.3 feet to 2.5 feet, and the material is very soft to soft with a high plasticity.

In the east central portion of the South Dunes site, the subsurface investigation encountered driftwood. The estimated extent of the driftwood is based on observations during subsurface investigations and a historical photo that shows accumulations of driftwood in the east central portion of the South Dunes site. The depth to driftwood and the maximum depth of the driftwood are not precisely known, but it is estimated to extend not more than 10 feet below ground surface.

Below elevation -30 feet, the subsurface conditions are fairly consistent across the South Dunes site. The native sand is predominantly fine-grained sand, with occasional shells and silt zones. A deep boring at the South Dunes site indicates that the native sand extends to elevation -151 feet. The native sand in this interval has a design N_{60}-value greater than 50 blows per foot.

Below elevation -151 feet, clayey silt was encountered that extended to an elevation of at least -223 feet, which was the lowest elevation of material classification. Description of the clayey silt from the boring log indicated weak cementation, with hard to very hard consistency based on N_{60}-values ranging from 12 to 75 blows per foot and with a general increase in stiffness with depth. Geotechnical tests on the clayey silt indicated high plasticity.

6.7.4 Groundwater Characterization Summary

The groundwater elevations for the LNG Terminal were estimated using the measured groundwater levels in piezometers along with data from CPT probes. Groundwater recordings indicate the following:

- The groundwater levels fluctuate seasonally in response to rainfall and have varied by up to approximately 4 feet.
- The groundwater levels are typically higher to the north as the distance from Coos Bay increases.
- The tidal influence to groundwater levels decreases as the distance from Coos Bay increases.
- Based on the historical groundwater monitoring, groundwater flow within the LNG Terminal site is generally to the south toward Coos Bay, with localized flow on the west side of the LNG Terminal site toward the Henderson Property.

Design groundwater elevations vary from 7 feet to 17.5 feet depending on the location.

6.7.5 Foundations Summary

A combination of spread or strip footings, mats, and deep foundations will be used to support the structural loads for the LNG Terminal site. Ground improvements discussed in Section 6.6.1.3 will be implemented below structures to mitigate liquefaction.

Miscellaneous structures will be supported by square and strip footings. All spread and strip footings will be founded a minimum depth of 1 foot below final grade based on the anticipated maximum depth of frost penetration. For square and strip footings less than 20 feet wide, settlement design charts are provided in the Geotechnical Report (J1-000-GEO-RPT-KBJ-50001-00) provided in Appendix J.13.4 of Resource Report 13. Larger and/or more heavily loaded foundations will be analyzed on an individual basis.

Shallow mat foundations will be used to support the LNG tanks on the ground surface. A base isolation friction pendulum bearing system is planned for the LNG tanks, with the isolators installed on top of plinths extending up from the shallow mat foundation. The tank slabs will be installed on the isolators, and will support the outer tank and inner tank.

The use of deep foundations will be required for structures and equipment that cannot satisfy the bearing capacity requirements, settlement limitations, lateral and/or uplift load requirement, and/or economics of shallow foundations. Deep foundations are primarily required to resist seismic shear and overturning, or wind loads. The deep foundations to support large compression, tension, and/or lateral loads will be pipe piles or drilled shafts depending on the required vertical or horizontal loading requirements.

The compression and tension axial capacity of pipe piles and drilled shafts are calculated with a safety factor of 2.0; therefore, full-scale compression and tension load tests are required to be performed on-site for pipe piles and drilled shafts. If load testing is not performed, the factor of safety will be increased to 3.0.

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FIGURES

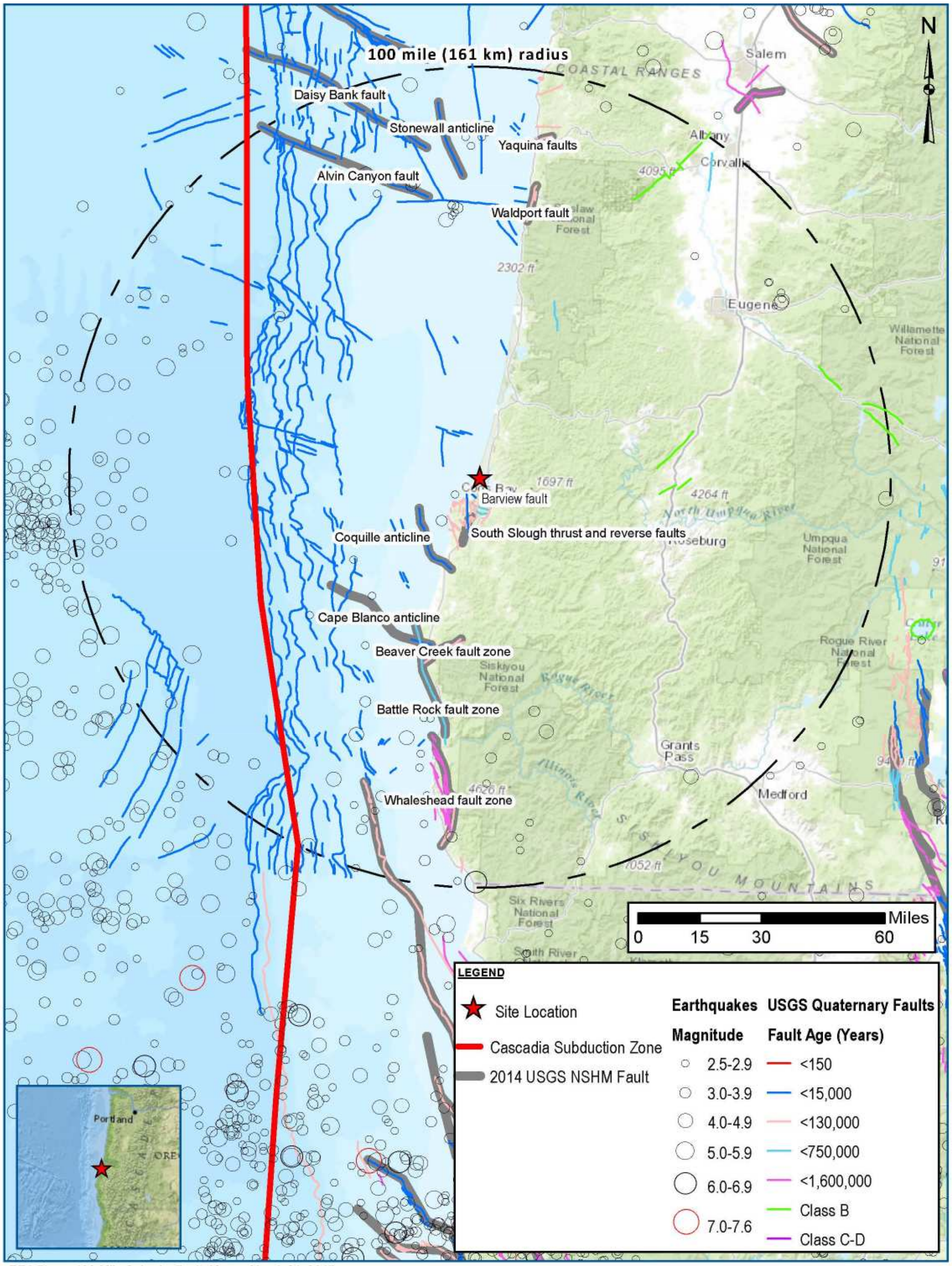

Figure 6.4-1: Potentially Seismogenic Fault Locations within 100 miles of the LNG Terminal Site

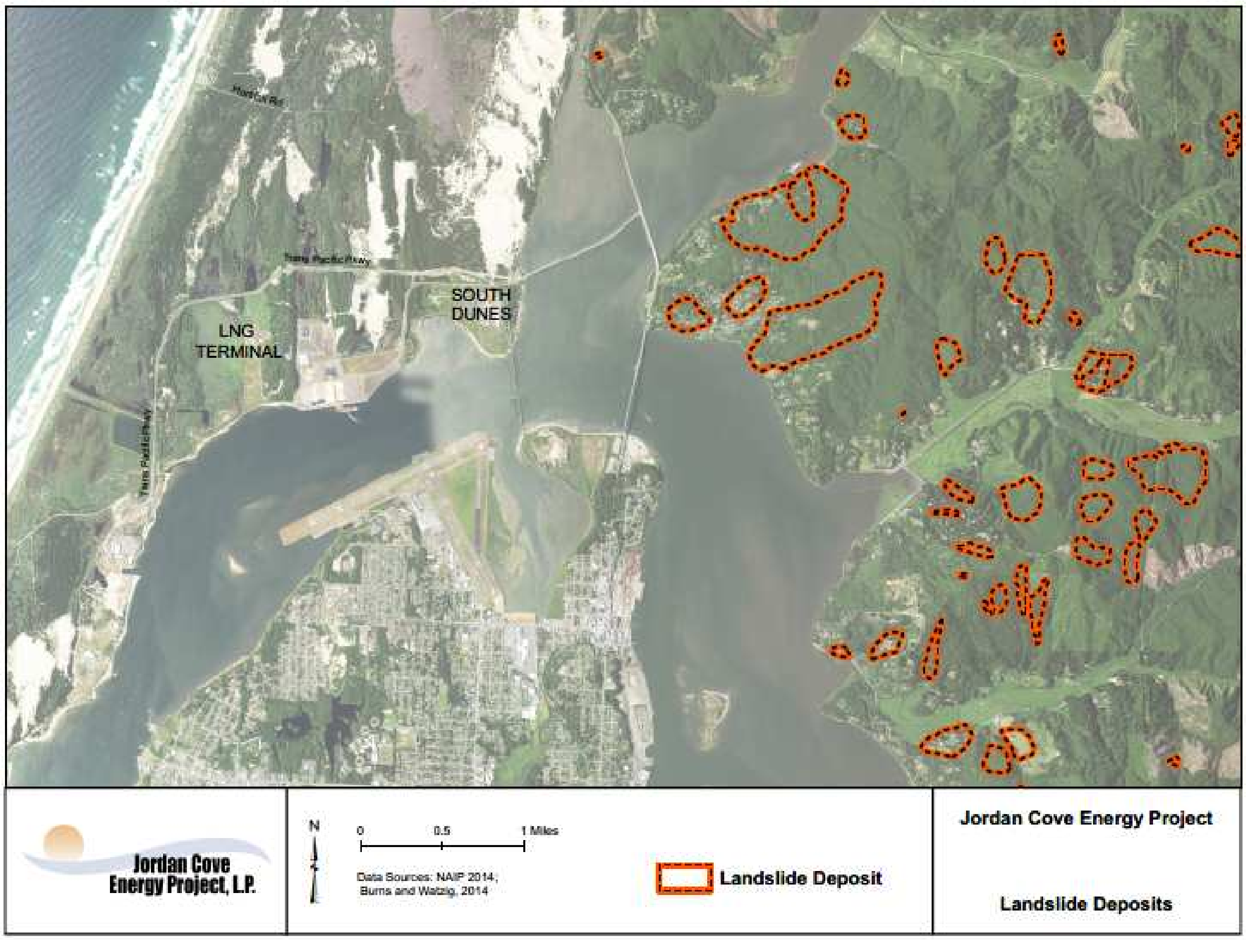

Figure 6.4-2: Mapped Landslide Deposits

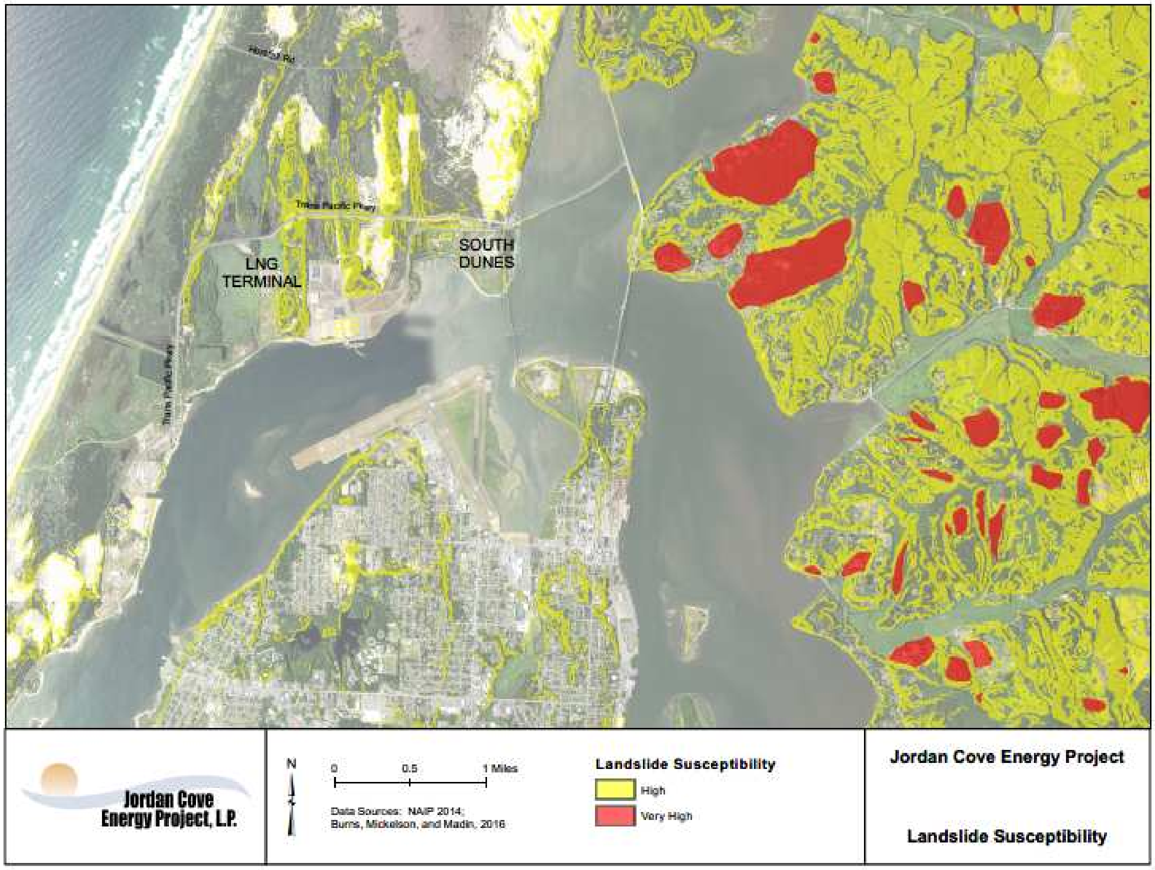

Figure 6.4-3: Landslide Susceptibility in the Vicinity of the LNG Terminal Site

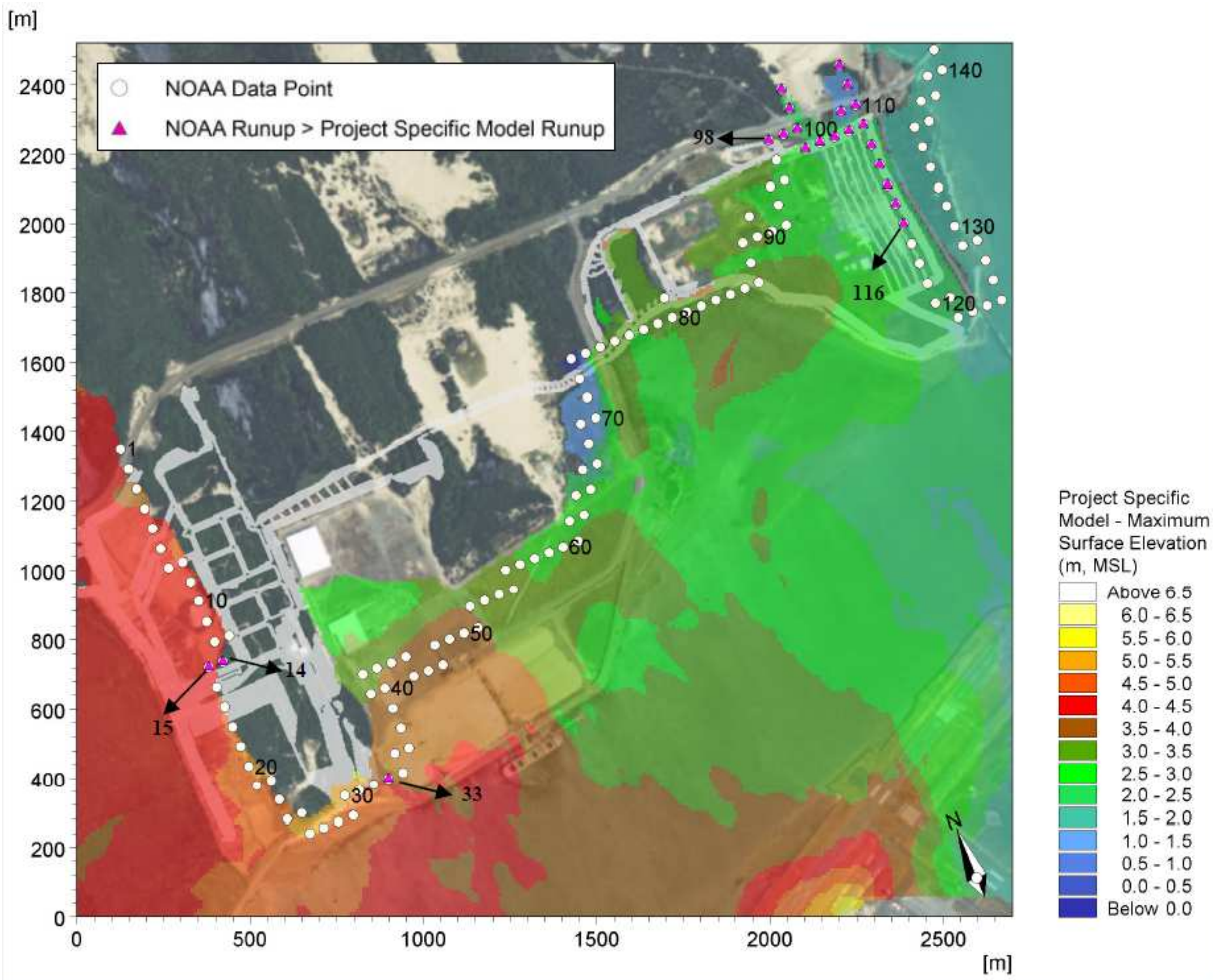

Figure 6.4-4: Project Specific Model Run-up and NOAA data points.

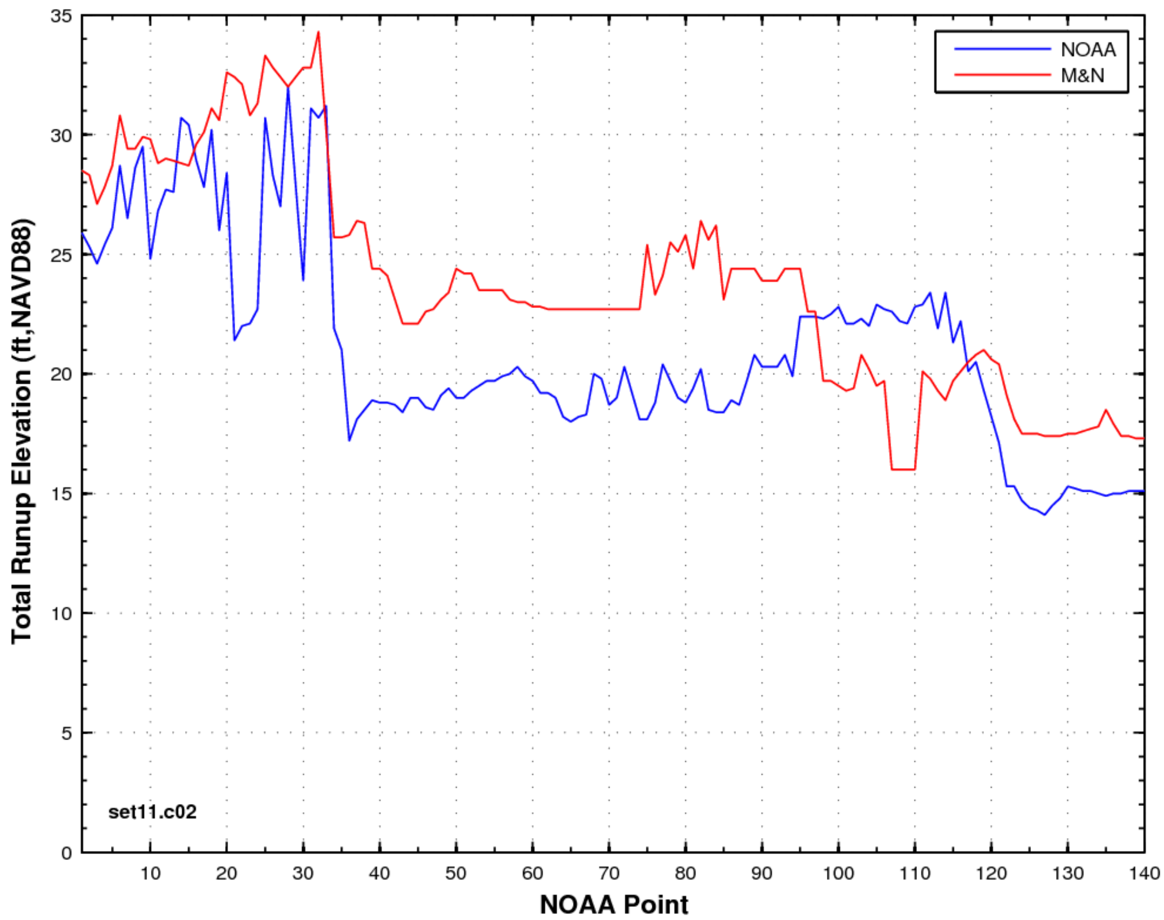

Figure 6.4-5: Project Specific Model Run-up vs. NOAA Run-up.