Exhibit H

Geologic Hazards Evaluation

Mist Resiliency Project
August 2024

Prepared for

NW Natural

Northwest Natural Gas

Prepared by

GeoEngineers

4000 Kruse Way Place
Building 3, Suite 200
Lake Oswego, Oregon 97035
503.624.9274

REGISTERED PROFESSIONAL ENGINEER
86240PE
Jerad Hoffman Digitally signed by Jerad Hoffman Date: 2024.08.07 09:18:35 -07'00'
OREGON
JUN 30, 2017
JERAD A. HOFFMAN
EXPIRES: 06/30/2025

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OREGON
BRIAN C. RANNEY
Brian C. Ranney Digitally signed by Brian C. Ranney Date: 2024.08.07 09:27:49 -07'00'
No. E1917
ENGINEERING GEOLOGIST
Expires 12/1/2024

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Acronyms and Abbreviations

ASCE	American Society of Civil Engineers
ATC	Applied Technology Council
bgs	below ground surface
CRB	Columbia River Basalt
CRBG	Columbia River Basalt Group
CSZ	Cascadia Subduction Zone
DEM	Digital Elevation Model
DOGAMI	Oregon Department of Geology and Mineral Industries
GeoEngineers	GeoEngineers, Inc.
GIS	Geographic Information System
IBC	International Building Code
I/W	Injection and Withdrawal
LiDAR	Light Detection and Ranging
MCE	Maximum Considered Earthquake
MMI	Modified Mercalli Intensity
MSL	Mean Sea Level
NMCS	North Mist Compressor Station
NW Natural	Northwest Natural Gas Company
OAR	Oregon Administrative Rule
ORWD	Oregon Water Resources Department
OSBGE	Oregon State Board of Geologist Examiners
PGA	Peak Ground Acceleration
PSHA	Probabilistic Seismic Hazard Analysis
ROW	Right-of-Way
SLIDO	State Landslide Information Database of Oregon
USGS	U.S. Geological Survey

1.0 Introduction

Exhibit H provides information about the geological and soil stability for the Mist Resiliency Project (Project). This exhibit demonstrates that the Project can comply with the Oregon Administrative Rules (OAR) 345-021-0010(1)(h) and the approval standards in OAR 345-022-020(1). This Exhibit was revised to address Oregon Department of Energy comments provided to NW Natural.

2.0 Project Description

As described in Request for Amendment No. 13, Northwest Natural Gas Company (NW Natural) proposes to amend the Mist Underground Natural Gas Storage Site Certificate for its underground natural gas storage facility in Columbia County, Oregon through the Project. In the Request for Amendment No. 13, NW Natural proposes to complete the following upgrades at the Miller Station and North Mist Compressor Station (NMCS).

- **Miller Station –** upgrade and replace the two existing natural gas turbine driven natural gas compressors with clean burning turbine driven natural gas compressors, and upgrade and replace the existing electric power supply line from its origin at Highway 202 to Miller Station. An approximately 7.5-acre area just north of the existing compressor station, called the Miller Station Storage Area will be graded as a permanent gravel surfaced equipment and materials staging area for Miller Station.
- **NMCS –** develop the existing Newton, Medicine, and Stegosaur underground storage reservoirs, install injection and withdrawal (I/W) wells and I/W pipelines to connect the underground storage reservoirs to the existing NMCS, install three reciprocating engine driven natural gas compressors, install two natural gas dehydration equipment systems, and construct a control and operations building to facilitate manned operations at the facility.

Development of the Newton, Medicine, and Stegosaur well pads will include grading, construction of I/W wells, and a finish grade rocked surface. The approximate location of the proposed well pads is shown with respect to topography and the surrounding area in Figure H-1. Table H-1 summarizes development at each well pad.

Table H-1. Proposed Well Pad Development Summary

Well Pad Name	Description	Proposed Grading[1]	Proposed Well Development
Newton	New well pad development on clearcut forest land.	Grading plans not developed at this time. Assumed cuts and fills up to 10 feet may be required to prepare the well pad.	Construct new 90,000 square foot well pad; Install injection and withdrawal wells.

Well Pad Name	Description	Proposed Grading[1]	Proposed Well Development
Medicine	New well pad development on clearcut forest land.	Grading plans not developed at this time. Assumed cuts and fills up to 20 feet may be required to prepare the well pad.	Construct new 90,000 square foot well pad; Install injection and withdrawal wells.
Stegosaur	Expansion of existing well pad on clearcut forest land.	Grading plans not developed at this time. Assumed cuts of up to 20 feet may be required to expand the well pad.	Expand existing approximately 43,000 square foot well pad to approximately 160,000 square feet.; install pipeline collection headers.
1. Approximate grading based on site topography. Final grading requirements to be determined by production of a site-specific grading plan.			

New arc-welded steel I/W pipelines that will be up to 16-inches in diameter will be constructed using conventional open trench methods to move natural gas between the well pads discussed above and the NMCS. The pipelines are termed the Newton to Stegosaur I/W Pipeline, the Medicine to Stegosaur I/W Pipeline, and the Stegosaur to NMCS I/W Pipeline. Most of the pipelines will be routed to follow existing logging roads. However, the Newton to Stegosaur I/W Pipeline and the Stegosaur to NMCS I/W Pipeline will traverse undeveloped forest land and/or follow an existing pipeline right-of-way (ROW). The proposed pipeline routes are shown with respect to topography and the surrounding area in Figure H-1. Table H-2 below summarizes the proposed pipelines.

Table H-2. Proposed Pipeline Summary

Name (Proposed Pipeline Diameter)	Approximate Length (Feet)	Generalized Route Description
Newton to Stegosaur (Up to 16-inches)	4,084	Traverses gentle to moderate sloping clearcut and forested slopes cross country from Newton Well pad to existing rock pit located about 1,300 feet west of the well pad, and then follows an existing logging road to the Stegosaur Well Pad.
Medicine to Stegosaur (Up to 16-inches)	6,407	Follows existing logging roads.
Stegosaur to NMCS (Twin pipelines up to 16-inches)	3,168	Traverses moderate sloping clearcut slope for approximately 300 feet, follows an existing logging road for about 440 feet, traverses gentle to moderate sloping clearcut slopes for about 350 feet, then follows the existing North Mist Expansion Pipeline ROW across gentle to moderately sloping clearcut slopes for about 1,400 feet to the NMCS.

A new, approximately 8,760-foot-long underground powerline will be constructed between Highway 202 and Miller Station. The majority of the powerline conduit will be installed using conventional open trench methods within the fill prism of the gravel surfaced Mainline Road, and along NW Natural's South Mist Pipeline ROW. However, two horizontal directional drilling (HDD) installations will be completed to cross Lindgren Creek and Lyons Creek. The HDD installations will be roughly 400 to 600 feet long. The proposed powerline route is shown with respect to topography and the surrounding area in Figure H-1.

3.0 Geologic Report – OAR 345-021-0010(1)(h)(A)

> *OAR 345-021-0010(1)(h) Information from reasonably available sources regarding the geological and soil stability within the analysis area, providing evidence to support findings by the Council as required by OAR 345-022-0020, including:*
>
> > *(A) A geologic report meeting the Oregon State Board of Geologist Examiners geologic report guidelines. Current guidelines must be determined based on consultation with the Oregon Department of Geology and Mineral Industries, as described in paragraph (B) of this subsection;*

GeoEngineers, Inc. (GeoEngineers) performed an evaluation that satisfies OAR 345-021-0010(1)(h)(A), which requires a geologic report meeting current Oregon State Board of Geologist Examiners (OSBGE) geologic report guidelines (OSBGE 2014). In accordance with OSBGE's guidance, only those topics recommended by OSBGE's guidelines that pertain to the Project are addressed in this report (OSBGE 2014).

GeoEngineers conducted several reconnaissance of the Mist Resiliency Project's proposed injection and withdrawal (I/W) pipeline routes, powerline route, well pads, NMCS, Miller Station and selected landslides on several dates. Brian Ranney, Certified Engineering Geologist of GeoEngineers conducted a reconnaissance of the proposed Newtown Well Pad on March 9, 2023 in association with a preliminary geologic hazard evaluation for siting of the well pad (GeoEngineers 2023a). Brian Ranney and Andrew Bauer, RG of NW Natural conducted a reconnaissance of the Newton to Stegosaur I/W Pipeline route, NMCS and Miller Station on May 18, 2023 to assess preliminary routing of the pipeline and observe potential geologic hazards at or near the NMCS and Miller Station. Brian Ranney and Brandi Petryk, LG of GeoEngineers conducted a reconnaissance of the Medicine to Stegosaur I/W Pipeline route, the Medicine Well Pad, Stegosaur to NMCS I/W Pipeline Route and selected landslides on July 6, 2023. Brian Ranney conducted a site reconnaissance of the Miller Station Storage Area and Newton Well Pad site on October 23, 2023.

4.0 Geological and Topographic Features

Geologic and topographic conditions within the Project Area were initially evaluated using the following reference materials:

- Publications, including oil and gas investigations (Newton and Van Atta 1976; Niem and others 1990; Van Atta and Kelty 1985; Warren and Norbisrath 1946), published state of Oregon geological literature (Madin and Burns 2013; Niem and others 1976 and 1994;

Niem and Van Atta 1973; Orr and Orr 1999; Schlicker and others 1972; Wells and Bemis 2020), Oregon Department of Geology and Mineral Industries (DOGAMI) and United States Geological Survey (USGS) bulletins and special papers (Niem and others 1994; Walker and MacLoed 1991), unpublished masters theses and doctoral dissertations (Eriksson 2002; Kadri 1982; Kelty 1981; Ketrenos 1986; Van Atta 1971), and the applicable county soil survey (Smythe 1986; NRCS 2023);

- State water well logs obtained from the Oregon Water Resources Department (ORWD 2023);
- Geologic and topographic maps, including the State of Oregon Geological Map (DOGAMI 2015, 2020 and 2021; Walsh 1987), oil and gas investigation mapping (Newton and Van Atta 1976; Niem and others 1990; Van Atta and Kelty 1985; Warren and Norbisrath 1946) unpublished mapping completed with doctoral dissertations and masters theses (Eriksson 2002; Kadri 1982; Kelty 1981; Ketrenos 1986; Van Atta 1971) and the applicable topographic quadrangle for the Project Area (USGS 1979);
- Light Detection and Ranging (LiDAR) based Digital Elevation Models (DEM) of the site; and
- Reports from GeoEngineers' files (GeoEngineers, 1999a, 1999b, 2001, 2003a, 2003b, 2003c, 2016; 2017a, 2017b; 2023a, 2023b, 2023c, 2023d, 2023e, 2023f).

4.1 Geologic Conditions

4.1.1 Geologic Setting

The Project is located within mountainous terrain of the Oregon Coast Range. In Oregon, the Coast Range is a belt of moderately high mountains, extending along a north-south axis between the Columbia River and the Klamath Mountains. The core of this anticlinal structural chain is underlain by early Tertiary aged pillow basalts, lavas, and basalt breccias that erupted underwater as oceanic islands. The flanks of the coast range are composed of marine sedimentary rocks that accumulated around the underwater oceanic islands. The volcanic and sedimentary rocks were later accreted onto the western edge of the North American continent by the subduction of the Juan de Fuca tectonic plate (Orr and Orr 2000). During and after this accretion, the Columbia River cut through the Oregon and Washington Coast Range forming a river valley and associated broad alluvial plain that forms the northern border of the Oregon Coast Range Mountains.

Because of the presence of natural gas in economic quantities, Columbia County has been subject to several generations of geologic research. The understanding of rock units and structures has progressed from the earlier work of Warren and Norbisrath (1946); to more intensive study in the 1970s by Van Atta (1971), Niem and Van Atta (1973), and Newton and Van Atta (1976); through the master's theses of Kelty (1981), Kadri (1982), and Ketrenos (1986); and more recently to the compilations of Niem and others (Niem et al. 1990a, Niem et al. 1994). Geologic mapping has been aided by the large number of wells drilled and geophysical surveys conducted in support of natural gas exploration.

4.1.2 Stratigraphy

In the Mist area, basement rocks of the upper to middle Eocene-aged Tillamook Volcanics (chemically equivalent to the Grays River Volcanics in the site area [Neim et al. 1990b]), which are remnants of a large mid-ocean volcanic complex, are overlain by several thousand feet of marine sedimentary rocks deposited on the emerging continental shelf. Deep in that sequence, shallow-marine to deltaic sandstones of the Cowlitz Formation (upper Eocene-aged) are the primary hydrocarbon reservoir rocks. Fine-grained sediment layers in the upper Cowlitz and the overlying Keasey Formation form the cap to the reservoir rocks. These sediments are in turn overlain by the Grays River Volcanics. Shallow marine sedimentary rocks of the Pittsburg Bluff and Scappoose formations overlie the Grays River Volcanics. Basalts belonging to the Columbia River Basalt Group (CRBG) overlie and are interfingered with the marine sedimentary rocks.

Geologic maps of the Project Area are provided in Figures H-2 and H-3. GeoEngineers utilized geologic mapping produced by the DOGAMI Oregon Geologic Data Compilation (OGDC) mapping, release 7 (DOGAMI 2020) to evaluate and describe geologic materials in the near vicinity of the Project. GeoEngineers also reviewed geologic mapping by Newton and Van Atta (1976), which is more detailed mapping, to inform our assessment of geology in the area.

Based on the referenced mapping, the Project Area is underlain chiefly by five major geologic units:

- Quaternary Landslide Debris (QLS) – Mapped by DOGAMI (2020) but not by Newton and Van Atta (1976).
- Columbia River Basalt (CRB) (Tco) – Mapped underlying the Project Area by Newton and Van Atta (1976), but not by DOGAMI (2020). However, the CRB is mapped elsewhere in the Mist area by the DOGAMI.
- Scappoose Formation (Tso, TA).
- Pittsburg Bluff Formation (TPs, TPI).
- Keasey Formation (TK).

In general, the oldest mapped rocks within the site area belong to the Keasey Formation. These marine sedimentary rocks are overlain by the Pittsburg Bluff Formation rocks. The younger Scappoose Formation rocks are valley fill sediments cut into the older Pittsburg Bluff Formation. The CRB overlies, and in some cases is interfingered with these sedimentary rocks. The DOGAMI geologic mapping shown in Figures H-2 and H-3 does not map the CRB rocks at the surface where the Project infrastructure will be located; however, they were encountered in borings completed at the NMCS for this project. Quaternary-aged landslide debris is mapped by the smaller scale DOGAMI mapping within a large region including the Newton Well pad, Stegosaur Well pad, NMCS, and all three proposed pipelines, and is not associated with discrete landslides. However, the smaller larger scale Newton and Van Atta (1976) mapping does not map this landslide debris underlying the proposed Project infrastructure. These geologic materials are described in more detail in Section 4.1.4.

4.1.3 Geologic Structure

The Mist area, including the Project site, is located on the Nehalem Arch, a high area formed in the basement Tillamook Volcanics connecting the Willapa Hills and Northern Coast Range uplifts (north and south, respectively), and separating the sediment-filled Nehalem and Astoria forearc basins (east and west, respectively; Niem et al. 1994). The Mist area is a relatively low saddle in the Nehalem Arch. In the Miocene epoch flood basalts of the Columbia River Basalt Group (CRBG) and sandstone and siltstone sediments of the Scappoose Formation were transported west when the Mist Saddle area was still low enough to receive these Scappoose sediments and flood basalts. The latest uplift of the Coast Range occurred in the late Neogene period.

Numerous faults have been identified in the Mist area; many are older faults dating from a late Eocene (pre-Keasey Formation) period of tectonism and are not exposed at the surface. The closest active fault mapped by the USGS (2023a) is the Gales Creek Fault Zone, which is located approximately 20 miles south of the site. This fault may have been active as recent as Holocene time (Wells 2018; Wells et. al. 2020). A series of mostly northwest-southeast and west-east trending normal faults cut across the Nehalem Arch, forming the Nehalem graben, generally coincident with the Nehalem River valley between the cities of Mist and Birkenfeld (Niem et al. 1990a). Three of these such faults are mapped near the southern end of the powerline alignment as shown in Figure H-3. Disruption of rock layers along faults causes zones of weakness that are exploited by erosion, commonly becoming stream valleys; a fault seems to be responsible for the valley of Lindgren Creek near the Project Area (Ketrenos 1986), also as shown in Figure H-3. Based on a review of active fault mapping by the USGS (2023), these faults are not active Quaternary faults. In addition, a review of a LiDAR hillshade model did not reveal lineaments, offset stream channels, truncated mountain faces or other disruption of Quaternary-aged alluvium at the mapped fault locations.

In general, major strata in the area are only gently deformed. Ketrenos (1986) stated that dips in bedding planes in the younger rocks are generally about 5 to 10 degrees to the northwest, whereas mapped dips in the older strata can be up to about 30 degrees (Newton and Van Atta 1976, Kelty 1981). But attitudes can change within short distances, particularly around faults. The extensive old faulting in the area has also probably contributed to local fault-zone deformation.

4.1.4 Site Geology

The following paragraphs describe the major geologic units and their mapped locations relative to the proposed Project infrastructure.

4.1.4.1 Landslide Debris (Quaternary)

Quaternary aged landslide debris (Qls) is mapped extensively in the mountainous terrain north of Highway 202 by DOGAMI (2020). The proposed Newton and Stegosaur well pads are situated on this mapped unit as well as the NMCS. The proposed pipelines between Newton and Stegosaur, Stegosaur to NMCS and Medicine to Stegosaur (ending at Pipeline Station 59+00) are also situated on this mapped unit, as shown in Figure H-2. Qls is described as mixed grain sediments derived from landslide deposition (DOGAMI 2020). Mapping of the Astoria and Ilwaco quadrangles (Walsh 1987) was the source for this mapping. Walsh (1987) describes the Qls deposits as "large deposits of mass wasting, from surface creep to coherent glide and slump blocks; many small slides not shown." However, more detailed larger scale mapping included in Newton and Van Atta (1976) shows the mapped area of landslide debris as being underlain by Columbia River Basalt. In addition, Newton and Van Atta (1976) map sandstone and siltstone units including the Scappoose and Pittsburg Bluff formations flanking the Columbia River Basalt.

Based on GeoEngineers' interpretation of a LiDAR generated hillshade model of the area, and borings conducted for the Project, the mapped extent of the landslide debris does not accurately represent discrete landslide deposits, and the broadly mapped area of landslide debris is likely underlain by basalt of the CRBG and the Scappoose Formation sediments. GeoEngineers' interpretation of landslides based on site-specific LiDAR hillshade models is shown in Figures H-4 and H-5 and discussed in more detail in Section 9.3 of this report.

4.1.4.2 Columbia River Basalt (Middle Miocene)

Basalts belonging to the CRBG (Tco), are mapped approximately 3,400 feet northeast of the Medicine to Stegosaur I/W Pipeline alignment and Medicine Well Pad, as shown in Figure H-2. Newton and Van Atta (1976) map the CRB underlying the well pads, pipelines, NMCS, Miller Station and most of the proposed powerline alignment. Although DOGAMI does not map the CRB underlying the proposed pipelines, NMCS, the Miller Station, well pads or powerline alignment, the CRB description has been included here because of the Newton and Van Atta mapping and borings completed for the NMCS (GeoEngineers 2023b) encountered CRB interfingered with weathered Scappoose Formation sediments.

The CRBG includes several sub-aerial basalt flows erupted from fissures near the Oregon-Idaho-Washington border. The individual flows within the group have been differentiated by many geologic studies and mapping projects in the Pacific Northwest. In general, the CRBG rocks consist of dark-gray to black, aphanitic basalt with some localized breccias and pyroclastics. The CRBG overlie and are interbedded with sandstone and siltstone of the Scappoose and Pittsburg Bluff Formations. Based on our experience, and borings completed for the Project, the CRBG rocks are often deeply weathered to clay, sand, and gravel.

4.1.4.3 Scappoose Formation (Early to Middle Miocene)

The Astoria Group of the Scappoose Formation (TA, Tso,) is mapped at the Medicine Well Pad, and approximately between Pipeline Stations 59+00 and 64+00 along the Medicine to Stegosaur I/W Pipeline, and the Miller Station Storage Area. The Astoria Group includes shallow marine micaceous sandstone and carbonaceous siltstone that have been mapped and described in various geologic maps and texts (Schlicker et al. 1972, Niem et al. 1990a). The Scappoose Formation includes fluvial, lacustrine, deltaic, and estuarine facies that generally represent a large valley fill deposited on an eroded surface cut into Pittsburg Bluff and older rocks.

4.1.4.4 Pittsburg Bluff Formation (Oligocene)

The Pittsburg Bluff Formation includes a siltstone member (TPs) and a laminated member (TPI). TPs is mapped at the Miller Station, the Miller Station Storage Area and approximately between Stations 0+00 and 40+00 of the proposed powerline alignment. The TPI member is mapped between approximate Stations 40+00 and 50+00 of the powerline alignment. The Pittsburg Bluff rocks are typically tuffaceous and arkosic sandstones, locally glauconitic and fossiliferous, with lesser tuffaceous siltstone, claystone, and coal. They were deposited in marine to deltaic waters that appear to have been becoming shallower with time; ultimately, the area rose above sea level, and there is an erosional unconformity between the top of the Pittsburg Bluff Formation and overlying strata.

4.1.4.5 Keasey Formation (Eocene)

The Keasey Formation (TK) is mapped as underling the proposed powerline alignment between Stations 50+00 and 84+00 north of the Nehalem River. The Keasey Formation is typically comprised of tuffaceous marine sedimentary rocks (Kadri 1982). These rocks make up the river terraces along the Nehalem River.

4.1.4.6 Unmapped Alluvium

It is expected that alluvium associated with the Nehalem River would overlie the bedrock of the Keasey Formation within the Nehalem River Valley. Based on a review of two well logs completed within the Nehalem River Valley, one of which was located within about 600 feet of the powerline alignment, and four borings conducted by GeoEngineers within the Nehalem River Valley, the Keasey Formation Bedrock is overlain by approximately 20 and 30 feet of sand, clay, and gravel alluvium (OWRD 2023).

4.1.4.7 Site Reconnaissance Observations

Geologic reconnaissance of the proposed pipeline routes, well pads and powerline alignment generally confirmed the stratigraphy described above, with the exception of unmapped fills associated with roads and within the NMCS. In road cuts along the proposed alignments, the rocks were observed to be completely decomposed to a tan sandy clay or clayey sand soil with gravel to sized sandstone and siltstone rock fragments in the soils, which likely represent the Scappoose or Pittsburg Bluff Formations. GeoEngineers also observed some road cuts in the Project Area where reddish-brown sandy clay or clayey sand soil with predominantly decomposed basalt gravel was observed, likely representing weathered CRBG rocks. A face of a quarry located near Station 25+00 of the Newton to Stegosaur pipeline route revealed between about 10 and 20 feet of Scappoose or Pittsburg Bluff Formation sediments overlying and also in vertical contact with CRB. GeoEngineers interpreted this outcrop to show interfingered sedimentary and basalt rocks and a valley fill basalt within the eroded Scappoose/Pittsburg Bluff surface.

GeoEngineers also observed areas near the pipeline alignment where portions of the gravel road are likely composed of artificial fill. Exposures of the fill to characterize the fill soils were not observed; however, as is typical in forest logging road construction, the fill is most likely composed of weathered Scappoose or Pittsburg Bluff Formation sediments (sandy clay, clayey sand, with gravel) that were cut from the inboard edge of the road and placed on the outboard edge of the road in specific locations. Indications of instability of the road fills (existing landslides, ground cracks, sunken road grades) that would indicate the road fills are unstable were not observed during site reconnaissance. Due to the scale of the geologic mapping included in this report GeoEngineers did not map areas of artificial fill along the pipeline alignments or at the NMCS.

Cut slopes revealing Scappoose Formation sediments and surface indications of artificial fill were observed at the NMCS and nearby Adams Well Pad. Cuts within the NMCS and adjacent Adams well pad revealed Scappoose Formation sediments composed of predominately decomposed siltstone and sandstone that is weathered to a sandy clay to clayey sand soil. Surface instability of these cut slopes was not observed during site reconnaissance. Surface indications of fill soils were observed covering most of the NMCS. The fill was reportedly placed in this area during construction of the existing NMCS facility. Borings completed for a geotechnical evaluation of the proposed NMCS construction area (GeoEngineers 2023b) confirmed that fill, ranging between about 5 and 9 feet thick, covers most of the site. The fill is composed of brown sandy silt with organic matter (wood chips and other organics), brown to dark brown silt with organic matter and brown-black silty sand with organic matter. These borings also confirmed that the site is underlain by weathered Scappoose Formation sediments that are interfingered with decomposed to fresh CRB.

4.1.5 Geologic Unit Stability

In general, geologic units within the Project Area are prone to landsliding, as is typical in the northern Oregon Coast Range. Based on the distribution of mapped landslides in the area (See Figures H-4 and H-5), the sedimentary units (Pittsburg Bluff Formation; Scappoose Formation) appear to be less stable than volcanic geologic units (CRB) mapped within the mountains near and outside of the Project Area. Most of the landslides mapped in the Project Area are associated with drainage slopes greater than 50 percent or occur at the contact of sedimentary units and the overlying volcanic units, where differential erosion leads to the oversteepening of slopes within the sedimentary units. Areas where slope gradients that are less than 50 percent, such as ridge tops, appear to be the most stable slopes within the Project Area.

Indications of instability of the road building associated artificial fill soils (ground cracking, existing landslides, sunken road grades) were not observed along the pipeline alignments. However, a road fill failure landslide was observed near the Miller Station Storage Area as discussed later in this report. Based on GeoEngineers' experience, artificial fill placed during cut/fill logging road construction methods is typically less stable than the cut sides of the road and natural geologic materials.

4.1.6 Soil

Shallow subsurface soil conditions in the Project's vicinity are identified by the U.S. Department of Agriculture, Natural Resources Conservation Service web soil maps (NRCS 2023) and the Soil Conservation Service Soil Survey of Columbia County (Smythe. 1986). The survey describes soil conditions in the upper 5 feet of the subsurface profile and classifies land use. Eleven soil units were identified by the Soil Conservation Service within the Project Area. A general description of each soil unit is provided in Table H-3 below. Exhibit I provides a more detailed assessment of soil conditions within the Project Area.

Table H-3. Soil Unit Descriptions

Soil Unit	Setting Within Project Location	Approximate Thickness	Formation Setting	Permeability	Runoff	Hazard for Erosion
Alstony Gravelly Loam	Moderate to steep slopes at higher elevations near ridge tops	2 feet	Colluvium derived from volcanic rocks and ash	Moderate	Very Rapid	High
Anunda Silt Loam	Gentle ridge top	4 feet	Colluvium derived from siltstone and mixed with volcanic ash	Moderately high to high	Medium	High
Braun-Scaponia Silt Loam, 5-30 percent slopes	Gentle to steep, active and stable, convex slopes	2.5 feet	Colluvium derived from siltstone	Moderate	Medium to rapid	High
Braun-Scaponia Silt Loam, 60-90 percent slopes	Steep stream channel banks	3.5 feet	Colluvium derived from siltstone	Moderately high to high	Medium	High
Eilertsen Silt Loam	Stream terraces	4 feet	Mixed alluvium	Moderately high to high	Medium	High
Hapludalfs-Udifluvents Complex	Gentle, concave slopes and side slopes	5 feet	Colluvium derived from volcanic rocks and sediment	Moderate	Medium to rapid	High
Murnen Silt Loam	Gentle to moderate, ridge tops and side slopes	4 feet	Colluvium and residuum derived from basalt mixed with volcanic ash	Moderate to high	Medium to rapid	Moderate to high
Natal Silty Clay Loam	Stream terraces	4 feet	Alluvium derived from mixed material	Moderately low to high	Medium to rapid	High

Soil Unit	Setting Within Project Location	Approximate Thickness	Formation Setting	Permeability	Runoff	Hazard for Erosion
Scaponia-Braun Silt Loam	Active north and south convex slopes	3 to 5 feet	Colluvium derived dominantly from siltstone	Moderate	Very rapid	High
Tolke Silt Loam	Broad stable ridge tops and on gentle to moderate side slopes	5 feet	Volcanic ash and colluvium derived from siltstone and shale	Moderate	Medium to rapid	Moderate to high
Treharne Silt	Broad terraces above river	3 to 5 feet	Alluvium derived from mixed sediments	Moderate	Medium to rapid	High

4.1.7 Groundwater

Regional groundwater is located approximately 180 to 200 feet below ground surface (bgs) in the Project Area located within mountainous terrain (well pads, pipelines, NMCS, Miller Station, Miller Station Storage Area). A well log obtained from the Oregon Water Resources Department for a water well drilled at the Miller Station compressor station indicated a static groundwater level of 198 feet bgs (OWRD 2023). However, localized perched groundwater may exist in the subsurface. No springs or seeps were observed along the proposed pipeline or powerline routes, at the well pads, within the NMCS or adjacent NMCS or Miller Station and Miller Statin storage area sites during GeoEngineers' site reconnaissance.

GeoEngineers drilled several borings in the Project Area in the approximate locations shown in Figures H-2 and H-3. For the Project, four borings were drilled within the NMCS (borings NM B-1 through NM B-4; GeoEngineers 2023b), two borings at the Miller Station (MM B 1 and NM B-2; GeoEngineers 2023c). GeoEngineers previously completed eight borings for the existing NMCS (GeoEngineers 2017a), three borings for the existing Adams Well Pad (GeoEngineers 2016), which is located adjacent to and southwest of the existing NMCS, and two borings for a building addition at Miller Station (GeoEngineers 2017b). The above noted borings are the pertinent borings related to groundwater levels within the Project Area. Based on our review of these reports, static groundwater levels are greater than 100 feet below ground surface (bgs). However perched groundwater was interpreted based on observations of free water within soil samples, or in a single piezometer installed at the existing NMCS in 2017, to be located at various depths ranging between 16.5 and 52 feet bgs. It is important to note that perched groundwater measured at a depth of 16.5 feet bgs and 52 feet bgs in a piezometer at the existing NMCS site was based on measurements completed immediately after drilling and during or soon after relatively large rain events. So these groundwater levels may have been influenced by drilling fluid remaining in the piezometer or surface water infiltration into the piezometer. Borings completed within NMCS and

Miller Station between the dates of June 9 through June 15, 2023 did not encounter perched or static groundwater. Borings completed on February 10 and 11, 2016 at the Adams Well Pad site also did not encounter static or perched groundwater. GeoEngineers anticipates that groundwater levels will fluctuate with precipitation, site utilization and other factors.

The southern portion of the proposed powerline alignment is located within the Nehalem River Valley. GeoEngineers drilled four borings within the Nehalem River Valley for two proposed HDD installations. These borings were drilled using mud rotary drilling techniques, therefore groundwater could not be directly measured in the borings because of drilling fluid occupying the hole. Based on a review of two well logs for borings drilled within the Nehalem River Valley (OWRD 2023), one of which is located within about 600 feet of the powerline alignment, static groundwater levels in the Nehalem River Valley are located about 10 feet below ground surface. The borings associated with the well logs GeoEngineers reviewed were drilled in November 1994 and March 2006, and therefore likely represent relatively high groundwater levels during the rainy winter season in northwest Oregon. However, groundwater levels could be located nearer to the ground surface during heavy rain or flooding events. Copies of these two well logs are provided in Attachment H-5.

4.2 Topography

4.2.1 General

Regional topographic conditions at the Miller Station, Miller Station Storage Area, NMCS, proposed well pads, and proposed pipeline and powerline alignments are shown in Figure H-1, and in Figures H-2 and H-3. Slope gradients in the Project Area are shown in Figures H-6 and H-7. The following paragraphs describe topographic conditions at the proposed facility site and linear alignments.

4.2.2 Miller Station

Miller Station is located on gentle slopes along a broad topographic knob with elevations ranging from approximately 1,097 feet above mean sea level (MSL) in the southwest corner of the site to approximately 1,055 feet above MSL on the eastern border of the site. The proposed compressor replacement area is located on a relatively flat graveled covered surface within the southern portion of the Miller Station facility.

4.2.3 Miller Station Storage Area

The Miller Station Storage Area is located on gentle slopes along a broad topographic ridge line just north of Miller Station. Elevations within the annex area range between about 1,100 feet MSL and 1,155 feet MSL. The site is currently vegetated with mature conifer trees.

4.2.4 NMCS

In general, the NMCS is located on gentle slopes of a broad mid-slope bench with elevations ranging from approximately 1,285 feet MSL in the southwest corner of the site to approximately 1,320 feet above MSL on the eastern border of the site. The site was previously developed for the existing NMCS. Development included grading that resulted in cuts and fills that are up to 20 feet high.

More specifically, the proposed construction area of the NMCS is located adjacent to and north of the existing NMCS facility and associated compressor equipment. The proposed construction area slopes gently to the southwest except for an approximately 20-foot-high south facing cut and fill slope that separates the existing NMCS from the proposed NMCS construction area for the Project. This slope is inclined at approximately 3H:1V. Elevations within the northern gently sloping portions of the site range between about 1,317 and 1,324 feet MSL. Elevations of the south facing slope separating the existing NMCS from the proposed construction area range from approximately 1,299 feet MSL at the base of the slope to about 1,320 feet MSL at the top of the slope. A gravel road located on the west side of the NMCS provides vehicular access between the NMCS and the NMCS construction area. This gravel road is situated atop an east facing cut slope that ranges between approximately 5 and 16 feet high. This cut slope is also inclined at approximately 3H:1V. The proposed construction area of the NMCS is sparsely vegetated with short grasses. Slopes immediately surrounding the NMCS are typically inclined at gradients ranging between 10 and 40 percent, although localized steeper slopes up to about 60 percent are present, particularly on the north and northeast sides of the NMCS.

4.2.5 Well Pads

4.2.5.1 Newton Well Pad

The proposed Newton well pad site is located on gentle slopes (5 to 25 percent) of a topographic knob with elevations ranging from approximately 1,200 feet MSL in the northwest corner of the site to approximately 1,215 feet MSL in the southwest corner. The proposed Newton Well Pad and surrounding slopes had been clear cut in the latter months of 2023.

4.2.5.2 Medicine Well Pad

The proposed Medicine well pad site is located on gentle slopes of a nose of a northwest-southeast oriented ridgeline. Elevations within the site range from approximately 1,437 feet MSL on the northwestern border of the site to approximately 1,413 feet MSL on the eastern border of the site. A steep slope, with gradients typically ranging between about 50 and 80 percent is located just northeast of the site. Otherwise, the site is surrounded by gentle slopes. The site had been recently clearcut and is vegetated with shrubs, grasses, and occasional small trees.

4.2.5.3 Stegosaur Well Pad

In general, the proposed Stegosaur well pad site is located along a broad topographic knob with elevations ranging from approximately 1,555 feet above mean sea level (MSL) in the northwest corner of the site to approximately 1,565 feet above MSL on the eastern border of the site. However, the knob has been altered by development of the existing Stegosaur Well Pad, which resulted in an approximately 150-foot-wide by 150-foot-long flat gravel surfaced area. This flat area is bounded by cut slopes to the northwest and southeast that range between about 8 and 20 feet high and are inclined at gradients ranging between about 15 and 35 percent. The flat area is covered with a gravel surface while the adjacent ungraded area is located in a clearcut that is vegetated with grasses shrubs and occasional small trees.

4.2.6 Pipelines

4.2.6.1 Newton to Stegosaur Pipeline

Beginning at the Stegosaur well pad, at an elevation of roughly 1,540 feet MSL, the proposed pipeline alignment climbs gentle slopes along existing gravel logging roads to a rock pit located near Station 23+00 and at an elevation of 1,440 feet MSL. Within this segment, the pipeline alignment passes above a convergent headwall where slopes may be as steep as about 70 percent. Indications of instability of the headwall were not observed during GeoEngineers' site reconnaissance, nor were indications of global instability observed within the cut or fill slopes associated with the gravel logging roads along the pipeline alignment.

At Station 23+00 the alignment turns northwest, leaves the gravel road, and follows an old logging road down a moderately steep northeast facing slope (gradients ranging between 40 and 60 percent) to approximate Station 30+00, which is situated at an elevation of approximately 1,220 feet MSL. This portion of the alignment is either vegetated with mature conifers that are growing straight or grass, brush, and occasional small trees within a clearcut. At approximate Station 30+00 the pipeline alignment begins heading westward traversing gentle (gradients ranging between about 10 and 30 percent) north and east facing slopes to the proposed Newton Well Pad, which is situated at an elevation of approximately 1,215 feet MSL. This portion of the alignment is either vegetated with mature conifer trees that are growing straight or had been clear cut in the latter months of 2023. The alignment passes downhill of an inactive deep-seated landslide (LS-1 in Figure H-4) near Station 35+00. This landslide is discussed further in Section 9.3 of this report.

4.2.6.2 Medicine to Stegosaur Pipeline

Beginning at the Stegosaur well pad, at an elevation of roughly 1,540 feet MSL, the proposed pipeline follows existing gravel logging roads that traverses gentle to moderate slopes (10 to 40 percent) to the Medicine Well Pad which is located at an elevation of approximately 1,425 feet MSL. The alignment passes above one headwall near station 23+50 where slopes downhill of the logging road (and pipeline alignment) are as steep as approximately 80 percent. Fill slopes on the outside, downhill side of the logging road typically range between 10 and 30 percent, but locally may be as steep as approximately 70 percent. Indications of global instability were not observed within the cut or fill slopes associated with the gravel logging roads along the pipeline alignment during GeoEngineers' site renaissance.

4.2.6.3 Stegosaur to NMCS Pipeline

The Stegosaur to NMCS pipeline begins at the Stegosaur well pad at approximately 1,540 feet MSL then heads southwest traversing a gentle to moderate (10 to 50 percent) southwest facing slope to Station 2+62 at an elevation of approximately 1,490 feet MSL. The proposed pipeline then turns southeast and follows an existing gravel logging road to Station 7+00 where it continues southwest traversing a moderate (30 to 50 percent) south to southeast facing slope to approximate Station 11+00. At this point, the pipeline alignment turns southwest and traverses a moderately sloping southwest facing slope along an existing pipeline ROW to Station 15+82 where the pipeline continues following the pipeline ROW within a gently sloping saddle between two knobs. At approximate Station 19+25, the pipeline alignment turns south and continues following the existing pipeline ROW southward across a gentle to moderately sloping northwest facing slope to the eastern boundary of the NMCS located at an elevation of approximately 1,320 feet MSL. The pipeline alignment then follows the eastern and southern fence lines of the NMCS traversing gently sloping northeast and south facing slopes to its terminus in the southwest corner of the NMCS at an elevation of approximately 1,300 feet MSL. All slopes traversed by the pipeline alignment had recently been clearcut and are vegetated with grass, brush, and occasional small trees. Indications of slope instability were not observed along the pipeline during our site reconnaissance.

4.2.7 Powerline Alignment

Beginning at Miller Station, which is situated at an elevation of approximately 1,050 feet MSL, the powerline alignment traverses a gently sloping broad ridge top following a gravel road (called Mainline Road) until reaching Station 9+25 where it leaves the Mainline Road and descends a gently to steeply sloping southeast facing slope following an existing powerline and pipeline ROW to approximate Station 53+00 which is situated at an elevation of about 540 feet MSL. Slope gradients along this segment of the alignment typically range between about 5 and 40 percent, but locally may be as steep as approximately 70 percent. At approximate Station 53+00, the pipeline rejoins Mainline Road and traverses a relatively flat to very gently southward sloping alluvial valley of the Nehalem River to the southern end of the powerline alignment which is situated at an elevation of approximately 535 feet MSL near the intersection of Mainline Road and Highway 202. Between Station 53+00 and the southern end of the powerline alignment, Mainline Road is mostly situated on a fill prism that is between approximately 5 and 8 feet higher than the adjacent ground surface and with side slope gradients ranging between about 20 and 50 percent. GeoEngineers did not observe instability of the fill prism slopes during our site reconnaissance. The alignment crosses two streams (Lindgren Creek and Lyons Creek) along this section of the alignment. The streams pass beneath Mainline Road through existing culverts.

Between approximate Stations 10+00 and 50+00 the powerline alignment traverses a large, ancient deep-seated landslide. This landslide is shown as LS-4 in Figure H-5. Most of the landslide scarp and body had been recently clearcut. Mature conifer trees that were growing straight were observed where the landslide had not been clear cut. The alignment also passes downhill of a landslide near Station 65+00. These landslides are discussed further in Section 9.3 of this report.

5.0 Evidence of Consultation – OAR 345-021-0010(1)(h)(B)

> *(B) A summary of consultation with the Oregon Department of Geology and Mineral Industries regarding the appropriate methodology and scope of the seismic hazards and geology and soil-related hazards assessments, and the appropriate site-specific geotechnical work that must be performed before submitting the application for the Department to determine that the application is complete;*

In preparing this exhibit, GeoEngineers consulted with DOGAMI and reviewed DOGAMI publications and guidance documents as listed in the references section of this exhibit. Jason McClaughry, RG of DOGAMI was contacted regarding this geologic hazard study, as required by OAR 345-021-0010 (1)(h)(b). GeoEngineers and NW Natural met with Jason McClaughry and Sarah Esterson of the Oregon Department of Energy on September 21, 2023 to discuss GeoEngineers' approach to evaluating geologic hazards. Based on our discussion on September 21, 2023 and subsequent email confirmation of our discussion, DOGAMI agreed with GeoEngineers' geologic hazard evaluation approach. A copy of an email confirming the discussions with Jason McClaughry is provided in Attachment H-1 of this exhibit.

6.0 Description of Site-Specific Geotechnical Work – OAR 345-021-0010(1)(h)(C)

> *(C) A description and schedule of site-specific geotechnical work that will be performed before construction for inclusion in the site certificate as conditions;*

GeoEngineers completed site-specific geotechnical work for the Project, including:

- Geotechnical investigation and report for the proposed construction area of the NMCS (GeoEngineers 2023b). This work included drilling four borings to depths ranging between 60 and 100 feet bgs within the NMCS, downhole seismic testing within two of the borings, electrical resistivity field testing at the site, a laboratory testing program to characterize engineering and corrosion properties of soils sampled from the borings and developing foundation and earthwork recommendations for grading and placement of structures at the site. Borings drilled within the NMCS are shown in Figure H-2. The geotechnical report is included in Attachment H-3.
- Geotechnical investigation and report for replacement of the compressors at Miller Station (GeoEngineers 2023c). This work included drilling two borings to depths ranging between 40 and 80 feet bgs near the proposed compressor replacement area, downhole seismic testing within one of the borings, a laboratory testing program to characterize engineering and corrosion properties of soils sampled from the borings and developing foundation and earthwork recommendations for grading and placement of structures at the site. Borings

drilled within Miller Station are shown in Figure H-3. The geotechnical report is included in Attachment H-4.

- Geologic site reconnaissance and preliminary routing assessments for the proposed Newton to Stegosaur, Stegosaur to Medicine and Stegosaur to NMCS pipelines. This work included walking or driving potential pipeline routes, observing the ground surface and vegetation characteristics for indications of slope instability, and observing landslides identified along or near the routes from a desktop review of a LiDAR hillshade model, published geologic mapping and a review of DOGAMI's web-based Statewide Landslide Information Database for Oregon (SLIDO). Based on these evaluations, GeoEngineers assisted NW Natural in choosing pipeline routes that avoided identified landslides or unstable slopes. In addition, GeoEngineers observed readily available exposures of geologic materials along the proposed pipeline alignments.
- Preliminary HDD feasibility studies for potential HDD installations of pipelines between the Newton and Stegosaur well pads and between the Medicine and Stegosaur well pads (GeoEngineers 2023d, GeoEngineers 2023e). This work included evaluating geometric and likely subsurface conditions and an evaluation of the feasibility of potential HDD installations based on publicly available DEM topographic data, geologic mapping and borings completed in the area. Because these alignments were determined to be practically infeasible due to the extreme topography along the potential HDD alignments, they are not presented or discussed in this report.
- Preliminary HDD feasibility study for an HDD installation of the powerline across the alluvial Nehalem River Valley between Highway 202 and just north of Lindgren Creek (GeoEngineers 2023f). This work included evaluating geometric and likely subsurface conditions and an evaluation of the feasibility of the potential HDD installation based on publicly available DEM topographic data, geologic mapping and borings completed in the area. This study revealed that an approximately 2,900-foot-long HDD with relatively complicated geometry would be required to install the HDD, and that pull forces during installation would likely exceed the proposed 2 to 4-inch diameter high density polyethylene (HDPE) conduits that would house powerlines. Therefore, the Project team is proposing to install the powerline conduits using conventional open trench methods along most of the route, and use the HDD trenchless method to cross Lindgren Creek and Lyons Creek only.
- Draft HDD design for HDD installations to install the proposed powerline conduit located in the Nehalem River Valley beneath Lyons Creek and Lindgren Creek. This work included evaluating surface conditions relative to HDD workspaces, characterizing subsurface conditions along the two proposed HDD alignments by drilling a total of four boring to depths ranging between 60.5 and 65.5 feet bgs (See Figure 3; borings LYCB-1, LYCB-2, LCB-1 and LCB-2), evaluating collapse potential of the proposed HDPE conduit that will house the powerline, evaluating installation forces for installation of the proposed HDPE

conduit, evaluating the risk of hydraulic fracture along the HDD profiles and preparing draft HDD designs for installation of the proposed powerline conduit.

- A site specific liquefaction analysis of soils within the Nehalem River Valley using information obtained from four borings drilled for the proposed powerline HDD installations. The liquefaction analysis was completed to evaluate estimated ground settlement from liquefaction along the proposed powerline route within the Nehalem River Valley during a design level earthquake.

7.0 Proposed Site-Specific Geotechnical Work for Transmission Lines and Pipelines – OAR 345-021-0010(1)(h)(D)

(D) For all transmission lines, and for all pipelines that would carry explosive, flammable or hazardous materials, a description of locations along the proposed route where the applicant proposes to perform site specific geotechnical work, including but not limited to railroad crossings, major road crossings, river crossings, dead ends (for transmission lines), corners (for transmission lines), and portions of the proposed route where geologic reconnaissance and other site specific studies provide evidence of existing landslides, marginally stable slopes or potentially liquefiable soils that could be made unstable by the planned construction or experience impacts during the facility's operation;

As summarized in this Exhibit, geologic hazards were not identified that would require further geotechnical study. However, as summarized in the potential adverse impacts to Slope Stability Section 9.3.3 in this exhibit, there may be areas where cutting into hillslopes would be required to construct well pads and Miller Station Storage Area, install the pipeline adjacent to existing roads or along cross country segments of the pipeline. GeoEngineers recommends that site-specific geotechnical studies be conducted in these areas once they have been delineated, to provide recommendations to mitigate potential adverse impacts to slope stability that may result from cutting into hillsides adjacent to the existing roadways or on cross country segments of the pipeline routes. Such evaluations should include recommendations for cut and fill slopes, and restoring site grades to pre-construction conditions, which may require placement of fill slopes in excess of 50 percent gradient. Recommendations for engineered fill slopes should include specifications for materials to be used, adequacy of native soils to be used as fill, lift thickness, and compaction criteria for wet and dry weather conditions.

8.0 Assessment of Site-Specific Seismic Hazards – OAR 345-021-0010 (h)(E)

(E) An assessment of seismic hazards, in accordance with standard-of-practice methods and best practices, that addresses all issues relating to the consultation with the Oregon Department of Geology and Mineral Industries described in paragraph (B) of this subsection, and an explanation of how the applicant will design, engineer, construct, and operate the facility to avoid dangers to human safety and the environment from these seismic hazards. Furthermore, an explanation of how the applicant will design, engineer, construct and operate the facility to integrate disaster resilience design to ensure recovery of operations after major disasters. The applicant must include proposed design and engineering features, applicable construction codes, and any monitoring and emergency measures for seismic hazards, including tsunami safety measures if the site is located in the DOGAMI-defined tsunami evacuation zone;

8.1 Historical Seismicity

Attachment H-2 provides a list of recorded earthquakes that have epicenters within approximately 50 miles of the proposed pipeline route (USGS 2023a). The first list (Table H-2.1) provides a list of recorded earthquakes over magnitude 2.5 that occurred within 50 miles of the site between November 1961 and August 2023. Reported magnitudes use duration magnitude, short period body wave magnitude, moment magnitude, or local Richter magnitude scales. The location of earthquakes with reported magnitude values listed in Table H-A.1 are shown with respect to the site in Figure H-8.

The second list (Table H-2.2) presents observed earthquakes that occurred between 1841 and 1964 that caused ground shaking near their respective epicenter that exceeded Modified Mercalli Intensity (MMI) IV (NOAA 2015). The MMI scale relies on observations of a general population during a seismic event, and event records included observations from numerous cities near the earthquake epicenters. The data presented in Table H-2.2 has been edited to only include the MMI observed in the city nearest the earthquake epicenter. The location of earthquakes with observed MMI values are listed in Table H-A.2, and the estimated locations of the earthquake epicenters provided in the data, are shown with respect to the site in Figure H-8. A description of the levels of shaking included in the MMI scale (Wood and Neumann 1931) is presented in Table H-4 below.

Table H-4. Description of the Levels of Modified Mercalli Intensity

Intensity	Shaking	Description/Damage
I	Not felt	Not felt except by a very few under especially favorable conditions.
II	Weak	Felt only by a few persons at rest, especially on upper floors of buildings.

Intensity	Shaking	Description/Damage
III	Weak	Felt quite noticeably by persons indoors, especially on upper floors of buildings. Many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibrations similar to the passing of a truck. Duration estimated.
IV	Light	Felt indoors by many, outdoors by few during the day. At night, some awakened. Dishes, windows, doors disturbed; walls make cracking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably.
V	Moderate	Felt by nearly everyone; many awakened. Some dishes, windows broken. Unstable objects overturned. Pendulum clocks may stop.
VI	Strong	Felt by all, many frightened. Some heavy furniture moved; a few instances of fallen plaster. Damage slight.
VII	Very Strong	Damage negligible in buildings of good design and construction; slight to moderate in well-built ordinary structures; considerable damage in poorly built or badly designed structures; some chimneys broken.
VIII	Severe	Damage slight in specially designed structures; considerable damage in ordinary substantial buildings with partial collapse. Damage great in poorly built structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned.
IX	Violent	Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb. Damage great in substantial buildings, with partial collapse. Buildings shifted off foundations.
X	Extreme	Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations. Rails bent.

During site reconnaissance, general observations were conducted by GeoEngineers to evaluate the presence of structural features such as faulting and other discontinuities that may be indicative of historical seismicity along the proposed pipeline alignments, powerline alignment, NMCS, Miller Station Storage Area and Miller Station. Faults in outcrops, distinct topographic lineations, vegetation patterns or surface water patterns that would indicate historical seismicity along the proposed pipeline alignment were not identified.

8.2 Contributing Earthquake Sources

A site-specific Probabilistic Seismic Hazard Analysis (PSHA) was completed using the most current edition of the USGS Unified Hazard Tool; Dynamic conterminous U.S. 2014 (update)(4.2.0). Seismic hazard deaggregation was performed for the 4,975-year, 2,475 year and 475-year hazard levels for rock outcrop condition (i.e. Vs30 = 760 m/s). The 475-year motion corresponds to a 10 percent probability of exceedance (PE) in 50 years. The 2,475-year motion has a 2 percent Probability of Exceedance in 50 years, and the 4,975-year motion has a 1 percent Probability of Exceedance in 50 years.

The seismic deaggregation results (USGS 2023b) show that the dominant seismic hazard source for the 475-year, 2,475-year and 4,975-year earthquake levels is the magnitude (M)8.8 to M9.1 Cascadia Subduction Zone (CSZ) interface event. The second strongest hazard is the M6.9 to

M7.0 deep (35 to 70 kilometers in depth) intraslab earthquake similar to the M6.8 Nisqually earthquake that occurred in February 2001 near Olympia, Washington. Crustal earthquakes of M6.2 from background seismicity that are associated with gridded crustal fault sources of non-discrete origin are included in the 475-year earthquake level. However, the background seismicity is effectively muted by the hazard presented by the Cascadia Subduction Zone earthquake when longer return periods are considered.

The background seismicity model places the fault distance somewhat subjectively, based on the relative probability that an undiscovered fault could become active near a site. However, one of these crustal sources could be the Gales Creek Fault which may have been active as recently as Holocene time (Wells 2018; Wells et al 2020). In the case of the CSZ and deep intraslab events, calculated distances of the fault sources are placed closer to the site as the hazard level increases to account for the uncertainty surrounding the potential epicenter of each anticipated event. Conversely, the relative probability of the background seismicity model considers historic earthquake activity within a given study area.

An overview of the range of distance-magnitude pairs and percent contribution to the seismic hazard described within the seismic deaggregation at the extremities of the Project site for all principal sources of seismicity is presented in Tables H-5 through H-7.

Table H-5. Summary of USGS Seismic Hazard Deaggregation for 475-year Hazard Level

Fault Source	Distance Range from Site (miles)	Magnitude	Percent Contribution to Hazard[1]
Cascadia Subduction Zone (Interface)	23.4 - 58.0	8.8 – 9.1	55.1
Deep Intraslab (35 to 70 kilometers)	38.6 – 61.7	6.9	16.5
Background Seismicity (Gridded Crustal Fault Source)	11.5 – 17.3	6.2	6.5
1. The percent contribution to hazard describes the relative contribution of the predicted ground motion from an individual fault source to the total seismic hazard for a given return period.			

Table H-6. Summary of USGS Seismic Hazard Deaggregation for 2,475-year Hazard Level

Fault Source	Distance Range from Site (miles)	Magnitude	Percent Contribution to Hazard[1]
Cascadia Subduction Zone (Interface)	23.4 – 58.0	8.8 – 9.1	75.2
Deep Intraslab (35 to 70 kilometers)	32.4 – 49.5	7.0	6.2
Background Seismicity (Gridded Crustal Fault Source)	N/A	N/A	N/A
1. The percent contribution to hazard describes the relative contribution of the predicted ground motion from an individual fault source to the total seismic hazard for a given return period.			

Table H-7. Summary of USGS Seismic Hazard Deaggregation for 4,975-year Hazard Level

Fault Source	Distance Range from Site (miles)	Magnitude	Percent Contribution to Hazard[1]
Cascadia Subduction Zone (Interface)	23.4 – 58.0	8.8 – 9.1	79.1
Deep Intraslab (35 to 70 kilometers)	31.4 – 36.9	7.0	3.40
Background Seismicity (Gridded Crustal Fault Source)	N/A	N/A	N/A
1. The percent contribution to hazard describes the relative contribution of the predicted ground motion from an individual fault source to the total seismic hazard for a given return period.			

In addition to fault hazards returned by the seismic deaggregation results, ten crustal faults capable of generating strong ground motion were identified by the USGS Earthquake Hazards Program's Quaternary Fault and Fold Database within 50 miles of the Project. A summary of Quaternary faults within 50 miles of the Project is provided in Table H-8, and approximate fault locations are shown with respect to the site in Figure H-8. Fault source parameters were obtained online from the USGS Quaternary Fault and Fold Database of the United States (USGS 2023c) or are otherwise referenced below Table H-8.

Table H-8. Quaternary Faults within 50 Miles of the Project

Fault Source	Nearest Distance to Site (miles)	Fault Length (kilometers)	Maximum Magnitude	Vertical or Horizontal Slip Rate (mm/year)
Gales Creek Fault Zone	17.6	73	6.75	0.016
Portland Hills Fault	33.7	49	7.05	0.115
Helvetia Fault	40.9	7	6.4	0.016
East Bank Fault	34.4	29	N/A	< 0.2
Oatfield Fault	33.5	29	N/A	< 0.2
Willapa Bay Fault Zone	38.4	37	N/A	0.2 – 1.0
Beaverton Fault Zone	39.1	15	N/A	< 0.2
Lacamas Lake Fault	42.3	24	6.67	0.026
Tillamook Bay Fault Zone	43.5	32	N/A	< 0.2
Nehalem Bank Fault	47.8	101	N/A	1.0 – 5.0
Cascadia Fold and Fault Belt	36.3	484	N/A	1.0 – 5.0
Fault H (no 790)	43.0	49	N/A	>5.0
Unnamed offshore faults	34.5	280	N/A	1.0 – 5.0
Note: N/A indicates that the maximum magnitude was not provided in USGS Earthquake Hazards Program's National Seismic Hazard Maps – Source Parameters reviewed.				

8.2.1 Crustal Seismicity

Comparison of the distance of Quaternary faults and gridded crustal faults provided by the PSHA to the proposed Project site suggests that fault sources provided by the PSHA are generally closer and yields more conservative estimate of the crustal seismic hazard for the 475-year earthquake levels. However, current deaggregation results suggest crustal seismicity is a far less likely hazard when compared with the CSZ and deep intraslab events for the 2,475-year and 4,975-year return intervals.

8.2.2 Cascadia Subduction Zone (CSZ) and Intraplate Seismicity

Most of the seismic hazard is from potential CSZ interface events and deep intraslab earthquakes. At the 475-year hazard level, deep intraplate events pose up to approximately 17 percent of the hazard described by the PSHA, whereas combined CSZ interface events present approximately 72 percent of the described hazard. The influence of the CSZ becomes even more dominant at the 2,475-year and 4,975-year hazard levels. As the USGS Probabilistic Seismic Hazard Model provides spectral acceleration parameters based on the most severe ground motion, the PSHA provides a reasonable and conservative description of contributing earthquake sources and ground motion parameters. Past comparisons of crustal, intraplate and CSZ interface estimated Peak Ground Accelerations (PGAs) at Astoria, Oregon and Portland, Oregon agree well with the conclusion that CSZ interface events will likely dominate deep intraplate seismicity (Geomatrix 1995).

8.3 Median Ground Response Spectrum

Subsurface pipelines and powerline design only utilize the PGA of the acceleration response spectra. For this reason, analysis of longer period spectral acceleration parameters was not performed for pipeline and powerline segments of the Project. However, site class adjusted PGA along the pipeline alignments were considered as described in Section 8.4.1.2.

Site response parameters for the design of structures at the NMCS and Miller Station are included with our geotechnical engineering reports provided as Attachment H-3 and Attachment H-4.

8.4 Site Seismic Hazards

8.4.1 Ground Shaking

Ground shaking for the 475-year, 2,475-year, and 4,975-year hazard level was assessed using the PSHA for rock outcrop conditions as described in Section 8.2. To characterize ground motion amplification effects along proposed pipeline route, a site class was assigned in accordance with methods outlined in Chapter 20 of American Society of Civil Engineers (ASCE) 7-16 and based on our geologic interpretation. The Applied Technology Council (ATC) hazards by location tool (ATC 2023) was then used to collect mapped acceleration parameters at each well pad, approximate midpoint of each pipeline between well pads, the NMCS, Miller Station, the south end of the powerline alignment, and the approximate midpoint of the powerline alignment.

8.4.1.1 Rock Outcrop PGA

A summary of PGAs for rock outcrop conditions as determined by the Dynamic conterminous U.S. 2014 (update) (4.2.0) version of the USGS Unified Hazard Tool at ten locations of the Project for the 475-year, 2,475-year and 4,975-year hazard levels are presented in Table H-9.

Table H-9. USGS Rock Outcrop Peak Ground Acceleration (PGA)

Location	475-year PGA	2,475-year PGA	4,975-year PGA
Miller Station/Miller Station Storage Area/ North Powerline Alignment	0.1918	0.5258	0.7179
NMCS	0.1938	0.5353	0.7305
Newton Well Pad	0.1946	0.5393	0.7359
Stegosaur Well Pad	0.1941	0.5356	0.7309
Medicine Well Pad	0.1931	0.5299	0.7233
Newton to Stegosaur I/W Pipeline	0.1942	0.5371	0.7329
Medicine to Stegosaur I/W Pipeline	0.1936	0.5329	0.7272
Stegosaur to NMCS I/W Pipeline	0.1939	0.5352	0.7303
Mid Powerline Alignment	0.1912	0.5249	0.7167
South Powerline Alignment	0.1907	0.5229	0.7140

8.4.1.2 Site Class Adjusted PGA_M

Site class was determined using geologic interpretation of conditions at the proposed infrastructure locations. Table H-10 summarizes the interpreted site class at the proposed infrastructure locations, and mapped maximum considered geometric mean (MCEG) PGA, site coefficient F_{PGA} and the site class adjusted mapped MCEG PGA (PGA_M). Mapped acceleration parameters were determined at the locations listed below from the ATC hazards by location tool (ATC 2023) for ASCE 7-16.

Table H-10. ASCE 7-16 Site Class, PGA and PGA_M

Location	Site Class	PGA	F_{PGA}	PGA_M
Miller Station/Miller Station Storage Area/North Powerline Alignment	D	0.458	1.142	0.523
NMCS	C	0.465	1.200	0.558
Newton Well Pad	D	0.465	1.135	0.528
Stegosaur Well Pad	D	0.465	1.135	0.528

Location	Site Class	PGA	F_{PGA}	PGA_M
Medicine Well Pad	D	0.461	1.139	0.525
Newton to Stegosaur I/W Pipeline	D	0.466	1.134	0.529
Medicine to Stegosaur I/W Pipeline	D	0.464	1.137	0.527
Stegosaur to NMCS I/W Pipeline	D	0.465	1.135	0.528
Mid Powerline Alignment	D	0.457	1.143	0.522
South Powerline Alignment	D	0.455	1.145	0.521

Buried pipelines are considered to have a low seismic vulnerability (Ballantyne 2008). Ballantyne also states that historically arc welded steel pipe has a low vulnerability under seismic loading when compared to other pipe materials (such as ductile iron, bell, and spigot joint steel) because it can accommodate both wave propagation and moderate levels of permanent ground deformation (Ballantyne 2008). Based on this information, it is our opinion that there is a low risk of ground shaking in the absence of other deformation adversely affecting the proposed pipeline or the proposed powerline.

8.4.2 Fault Rupture

No lineaments were identified in the LiDAR hillshade model, vegetation patterns or soil contrasts in aerial photographs that may indicate previously unidentified faults crossing the proposed infrastructure. The Gales Creek Fault Zone is the closest fault structure to the site that has been mapped by the USGS Quaternary Fault and Fold Database (USGS 2023c). Recent studies on the fault have verified Quaternary movement along the fault zone and possibly identified movement of the fault in the Holocene based on offset of bedrock, loess and flood plain deposits in a trench excavated across the fault (Wells 2018, Wells et al. 2020). However, the closest expression of the Gales Creek Fault Zone is located approximately 18 miles south of the site.

Two faults are mapped by DOGAMI (2020) crossing the powerline alignment within the Nehalem River Valley. These faults are not mapped by the USGS quaternary fault and fold database and therefore are not considered active. As such, there is a low probability of fault rupture adversely affecting the proposed subsurface powerline.

8.4.3 Seismically Induced Landslides

Earthquake forces can cause slope failures and movement of sloping ground. Existing landslides are most susceptible to seismic slope failure, but very steep slopes and jointed rock outcrops are also vulnerable. The proposed pipelines, NMCS and Stegosaur and Medicine well pads are not located in close proximity to existing landslides that could be re-activated during a seismic event and avoid very steep slopes. Therefore, there is a relatively low risk of seismically induced landsliding affecting the proposed pipelines, NMCS, and Stegosaur and Medicine well pads.

Miller Station is located near LS-4 (the Lindgren Creek Landslide) and LS-5 (the Miller Station Landslide). The powerline alignment also crosses LS-4. There is a risk that the LS-4 could be reactivated during a seismic event. LS-5 has been regraded and mitigated as discussed in Section 9.3.2.4 therefore, it is unlikely that this landslide would be reactivated by earthquake shaking.

Seismically reactivated landslides present a low to moderate risk to Miller Station, depending on the location of the earthquake and the magnitude of landslide movement. Based on ground deformation due to landsliding mapping for a simulated M9 earthquake (Madin and Burns 2013), ground deformation due to landsliding in the vicinity of LS-4 is expected to be approximately 4 to 12 inches. If LS-4 is reactivated, there is a low to moderate risk that the scarp would retrogress through the in-tact weathered Pittsburg Bluff Formation underlying the compressor station replacement area (as documented by borings completed at the site for the Project) and adversely affect the proposed Miller Station.

The powerline alignment passes through LS-4 and therefore could potentially be affected by reactivation of LS-4 during a seismic event. Depending on the magnitude of movement, HDPE conduits are flexible and may accommodate some movement. However, if this landslide is reactivated during a seismic event and the landslide damages the powerline and adjacent pipelines within the ROW following the powerline, there is a low risk to public safety because the known landslide is located in an unpopulated area.

The Miller Station Storage Area is located in close proximity to LS-7, which is a road fill related failure. The scarp of this landslide was stabilized in October through December 2023 by constructing a soldier pile wall with tiebacks, and the road surface was restored. Although the risk of the scarp of the landslide retrogressing up slope to involved Mainline Road has been mitigated, there is a risk that additional failure of LS-7 could occur during a seismic event. However, it is unlikely that additional failure of LS-7 would retrogress upslope to involve the Miller Station Storage Area.

8.4.4 Liquefaction and Liquefaction-Induced Hazards

Liquefaction is not typically associated with mountainous terrain where static groundwater is located over 100 feet bgs; rather it is associated with thick deposits of saturated, loose to medium dense granular alluvium, typically in low-lying alluvial plains with high groundwater conditions, such as the Nehalem River Valley. Based on our interpretation of the geological conditions along the proposed pipeline alignments and at the NMCS, Miller Station, Miller Station Storage Area and well pads, liquefaction is not considered to be a credible hazard to these proposed Project elements.

The Nehalem River Valley contains alluvial materials (sand, silt, clay, gravel) and relatively high groundwater levels and therefore may be susceptible to liquefaction during earthquake shaking. Based on four borings conducted by GeoEngineers within the Nehalem River Valley (See Figure 3; borings LYCB-1, LYCB-2, LCB-1 and LCB-2) and our review of well logs, we anticipate that soils susceptible to liquefaction will be between 20 and 30 feet in thickness overlying bedrock. No structures are proposed for construction within the Nehalem River Valley, but the proposed powerline alignment is partially located within the valley.

GeoEngineers conducted a liquefaction triggering and settlement analysis for each of the four logged soil borings that represent subsurface conditions along the proposed powerline alignment using three separate methods[1] based on the PGA_M (ATC 2023), mean earthquake magnitude (USGS 2023b), and assuming groundwater is within 5 feet of the ground surface along the portion of the powerline alignment in the Nehalem River Valley. The analyses was conducted using methods developed by Boulanger and Idriss (2014), Youd et al (2001), and Seed et al (2003). Based on this analysis, liquefaction induced settlement is estimated to result in approximately 1½ inches to 7 inches of surface settlement along the powerline alignment located within the Nehalem River Valley after a design level earthquake. The liquefaction settlement primarily occurs from 5- to 10-foot thick loose sand layers observed in the borings.

HDPE powerline conduits are relatively flexible and unlikely to be adversely affected by earthquake shaking or liquefaction. These conduits are relatively more susceptible to damage from differential settlement occurring during and after an earthquake. Based on soil conditions documented by our borings and our well log research, we expect liquefaction settlement to occur as described above. However, we anticipate that the conduit alignment will be underlain by locally similar subsurface conditions within the Nehalem River Valley, which should reduce the potential for differential settlement that could impart large shear and bending stresses on the conduit. Therefore, it is our opinion that there is a relatively low risk of liquefaction or liquefaction induced settlement damaging the HDPE powerline conduit. Furthermore, failure of the underground powerline conduit and housed powerline in the event that liquefaction does occur during a design level earthquake does not represent a life safety issue.

8.4.5 Coseismic Subsidence

Discussions of subsidence associated with M9.0 CSZ events are typically limited to areas in close proximity to the coastline in the Northwest. Coseismic Subsidence Map for Simulated Magnitude 9 Cascadia Earthquake: Clatsop County, Oregon (Madin and Burns 2013) and associated geographic information system (GIS) data depicting modeled coseismic subsidence developed as part the 2012 Oregon Resilience Plan for Cascadia Subduction Zone Earthquakes were reviewed to provide a qualitative assessment of the potential for subsidence at the site.

Mapping of subsidence presented by Madin and Burns (2013) does not extend to the Project Area. However, GIS data included with the report reaching approximately 15 miles east beyond the Clatsop County map's published boundary, suggests that maximum subsidence in the Project Area may range between zero and 1 foot. While the GIS data reviewed falls outside of Madin and Burns' (2013) published coseismic subsidence mapping, it could be inferred that a relatively small amount of subsidence associated with a M9.0 CSZ event may impact surface elevations as far east as the Project.

8.5 Seismic Hazard Mitigation

The Oregon Structural Specialty Code incorporates the 2021 International Building Code (IBC), with current amendments by the state of Oregon and local agencies. Pertinent design codes as they relate to geology, seismicity, and near-surface soil, are contained in IBC Section 1613, with slight modifications by the current amendments of the state of Oregon and local agencies. The Project will be designed to meet these minimum standards.

Subsurface conditions within the mountainous area north of the Nehalem River valley are not susceptible to liquefaction. Therefore, no specific seismic hazard mitigation is recommended other than typical seismic structural design of structures within the NMCS and Miller Station. Alluvial materials within the Nehalem River Valley are about 25 feet in thickness and will be susceptible to liquefaction during a design level earthquake. Our site-specific liquefaction analysis indicates that post-liquefaction surface settlement would be between 1½ inches and 7 inches along the powerline conduit alignment within the Nehalem River Valley during a design level earthquake. We expect differential settlement will be far enough dispersed that the risk of damage to the HDPE powerline conduit will be relatively low. Because no structures that are sensitive to damage from liquefaction or settlement will not be constructed within the Nehalem River Valley, no other site-specific liquefaction mitigation should be required.

No quaternary faults (active faults) are mapped crossing the pipeline alignments, powerline alignment, well pads, NMCS, Miller Station Storage Area or Miller Station. Therefore, no fault specific mitigation is necessary.

9.0 Assessment of Geology and Soil Related Hazards – OAR 345-021-0010(1)(h)(F)

> *(F) An assessment of geology and soil-related hazards which could, in the absence of a seismic event, adversely affect or be aggravated by the construction or operation of the facility, in accordance with standard-of-practice methods and best practices, that address all issues relating to the consultation with the Oregon Department of Geology and Mineral Industries described in paragraph (B) of this subsection. An explanation of how the applicant will design, engineer, construct and operate the facility to adequately avoid dangers to human safety and the environment presented by these hazards:*

An assessment of soil related hazards, including landsliding, erosion, flooding and groundwater was completed for this study. The sections below provide an assessment of each of these hazards as required by the OARs.

9.1 Erosion

Erosion can be caused by air or water. Wind erosion is not a significant concern because of the fine-grained surface soils, tree cover or gravel surfacing along the pipeline/powerline alignments, planned post-construction revegetation of the pipeline/powerline corridors that do not follow gravel roads, and the subgrade protection measures that will be implemented to provide equipment access. In addition, the Miller Station, Miller Station Storage Area, NMCS and well pads will be surfaced with gravel, which will reduce erosion potential.

The soils at the Project Area are susceptible to water erosion as indicated in Section 4.1.6. However, where the pipeline and powerline alignments follow the existing roadways, water erosion will be minimal because of existing surface water drainage systems and crushed rock road surfacing.

There is a relatively high risk of water erosion where the Newton to Stegosaur pipeline alignment and the powerline alignment traverse slopes cross country. Mitigation for this risk is discussed in Section 9.4.

9.2 Flooding and Groundwater

The proposed pipelines, well pads, NMCS, Miller Station and Miller Station Storage Area are located in mountainous terrain north of Highway 202 and more than 500 feet higher in elevation than the Nehalem River, which is the nearest river with flood potential near the Project site. In addition, static groundwater is located more than 150 feet below the ground surface within the mountainous terrain. Therefore, there is a low risk of groundwater or flooding affecting the proposed pipelines, well pads, NMCS, Miller Station Storage Area and Miller station site.

Federal Emergency Management Agency flood mapping maps a flood zone denoted as Flood Zone A along Lindgren Creek from just north of Mainline Road to the Nehalem River, and on the south side of Highway 202 near the proposed powerline alignment (FEMA 2023). Zone A is noted as having a 1 percent annual chance of flood hazard. The proposed powerline will be installed in conduits beneath the mapped flood hazard zone using HDD installation, which mitigates potential buoyancy associated with flooding. Based on the powerline installation method and the low chance of annual exceedance mapped by Federal Emergency Management Agency, there is a low risk of flooding adversely affecting the powerline.

The southern region of the powerline alignment is within the Nehalem River valley, where groundwater levels could be located near the surface during heavy rain events. The powerline will be installed within HDPE conduits placed in approximately 3- to 4-foot-deep trenches within the fill prism of Mainline Road (which is situated between approximately 5 and 8 feet above the adjacent ground surface) using conventional open trench methods, or beneath Lyons and Lindgren Creeks using HDD installation methods. Because the conduit will be located above the regional ground surface, or confined by the drilled hole of an HDD, there is a low risk of high groundwater levels adversely affecting the powerline installations.

9.3 Landslide and Slope Stability

9.3.1 General

GeoEngineers completed a desktop study to identify landslide hazards at the Project site by reviewing SLIDO (v4.4; DOGAMI 2021) and by interpreting a LiDAR generated hillshade model. A site reconnaissance of the Project Area was then completed focusing on landslides identified in the desktop study and observing conditions along the proposed pipeline and powerline routes, well pads, NMCS, Miller Station and Miller Station Storage Area . The proposed well pads were sited and the proposed pipelines and powerline alignments were routed to avoid existing landslides and/or unstable slopes to the extent practical.

The compilation landslide mapping by the DOGAMI (DOGAMI 2021) shows numerous landslides within the Project Area as shown in Figures H-4 and H-5. Most of these landslides are not in close proximity to the proposed pipeline/powerline alignment, well pads or compressor station sites such that they are unlikely to affect the Project. However, there are a total of 7 landslides that were identified to be in close proximity to various Project facilities or crossed by the powerline alignment that were evaluated. The following sections describe these landslides relative to the proposed Project infrastructure.

9.3.2 Site Specific Mapping and Evaluation

9.3.2.1 Pipeline Alignments

Two landslides were identified, LS-1 and LS-3, during desktop review and site reconnaissance near the proposed Newton to Stegosaur and Medicine to Stegosaur pipeline routes, respectively. The toe of LS-1 is located approximately 180 feet south of the proposed Newton to Stegosaur pipeline route near Station 35+00, as shown in Figure H-4. LS-1 was previously mapped by SLIDO (DOGAMI 2021) and confirmed in the field by GeoEngineers during a routing site reconnaissance for the Newton to Stegosaur pipeline route. The landside is a deep-seated rotational feature that is approximately 200 feet wide and 550 feet long. It is characterized by a weathered arcuate-shaped scarp that is inclined at approximately 55 percent, relatively flat mid-slope bench, slightly hummocky slide body, very vague lateral margins, and weathered landslide toe. The vague nature of the lateral margins made the lateral slide boundaries difficult to map in the field. However, the toe bulge was observed, and defined the downhill extent of the slide. The slide initiated from a moderate to steep (approximately 50 to 70 percent) northeast facing slope and came to rest on a gentle (approximately 20 percent) portion of the slope. The slide scarp, body and toe are vegetated with mature conifer trees that are growing straight. In-tact old growth stumps are present within the limits of the slide. Observations of the slide indicate that LS-1 is a dormant-young (Keaton and DeGraff 1996) landslide that is inactive. The proposed Newton to Stegosaur pipeline was routed on gentle (10 to 20 percent) slopes downhill of LS-1 to avoid the landslide. Based on the dormant nature of the landslide and its location relative to the proposed Newton to Stegosaur pipeline, LS-1 presents a low risk to the pipeline.

LS-3 is a relict deep-seated landslide (Keaton and Degraff 1996) that is up to approximately 1,200 feet wide and is about 1,150 feet long. It is characterized by a weathered arcuate shaped scarp, hummocky slide body and somewhat vague lateral margins. The slide had been recently clearcut at the time of the site reconnaissance. The slide is located on a southwest facing slope of a topographical knob opposite of the Medicine to Stegosaur pipeline between Stations 55+00 and 45+00, as shown in Figure H-4. Routing of the Medicine to Stegosaur I/W pipeline considered this landslide and was based on avoidance of LS-3. Based on LS-3's topographical relationship to the pipeline, the slide does not pose a risk to the pipeline.

9.3.2.2 Well Pads

The proposed Newton well pad is situated approximately 100 feet east of the headscarp of LS-2, as shown in Figure H-4. LS-2 is a dormant-old landslide (Keaton and Degraff 1996) that was mapped by the SLIDO (DOGMAI 2021) and confirmed in the field by GeoEngineers. LS-2 is approximately 1,250 feet long and 1,200 feet wide and characterized by a steep arcuate- to sinuous-shaped scarp, gently sloping mid-slope bench, hummocky slide body and vague lateral margins. Based on a review of aerial photographs, the scarp of the slide was clearcut between 2012 and 2016, and the body of the slide was clear cut between 2017 and 2018. DOGAMI classified the landslide age as historic (>150 years). Thick blackberries and young conifer trees precluded observation of much of the landslide during a site reconnaissance conducted on March 9, 2023 (GeoEngineers 2023a). Where observable, the scarp of LS-2 is highly weathered, somewhat indistinct and inclined at gradients up to about 70 percent. Based on the activity level and age if this landslide, LS-2 presents a low risk of affecting infrastructure at the Newton Well Pad.

No landslides were identified near the Medicine well pad during the desktop review. During a site reconnaissance conducted on July 6, 2023, GeoEngineers traversed the steep slope adjacent to the northeast border of the proposed well pad. GeoEngineers did not observe indications of instability of this slope such as existing landsliding, accumulations of bowed conifer trees or ground cracking.

9.3.2.3 Powerline Alignment

The powerline alignment follows Mainline Road and an existing powerline and pipeline ROW. The route crosses LS-4 (the Lindgreen Creek Landslide) between stations 11+00 and 50+00 and passes near LS-6 located near station 65+00 along Mainline Road.

LS-4 is a large, deep-seated relict landslide (Keaton and Degraff 1996) that is characterized by a weathered sinuous-shaped scarp, mid-slope benches, hummocky topography, and well-developed internal drainage network. SLIDO mapped the northern extent of this landslide about 900 feet south of Miller Station (DOGAMI 2021); however, based on GeoEngineers' review of a LiDAR Hillshade model, GeoEngineers interprets that the landslide extends further north than the SLIDO mapping, extending just east of Miller Station, as shown in Figure H-5. Much of the landslide had been clearcut at the time of GeoEngineers' reconnaissance. Areas of the landslide that were still forested were vegetated with tall conifer trees that were growing straight. In-tact old growth stumps are present within the limits of the slide. Indications of recent movement of this landslide were not observed during the reconnaissance. Based on the landslide morphology and lack of surficial evidence of recent movement, LS-4 poses a low risk to the proposed powerline alignment.

LS-6 is a dormant-mature landslide (Keaton and Degraff 1996) located near Station 65+00. It is approximately 115 feet wide by 400 feet long and characterized by an arcuate-shaped initiation area and downslope debris deposit that is readily visible in the LiDAR hillshade model reviewed by GeoEngineers. SLIDO maps the toe landslide crossing Mainline Road (DOGAMI 2021); however, GeoEngineers' interpretation of the slide is that the deposit is located approximately 150 feet upslope (northeast) of Mainline Road. Indications of recent activity of this landslide such as bowed conifer trees, recent landslides or ground cracks within the slide mass were not observed during a site reconnaissance conducted by GeoEngineers. There is a low risk of this landslide affecting the proposed powerline.

9.3.2.4 Miller Station

There are two landslides in close proximity of the Miller Station as shown in Figure H-5; LS-4 (also called the Lindgren Creek Landslide) and LS-5 (also called the Miller Station Landslide).

Refer to Section 9.3 for a description of LS-4. Based on the landslide morphology and lack of surficial evidence of recent movement, LS-4 poses a low risk to the proposed compressor infrastructure at the Miller Staton compressor replacement area.

LS-5 is a deep-seated landslide first identified by NW Natural and GeoEngineers in October 1999 (GeoEngineers 1999a). At that time the landslide was approximately 200 feet wide by 300 feet long with an approximately 25-foot-high arcuate-shaped scarp that initiated from a gravel pad on the southeast corner of Miller Station. The landslide was caused by a combination of fill placement on the scarp of the larger Lindgren Creek Landslide, and direction of surface and near surface water to the area of the slope failure. GeoEngineers provided mitigation recommendations for the landslide (GeoEngineers 1999b, GeoEngineers 2001, GeoEngineers 2003a, GeoEngineers 2003b, GeoEngineers 2003c). The risk that the landslide posed to NW Natural was subsequently mitigated by installing drainage features and regrading the landslide. The landslide is densely vegetated with deciduous trees and young to submature conifer trees that were growing straight during GeoEngineers' reconnaissance. GeoEngineers did not observe indications of instability of LS-5 such as scarp retrogression, recent landsliding or ground cracks above the landslide.

9.3.2.5 Miller Station Annex

There is one landslide (LS-7) in close proximity to the Miller Station Storage Area as shown in Figure H-5. This landslide is a shallow earth flow road fill failure that occurred on the outboard side of Mainline Road where fill was placed across the head of a first order drainage. The landslide was characterized by an approximately 4.5-foot-high arcuate-shaped scarp that aligns with the west edge of the road, an approximately 1.5-foot-high secondary scarp that has encroached on the road, approximately 4-foot-high side margins, hummocky slide body and internal minor scarps. The slide is approximately 100 feet wide by 200 feet long.

The risk that LS-7 posed to Mainline Road was mitigated in October through December 2023 by construction of a soldier pile and lagging wall with tiebacks to stabilize the scarp of the landslide and rebuild the outboard road surface. Because the scarp of this landslide was stabilized, there is a low risk of this landslide adversely affecting the Miller Station Storage Area.

9.3.2.6 NMCS

Based on a review of SLIDO (DOGAMI 2021) and GeoEngineers' LiDAR review, no mapped landslides are located near the NMCS and in a topographic setting that could adversely affect the proposed development. Indications of landsliding within or directly adjacent to the NMCS were not observed during site reconnaissance at the site.

9.3.2.7 Broadly Mapped Landslide Deposits (QLS)

Indications of a large landslide as suggested by the mapped landslide debris that covers most of the area of the well pads, pipeline routes and NMCS were not observed during GeoEngineers' various reconnaissance. The mapping likely simply mapped broadly irregular topography in the area as potential landslide deposits, rather than mapping the many discrete landslides located in the area as shown in Figure H-4.

9.3.2.8 Summary

Table H-11 summarizes the landslides mapped by GeoEngineers for the Project that could pose a threat to the proposed well pads and pipelines, Miller Station, Miller Station Storage Area, NMCS and the powerline conduit.

Table H-11. Landslides

Site ID	Location	Proximity to Project Facilities	Landslide Classification[1]	Description	Potential Risk
LS-1	Newton to Stegosaur Pipeline route station 35+00	Approximately 180 feet southeast of proposed pipeline.	Earth Slide	Dormant-young landslide. No recent indications of movement were identified during site reconnaissance.	Low
LS-2	Northwest facing slope west of Newton well pad	Approximately 100 feet west of Newton well pad.	Earth Slide	Relict landslide. No recent indications of movement were identified during site reconnaissance.	Low
LS-3	Southwest facing slope west of Medicine well pad and south of Medicine to Stegosaur Pipeline	Approximately 600 feet west of Medicine well pad and 500 feet south of Medicine to Stegosaur pipeline.	Earth Slide	Relict landslide. No recent indications of movement were identified during site reconnaissance. Slide is located in a topographical setting such that is poses no risk to the pipeline or well pad.	No Risk
LS-4 (Lindgren Creek Landslide)	Southwest slope adjacent to Miller Station	Powerline alignment crosses landslide for approximately 3,700 feet.	Earth Slide	Large relict landslide. No recent indications of movement were identified during site reconnaissance. Monitoring of this landslide by NW Natural's monitoring program has not identified movement.	Low

Site ID	Location	Proximity to Project Facilities	Landslide Classification[1]	Description	Potential Risk
LS-5 (Miller Station Landslide)	Southeast facing slope on south end of Miller Station	Approximately 125 feet southwest of Miller Station.	Earth Slide	Landslide risk mitigated in 2003. No recent indications of movement were identified during site reconnaissance or Annual monitoring by NW Natural.	Low
LS-6	Southeast facing slope east of powerline alignment within Mainline Road	Approximately 150 feet northeast of powerline alignment Station 65+00.	Earth Flow	Mapped by the SLIDO as crossing Mainline Road, mapped by GeoEngineers up-slope of Mainline Road. No recent indications of movement were identified during site reconnaissance.	Low
LS-7	Mainline Road west of Miller Station Storage Area	Approximately 50 feet west of Miller Station Storage Area.	Earth Flow (Road Fill Failure)	Mapped and evaluated by GeoEngineers; Mitigated by soldier pile and lagging wall with tiebacks.	Low
1. Classification based on Cruden and Varnes (1996).					

9.3.3 Potential Adverse Impacts to Slope Stability

9.3.3.1 Pipelines and Powerline Installation

The proposed pipeline and powerline routes generally follow existing logging roads, or relatively gently sloping forested or recently clearcut terrain. However, relatively short portions of the Newton to Stegosaur pipeline route and powerline route traverse slopes that are in excess of 65 percent, as shown in the slope gradient maps, Figures H-6 and H-7. Where the proposed pipelines follow existing roads that are located on steep side slopes, GeoEngineers assumes based on information from NW Natural that the pipeline will be installed using only the workspace provided for by the road surface. However, there may be localized areas where excavation into steep cut slopes may be required for temporary construction workspace. These areas have not been delineated at this time. Cutting and/or filling for construction of ROW workspace on slopes in excess of 50 percent could create localized slope instability and should be evaluated on a site-specific basis prior to construction.

The following measures will be included in the final design of construction corridors along overland segments to minimize the potential to adversely affect slope stability:

- Permanent cut and fill slopes will be inclined at a maximum gradient of 2H:1V (horizontal to vertical; 50 percent).
- Permanent fill slopes (not anticipated) will be keyed into undisturbed, firm native material.

Temporary construction corridors on side slopes will be re-graded to match pre-existing topography. If these pre-existing slopes are greater than 50 percent (not anticipated), they will be replaced as structural fill in accordance with the geotechnical engineer's recommendations.

- Corridors on sloping ground will be restored with waterbars to prevent capturing, concentrating and rerouting surface water runoff. Waterbar spacing will be based on the slope gradient of the corridor as outlined in NW Natural's standard construction procedures.
- The following measures will be included in the final design of construction corridors along gravel road segments to minimize the potential to adversely affect slope stability.
    - The pipeline will be installed within the in-board, cut side of roadways and avoid road fills.
    - No fill will be placed on the outboard edges or roads.
    - If significant cut slopes are required to prepare the construction corridor (greater than 5 feet high), site specific geotechnical recommendations will be developed for design, construction, and restoration.

9.3.3.2 NMCS

The proposed NMCS construction area is relatively flat except for existing cut slopes located on the south side of the construction area. As currently envisioned, the NMCS construction area will be graded (cut) to match the elevation of the adjacent developed NMCS. This grading will require placement of fill on gentle to moderate slopes adjacent to the NMCS or hauling the excavated soils offsite to an approved disposal area. The following measures will be included in the final design and construction of the NMCS to minimize the potential to adversely affect slope stability:

- Permanent cut and fill slopes within the NMCS (if required) will be inclined at a maximum gradient of 2H:1V (horizontal to vertical).
- Fill placed on slopes inclined greater than 20 percent on slopes surrounding the NMCS, if required, will be keyed into undisturbed, firm native material. Final fill slopes will be inclined at 3H:1V (33 percent) or less. If steeper final fill slopes are required, the material will be placed as structural fill in accordance with the NMCS geotechnical engineering report and final fill slopes will not exceed 2H:1V (50 percent). In addition, no fill will be placed on slopes inclined greater than 40 percent.
- Design and construction will follow the geotechnical recommendations presented in the final geotechnical report for the site.

9.3.3.3 Miller Station

The Miller Station compressor replacement area is located within the existing Miller Station on a relatively flat surface. However, in-active landslides are located just outside of the boundary of

Miller Station as previously discussed. The following measures will be included in the final design and construction of the Miller Station to minimize the potential to adversely affect slope stability.

- No earthwork will occur outside the boundaries of the Miller Station site.
- Construction will not impact existing drainage systems installed to mitigate the Miller Station Landslide.
- Drainage paths within Miller Station will not be altered.
- No fill is anticipated to be required for replacement of the compressors; however, excavations for new compressor station foundations and utility trenches will be required. Spoils from these excavations will not be placed on or within 50 feet of the Miller Station or Lindgren Creek Landslides.
- Design and construction will follow the geotechnical recommendations presented in the final geotechnical report for the site.

9.3.3.4 Well pads and Miller Station Storage Area

The proposed well pads and Miller Station Storage Area are located on stable ridge lines and hill tops that will require grading to prepare the sites. Grading will likely require construction of cut and fill slopes to prepare relatively flat gravel surfaces at each well pad and the Miller Station Storage Area. The following measures will be included in the final design and construction of the well pads and Miller Station Storage Area to minimize the potential to adversely affect slope stability.

- No earthwork will occur outside the boundaries of the well pad or Miller Station Storage Area sites.
- A geotechnical investigation and associated report will be prepared for each well pad and the Miller Station Storage Area to provide design and construction recommendations. Recommendations will include cut and fill slope gradients, fill slope preparation and compaction recommendations, typical earthwork recommendations (site stripping, fill materials, use of onsite soils as structural fill, utility trench recommendations) and gravel thickness recommendations appropriate for the anticipated use.
- Fill will not be placed at the heads of stream drainages.

9.4 Non-Seismic Hazard Mitigation

9.4.1 Erosion

Where the proposed pipelines and powerline alignment follow existing gravel roads, erosion is expected to be minimal, and no special mitigation will be required. In overland segments, the pipeline, and powerline permanent and temporary construction easements will be relatively narrow (40 feet wide and 80 feet wide, respectively) and will be protected from erosion using current erosion control best management practices. Best management practices will also be used during and after construction of the NMCS, Miller Station, Miller Station Storage Area and proposed well pads. A detailed erosion and sediment control plan will be completed to fulfill requirements of the National Pollutant Discharge Elimination System (NPDES) Permit 1200-C. Erosion control measures that may be employed during and after construction are discussed in detail in Exhibit I but generally include:

- Installing sediment fence or other approved best management practices at downslope side of excavations and disturbed areas.
- Straw mulching within disturbed cross-country segments of the pipeline and powerline corridors and locations adjacent to roads that have been affected during construction.
- Planting designated seed mixes within disturbed cross-country segments of the pipeline and powerline corridors at affected areas adjacent to the road.
- Planting designated seed mixes or hydroseeding of cut and fill slopes at the well pads and Miller Station Storage Area.
- Waterbars along cross country segments of the pipeline and powerline routes.
- Restoration of gravel surfacing along roadways.
- Gravel surfacing within well pads and the Miller Station Storage Area.

Exposed soil areas that are affected by the construction will be seeded after construction when there is adequate soil moisture. These areas will be reseeded in the spring if a healthy cover crop does not grow. The sediment fences will remain in place until the affected areas are well vegetated.

Whenever feasible, overland corridors will be constructed with waterbars so that surface drainage continues to natural drainage patterns, with minimal diversions through ditches and culverts. Regular maintenance of drainage facilities will ensure continued proper operation.

9.4.2 Groundwater

Shallow groundwater is not expected to be a concern during construction in upland areas of the Project. If localized groundwater is encountered during open trench construction of the powerline, the trench will be dewatered, the effluent will be treated, if necessary, and discharged onsite through filter bags or other similar water discharge structures.

No mitigation is deemed necessary for Newton to Stegosaur, Medicine to Stegosaur and Stegosaur to NMCS pipeline alignments the NMCS, the Miller Station and Miller Station Storage Area since these will be located on upland areas and ridge tops. No mitigation is deemed necessary for the powerline alignment within the Nehalem River Valley because the pipeline will either be installed within the fill prism of Mainline Road (which is higher than the adjacent natural ground surface) or will be installed via HDD trenchless methods where the bore hole will resist buoyant forces.

9.4.3 Landslides

9.4.3.1 Landslide Hazard Mitigation Discussion

Three new landslides (LS-1, LS-2, and LS-6) were identified by this study that present a low risk to the proposed Newton to Stegosaur Pipeline, proposed Newton Well Pad, or proposed powerline. One landslide near the Miller Station Storage Area, LS-7, had been identified during this study and was mitigated. This landslide presents a low risk to the Miller Station Storage Area. Two landslides that are already being monitored by NW Natural (LS-4 and LS-5) present a low risk to the Miller Station and proposed powerline. Because of the low-risk nature of these landslides, no landslide specific mitigation other than periodic monitoring is recommended.

Although indications of instability or recent movement of fill slopes along the proposed pipeline alignments were not observed during site reconnaissance, in GeoEngineers' experience road related fill slopes in mountainous terrain are more susceptible to landsliding than cut sections of a roadway. As such, there is a heighted risk of future failure of the gravel road fill slopes adversely affecting the proposed pipelines. As such, NW Natural will install the proposed Newton to Stegosaur, Medicine to Stegosaur I/W pipelines within the cut side (inboard, upslope) of the roads.

9.4.3.2 NW Natural's Landslide Hazard Monitoring Program

NW Natural developed a landslide risk ranking and monitoring program for landslides and well pads that may affect their transmission pipelines and well pads, and manages those landslides in a GIS database. In general, the program classifies landslide risk to NW Natural's pipelines and well pads into categories of high, moderate, and low, and then establishes a monitoring schedule for landslides placed into those categories accordingly. NW Natural's landslide risk and monitoring schedule is presented below.

Landslide Risk

High Risk

- Pipeline crosses or well pad is located within a landslide mass or is within the landslide expansion hazard zone; and
- Surficial, geomorphic, and vegetative features suggest that the landslide is active or recently active; and
- If the landslide is instrumented with inclinometers and/or surface survey monuments:
    - Greater than 1 inch of movement was measured annually over at least two winters; or
    - The landslide has not been monitored through at least two winters.

Moderate Risk

- Pipeline crosses or well pad is located within a landslide mass or is within the landslide expansion hazard zone; and

- Surficial, geomorphic, and vegetative features suggest that the landslide is dormant; or
- Surficial, geomorphic, and vegetative features suggest that the landslide moves at a slow rate (<1 inch/year) and rapid movement is unlikely; or
- If the landslide is instrumented with inclinometers and/or surface survey monuments, less than 1 inch of movement is measured annually over at least two winters of above average rainfall.

Low Risk

- Pipeline or well pad is outside the landslide expansion hazard zone of a potentially active landslide/dormant-young landslide; or
- Pipeline crosses a landslide that is inactive or moves at a very slow and predictable rate based on one or all of the following criteria:
    - Surficial, geomorphic, and vegetative features suggest the landslide is dormant-mature; or
    - The apparent cause of the landslide has been removed or the landslide has been stabilized (i.e., drainage improvements, grading); or
    - Instrumentation confirms that less than ¼ inch of movement has occurred annually for at least two winters of above-average rainfall.

<u>Monitoring Schedule</u>

Monitoring includes visual surface observation of the pipeline ROW or well pad and adjacent areas, and reading of instrumentation (inclinometers/strain gauges, if installed within a known landslide) in accordance with the monitoring schedule presented below. Inclinometer and strain gauge data is evaluated by an outside consultant. If during a monitoring event surface conditions suggest that movement may have occurred within the pipeline ROW, NW Natural will evaluate the newly reported surface indications of landslide movement or obtain a consultant to evaluate the reported movement further.

- **(F) Frequently (High Risk Landslides)**
    - At least once per month from October[2] through April.
    - Within 48 hours after 4 or more inches in 48 hours.
    - Within 48 hours after 6 or more inches in 7 days.
    - Within 48 hours after a rain on snow event.
    - Immediately after an earthquake that generates PGAs in excess of 0.1g along the corridor.

[2] If the landslide is instrumented with inclinometers or strain gauges, an initial fall reading should be taken earlier if precipitation conditions warrant.

- **(P) Periodically (Moderate Risk Landslides)**
  - At least twice from October[1] through April.
  - Within 48 hours after 4 or more inches in 48 hours.
  - Within 48 hours after 6 or more inches in 7 days.
  - Within 48 hours after a rain on snow event.
  - Immediately after an earthquake that generates PGAs in excess of 0.1g along the corridor.
- **(A) Annually (Low Risk Landslides)**
  - At least once per year during the winter.
  - Within 48 hours after 25 or more inches in 30 days.
  - Immediately after an earthquake that generates PGAs in excess of 0.1g along the corridor.

Although the risk and hazard classifications above indicate that NW Natural will monitor the low-risk landslides at least once yearly, the staff at Miller Station are onsite at Miller Station and the NMCS daily, travel along the gravel road portion of the proposed powerline route daily and along gravel roads where the pipelines are proposed weekly to visit well sites in the area. In addition, NW Natural staff will visit the proposed well pads on a weekly basis in accordance with their well monitoring program. As such, landslide movement across the proposed pipelines and powerline route, or at the well pads, NMCS and Miller Station could be quickly identified. These staff were most recently provided landslide hazard training on July 10 and August 1, 2023, and will be periodically re-trained in landslide identification. The landslide identification training includes a 4-hour class conducted by GeoEngineers and NW Natural. The class discusses landslide classification, causes and field identification, reporting of a landslide, hazard assessment, landslide mitigation and landslide prevention. The field identification and reporting aspects of the training are stressed for all field staff responsible for landslide monitoring.

10.0 Disaster Resilient Design – OAR 345-021-0010(1)(h)(F)(i)

> *(i) An explanation of how the applicant will design, engineer, construct and operate the facility to integrate disaster resilience design to ensure recovery of operations after major disasters; and*

The primary natural disaster that could affect the Project's infrastructure is earthquake shaking. As discussed elsewhere in this report, modern natural gas pipelines utilize arc welded joints that perform well during seismic shaking events. NW Natural will require certified welders to perform arc welding of pipe joints and at tie in locations and will test welds using non-destructive X-ray testing to verify the competency of the welds. In addition, the pipeline route design avoids known landslides and active faults which limits the risk of permanent ground deformation along the pipeline during a seismic event.

The proposed powerline will be installed underground, which mitigates potential powerline related with wind and fire hazards. In addition, the pipeline will be installed in flexible HDPE conduits, which allows flexibility if landslide movement, seismic shaking or liquefaction should occur that may affect the powerline during its lifespan.

Structures at the NMCS and Miller Station will be designed to meet current structural specialty codes considering earthquake loading.

11.0 Potential Impact of Future Climatic Conditions – OAR 345-021-0010(1)(h)(F)(ii)

> *(ii) An assessment of future climate conditions for the expected life span of the proposed facility and the potential impacts of those conditions on the proposed facility.*

Future climatic conditions in the pacific northwest are estimated to result in an average increase of annual temperatures of 1.1^0 C (2.0^0 F) by the 2020s, 1.8^0 C (3.2^0 F) by the 2040s and 3.0^0 C (5.3^0 F) by the 2080s (Mote & Salathé 2010). Projected changes in annual precipitation are expected to be relatively minor (+ 1 percent to + 2 percent) with more precipitation occurring in fall and winter and less precipitation occurring in summer (Mote et.al. 2005, Mote & Salathé 2010). As such, the expected result would be a relatively minor increase in rainfall precipitation and a decrease in snowfall precipitation over the next century and a significant rise in sea level. The average water flow (and level) of rivers in the region would also increase. Because the Project is not located in a coastal zone and no above-ground structures are planned within flood plain environments, GeoEngineers does not expect that sea level rise, or an increased incidence of flooding, would impact the Project.

Increased precipitation could affect the landslides identified near the Project Area in that the landslides would have a higher risk of becoming reactivated. In addition, increased precipitation could destabilize otherwise stable slopes, creating a higher incidence of new landslides. However, as discussed elsewhere in this report, the proposed pipelines, well pads, compressor stations and powerline alignments will be periodically monitored, and NW Natural has an established landslide hazard monitoring program that will monitor landslides identified by this study that pose a risk to the proposed infrastructure. As such, if landslide movement of existing landslides or along the pipeline or powerline routes occur, there is a high likelihood that the land movement will be identified and the risk to the proposed infrastructure could be mitigated before a landslide adversely affects the proposed infrastructure.

12.0 Limitations

This exhibit has been prepared for use by NW Natural and other members of the design team involved with the Project. The exhibit is not intended for use by others, and the information contained herein is not applicable to other sites. The data and exhibit should be provided to prospective contractors, but this exhibit, conclusions and interpretations should not be construed as a warranty of the subsurface conditions. The conclusions and recommendations in this exhibit should be applied in their entirety.

Variations in subsurface conditions from those found during this research are possible. Subsurface conditions may also vary with time. A contingency for unanticipated conditions should be included in the Project budget and schedule for such an occurrence. GeoEngineers recommends that sufficient monitoring, testing and consultation be provided during construction to confirm that the conditions encountered are consistent with those indicated by this research, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork and pipeline installation activities comply with contract plans and specifications.

The scope of GeoEngineers' services does not include services related to construction safety precautions. These recommendations are not intended to direct the contractor's methods, techniques, sequences or procedures, except as specifically described in this report for consideration in design.

Within the limitations of scope, schedule and budget, GeoEngineers' services have been executed in accordance with generally accepted practices in this area at the time the report was prepared. No warranty or other conditions, express or implied, should be understood.

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Figures

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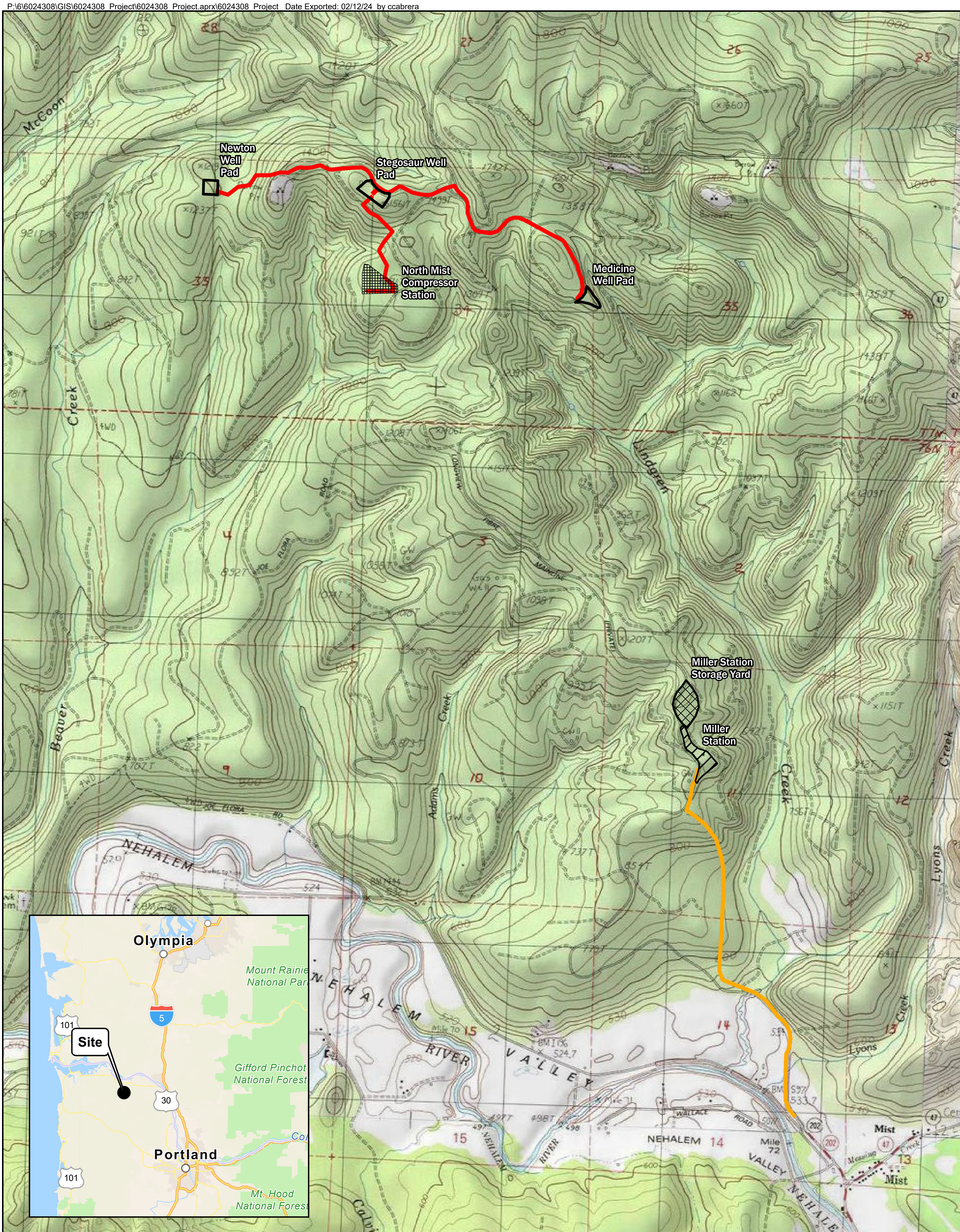

Legend

- Proposed Injection/Withdrawal Pipelines
- Proposed Underground Powerline
- Proposed Well Pad
- Miller Station
- North Mist Compressor Station
- Miller Station Storage Yard

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Feet

Source(s):
- USGS Topo

Coordinate System: NAD 1983 StatePlane Oregon North FIPS 3601 Feet

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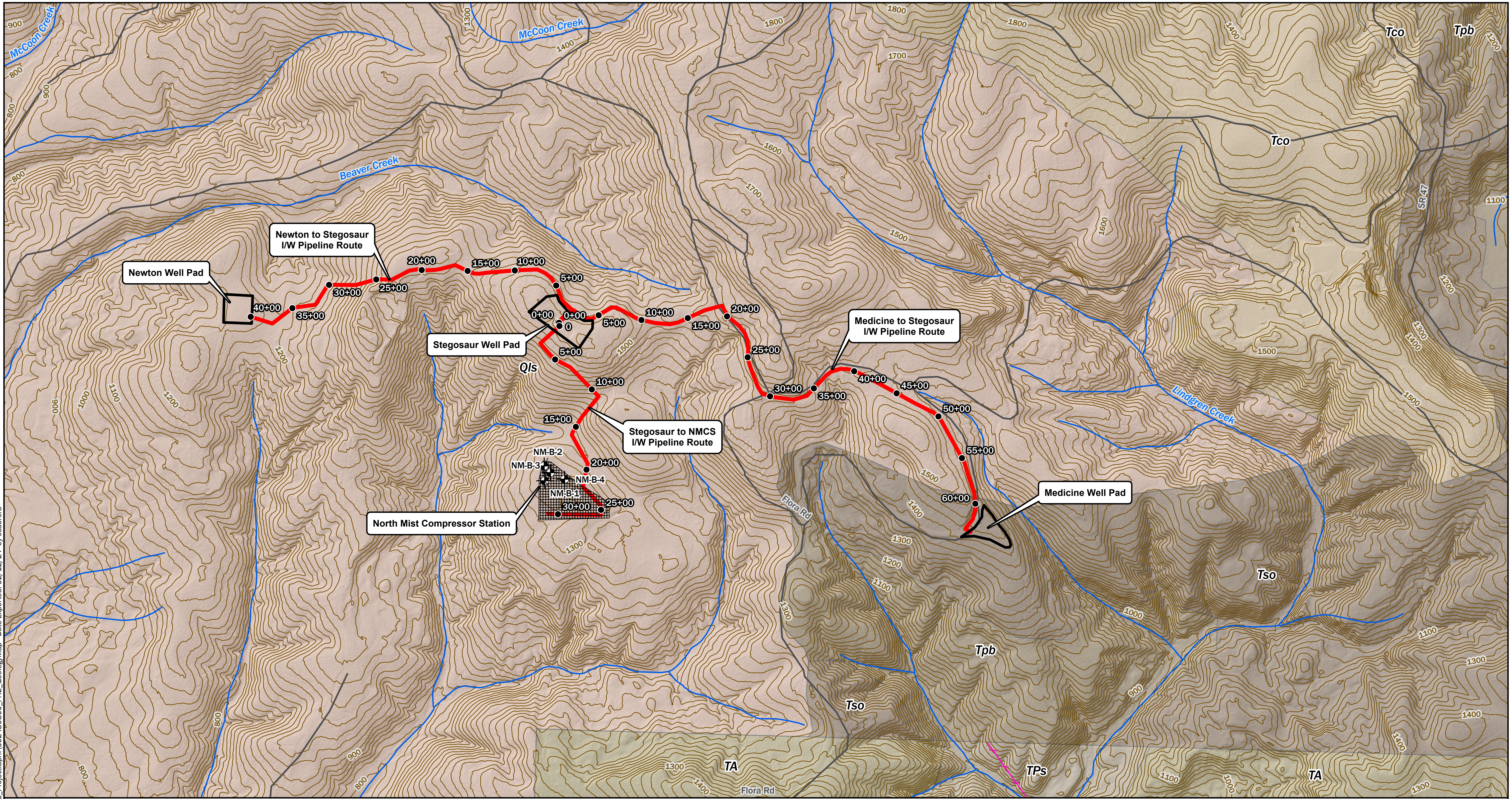
Vicinity Map

Mist Resiliency Project
Columbia County, Oregon

GeoEngineers

Figure H-1



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Source(s):
- Lidar and Geology from DOGAMI's Oregon Geologic Data Compilation, (OGDC-6).
- Water features from USGS, National Hydrography Dataset.
- 2014 ESRI Street Map and ArcGIS Online,

Coordinate System: NAD 1983 StatePlane Oregon North FIPS 3601 Feet

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Legend
- Stationing
- Boring
- Proposed Injection/Withdrawal (I/W) Pipeline
- Proposed Well Pad
- North Mist Compressor Station
- Streams
- Contours (10 foot)
- Faults

Surficial Geology
- Qls,Landslide debris
- TA,Astoria Formation
- TPs,Pittsburg Bluff Formation, siltstone member
- Tco,Columbia River Basalt Group
- Tpb,Pittsburg Bluff Formation
- Tso,Scappoose Formation
- TK,Keasey Formation
- TPl,Pittsburg Bluff Formation, laminated member
- TPs,Pittsburg Bluff Formation, siltstone member

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Feet

Geology Map - Well Pad, Pipelines and NMCS

Mist Resiliency Project
Columbia County, Oregon

GEOENGINEERS

Figure H-2

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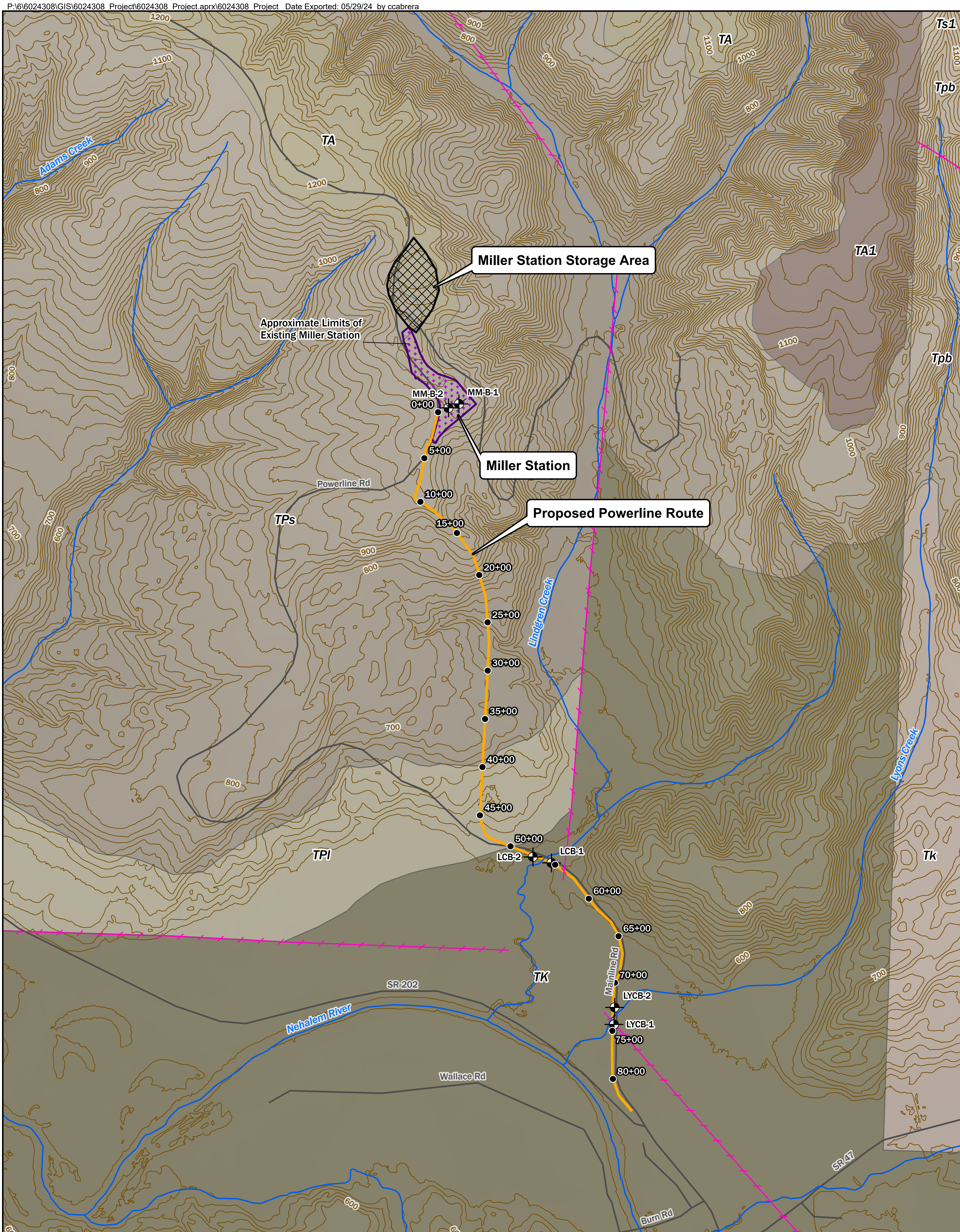

Legend

- Stationing
- Boring
- Proposed Powerline Route
- Approximate Limits of Existing Miller Station
- Miller Station Storage Area
- Streams
- Faults

Surficial Geology

- TA,Astoria Formation
- TA1,Astoria Formation
- TK,Keasey Formation
- TPl,Pittsburg Bluff Formation, laminated member
- TPs,Pittsburg Bluff Formation, siltstone member
- Tk,Keasey Formation
- Tpb,Pittsburg Bluff Formation, East Fork Member
- Ts1,Scappoose Formation

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Feet

Source(s):
- Lidar and Geology from DOGAMI's Oregon Geologic Data Compilation, (OGDC-6).
- Water features from USGS, National Hydrography Dataset.
- 2014 ESRI Street Map and ArcGIS Online.

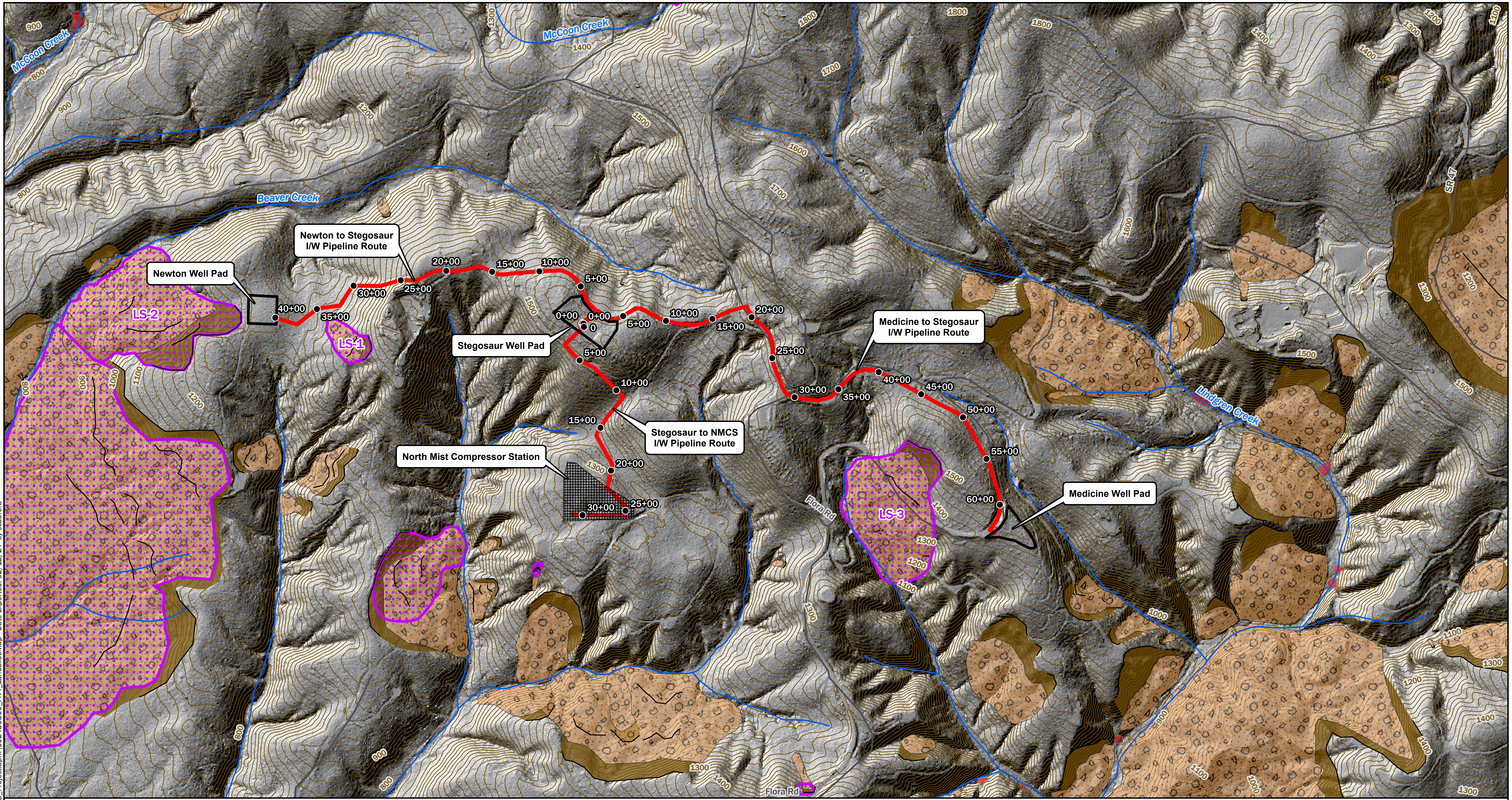
Coordinate System: NAD 1983 StatePlane Oregon North FIPS 3601 Feet

Geology Map - Miller Station/Powerline Route

Mist Resiliency Project
Columbia County, Oregon

GEOENGINEERS

Figure H-3



Legend

- Stationing
- Proposed Injection/Withdrawal (I/W) Pipeline
- Proposed Well Pad
- North Mist Compressor Station
- Streams
- Contours (10 foot)
- Mapped Landslide (GeoEngineers)

DOGAMI Landslide

- Fan (DOGAMI)
- Landslide (DOGAMI)
- Head Scarp (DOGAMI)
- Scarp (DOGAMI)

N

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Feet

Landslide Map - Well Pads, Pipelines and NMCS

Mist Resiliency Project
Columbia County, Oregon

GeoEngineers

Figure H-4

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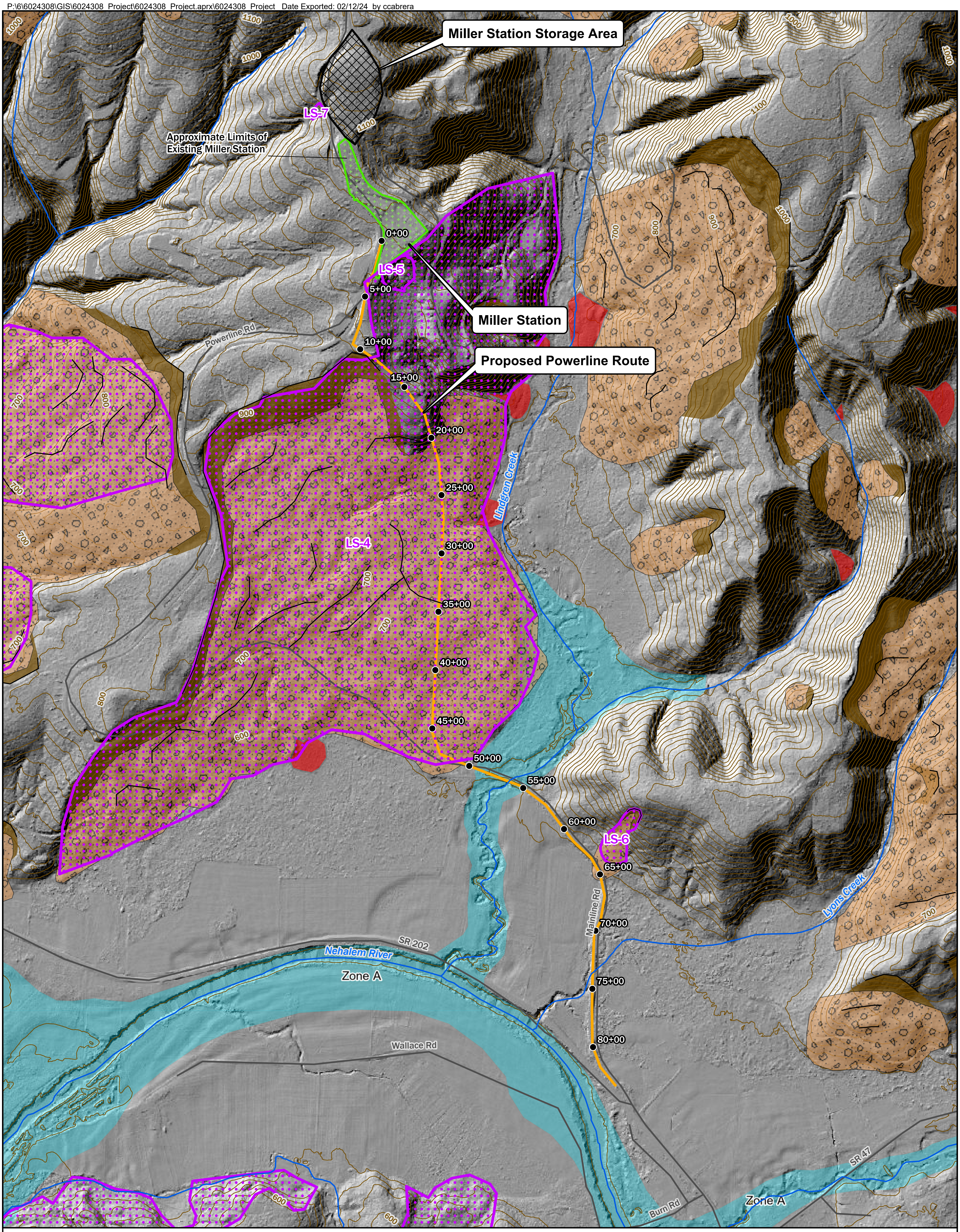


Legend

- Stationing
- Boring
- Proposed Powerline Route
- Approximate Limits of Existing Miller Station
- Miller Station Storage Area
- Streams
- Contours (10 foot)
- FEMA Mapped Flood Plain
- Mapped Landslide (GeoEngineers)

DOGAMI Landslide

- Fan (DOGAMI)
- Landslide (DOGAMI)
- Head Scarp (DOGAMI)
- Scarp (DOGAMI)

N

0 800

Feet

Source(s):
- Landslides from DOGAMI Slido v 3.4.
- Water features from USGS, National Hydrography Dataset.
- FEMA Floodplain Data
- 2014 ESRI Street Map

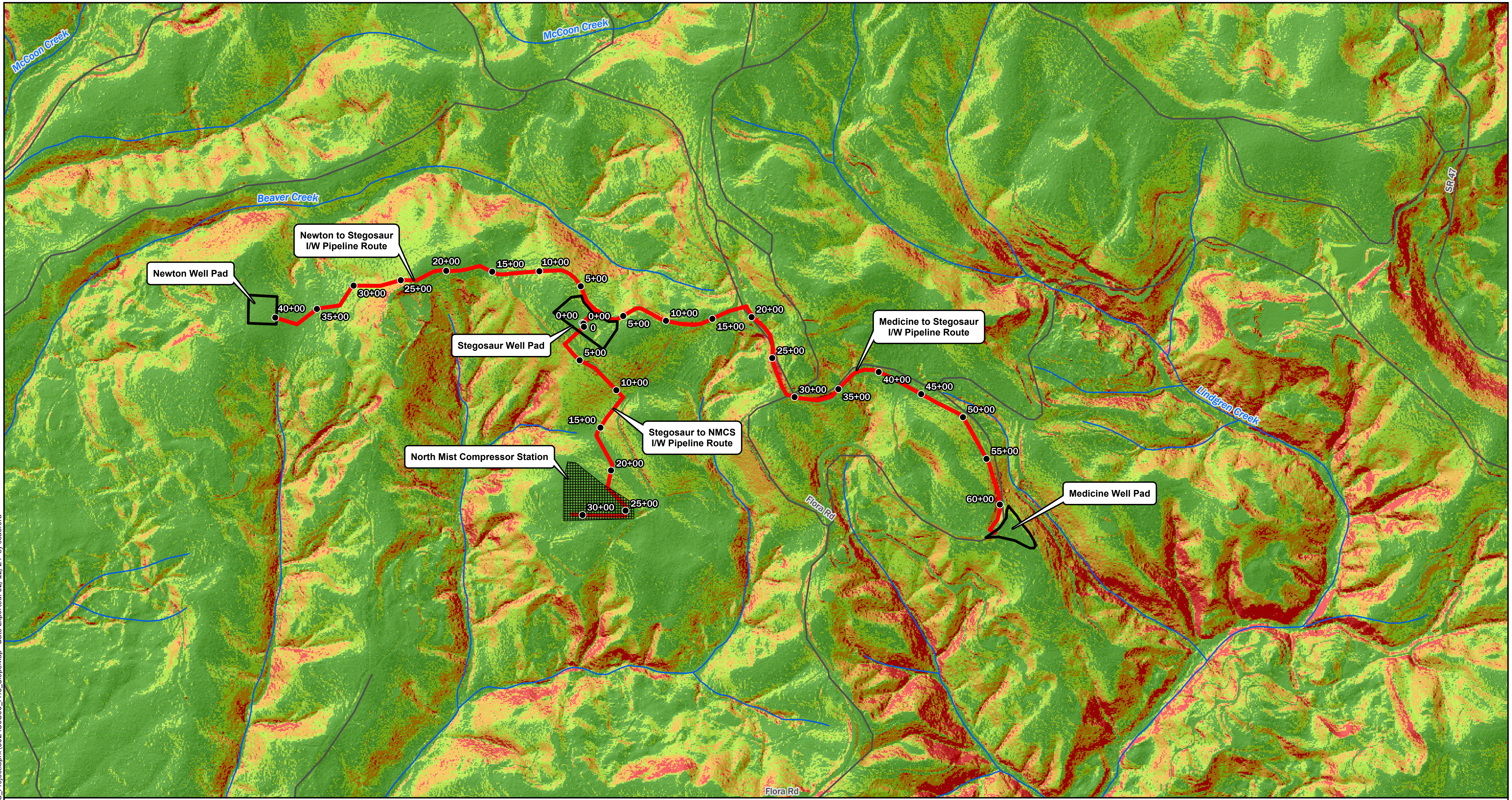
Coordinate System: NAD 1983 StatePlane Oregon North FIPS 3601 Feet

Landslide Map - Miller Station and Power Line Route

Mist Resiliency Project
Columbia County, Oregon

GeoEngineers

Figure H-5



Source(s):
- Lidar and Geology from DOGAMI's Oregon Geologic Data Compilation, (OGDC-6).
- Water features from USGS, National Hydrography Dataset.
- 2014 ESRI Street Map

Coordinate System: NAD 1983 StatePlane Oregon North FIPS 3601 Feet

Disclaimer: This figure was created for a specific purpose and project. Any use of this figure for any other project or purpose shall be at the user's sole risk and without liability to GeoEngineers. The locations of features shown may be approximate. GeoEngineers makes no warranty or representation as to the accuracy, completeness, or suitability of the figure, or data contained therein. The file containing this figure is a copy of a master document, the original of which is retained by GeoEngineers and is the official document of record.

Legend

- Stationing
- Proposed Injection/Withdrawal (I/W) Pipeline
- Proposed Well Pad
- North Mist Compressor Station
- Streams

Slope

- 0 - 30%
- 30 - 50%
- 50 - 80%
- >80%

N

0 1,000
Feet

Slope Map - Well Pads, Pipelines and NMCS

Mist Resiliency Project
Columbia County, Oregon

GEOENGINEERS

Figure H-6

P:\6\6024308\GIS\6024308_Project\6024308_Project.aprx\602430800_FH6_SlopeMap Date Exported: 02/12/24 by ccabrera

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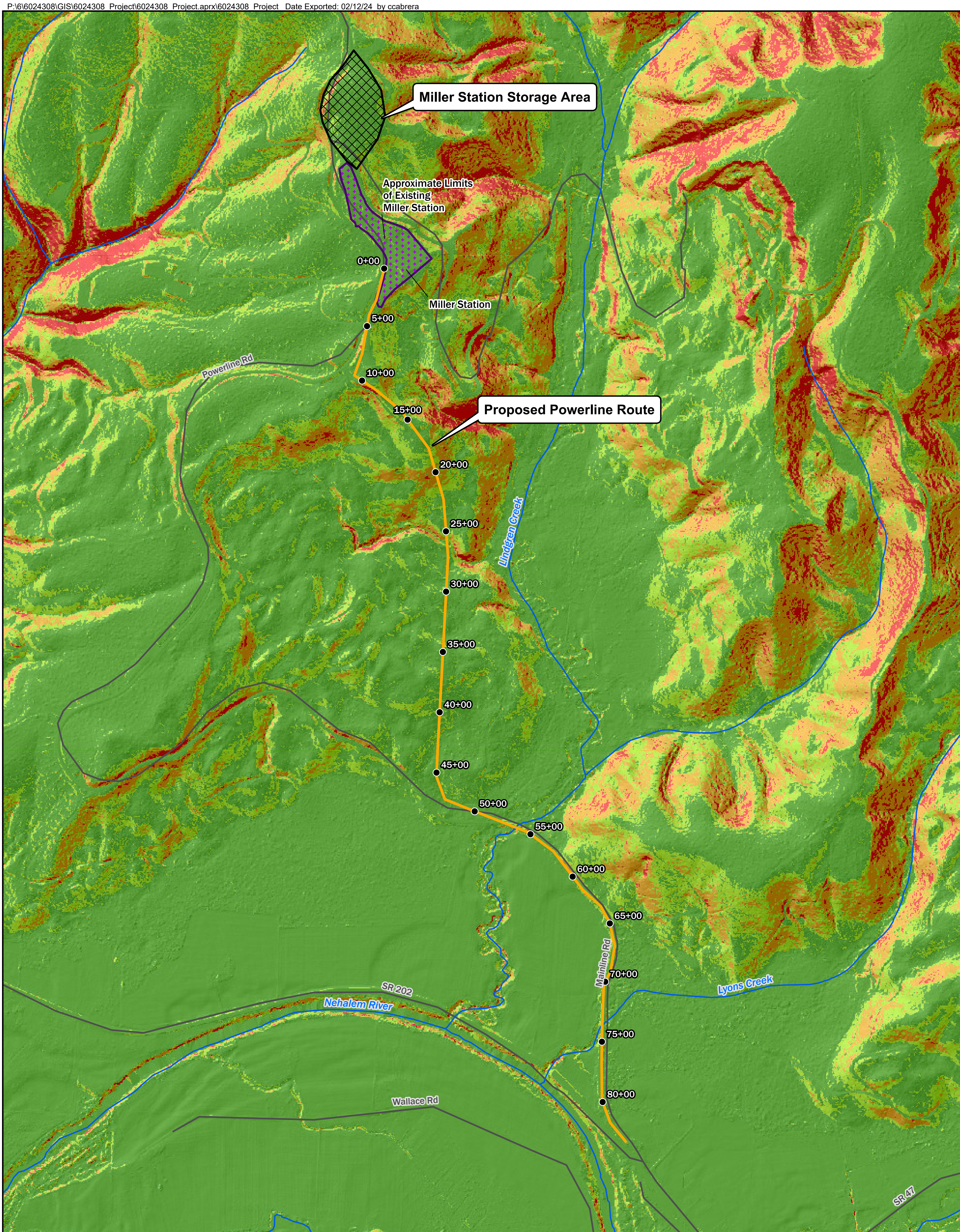

Legend

- Stationing
- Streams
- Approximate Limits of Existing Miller Station
- Miller Station Annex

Slope

- 0 - 30%
- 30 - 50%
- 50 - 80%
- >80%

N

0 800

Feet

Source(s):
- Lidar from DOGAMI.
- Water features from USGS, National Hydrography Dataset.
- 2014 ESRI Street Map

Coordinate System: NAD 1983 StatePlane Oregon North FIPS 3601 Feet

Slope Map - Miller Station and Power Line Route

Mist Resiliency Project
Columbia County, Oregon

GeoEngineers

Figure H-7

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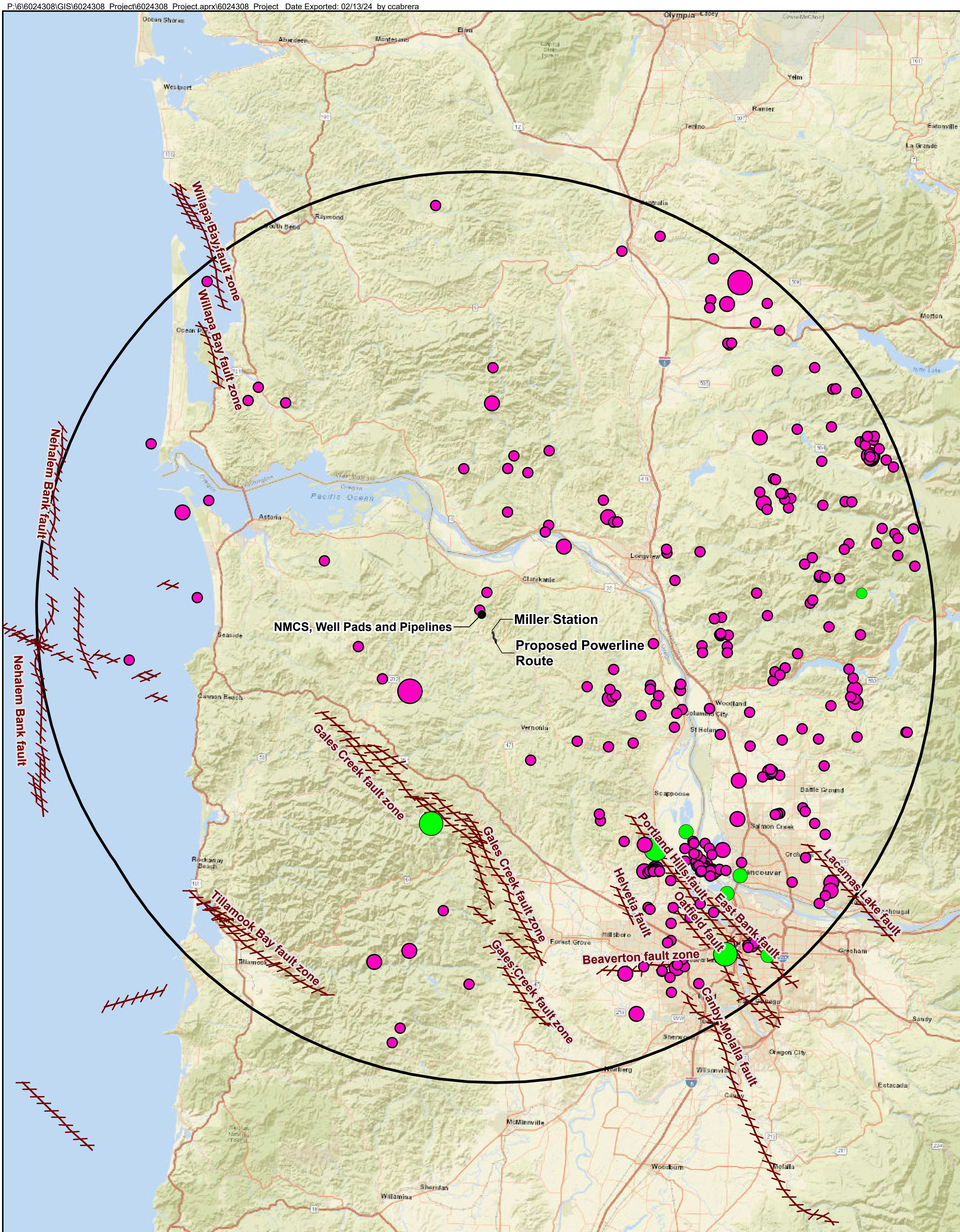

Legend

50 Mile Buffer
USGS Quaternary Faults

USGS Historic Earthquakes (Magnitude)
- < 2
- 2 - 3
- 3 - 4
- 4 - 5
- < 5

U.S. Earthquake Intensity Database 1941-1968 (MMI)
- IV
- V
- VI

N

0 11
Miles

Source(s):
- Active Faults (2010) and USGS Historic Earthquakes (2023) locations from United States Geological Survey (USGS). Earthquake intensity database maintained by NOAA (2013).
- ESRI

Coordinate System: NAD 1983 HARN Oregon Statewide Lambert Feet Intl

Historical Earthquake and Quaternary Fault Map

Mist Resiliency Project
Columbia County, Oregon

GeoEngineers

Figure H-8

Attachment H-1.
Evidence of Consultation with DOGAMI

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From:	MCCLAUGHRY Jason * DGMI
To:	Brian C. Ranney
Cc:	Andy Bauer (andrew.bauer@nwnatural.com); ESTERSON Sarah * ODOE
Subject:	Re: Evidence of Consultation with DOGAMI - NW Natural"s Mist Resiliency Project
Date:	Thursday, September 28, 2023 7:11:09 AM

Hi Brian:
I concur that your assessment of the meeting is accurate. One correction is needed under the header Geologic Hazard Assessment where you talk about replacing OGDC maps with smaller scale maps. What you mean are larger scale, more detailed maps. For example a 1;8000 scale map has a larger scale than 1:100,000 scale map.
Thanks for such a great outline and discussion, it's very helpful.

Best Regards,
Jason

Jason D. McClaughry, R.G.
Geological Survey and Services Program Manager
Oregon Department of Geology and Mineral Industries
Baker City|1995 3rd Street, Suite 130|Baker City, Oregon 97814
Portland|800 NE Oregon Street, Suite 965|Portland, Oregon 97232
Cell: (541) 519-3419
jason.mcclaughry@dogami.oregon.gov | www.oregon.gov/dogami

On Sep 26, 2023, at 12:05 PM, Brian C. Ranney <branney@geoengineers.com> wrote:

Good afternoon Jason,

The purpose of this email to provide evidence of consultation with DOGAMI for NW Natural's Mist Resiliency Project in accordance with OAR 345-021-0010(h)(B). NW Natural's project team met with Jason McClaughry of DOGAMI and Sarah Esterson of the Oregon Department of Energy on Thursday September 21, 2023 to discuss the project and consult with DOGAMI regarding our methods for evaluating geologic for the project. The text below summarizes our agenda and discussion during the meeting.

Text in italics notes discussions that occurred during our meeting and DOGAMI recommendations for evaluating geologic hazards for the project.

We respectfully request that you reply to this email with your agreement on the discussions below. You reply will be included with our report as evidence of consultation. If there is anything I left out, or inaccurately summarized, please let us know.

1. Attendees
    a. Brian Ranney, CEG - GeoEngineers
    b. Andrew Bauer, RG - NW Natural
    c. Sarah Esterson, Oregon Department of Energy
    d. Jason McClaughry, Oregon Department of Geology and Mineral Industries
2. Project Description (Bauer)
    a. North Mist Compressor Station (NMCS)
    b. Miller Station
    c. Well Pads – Newton, Stegosaur, Medicine
    d. Injection/Withdrawal Pipelines – Newton to Stegosaur, Medicine to Stegosaur, Stegosaur to NMCS
    e. Power Line.
        i. *Discussed that the power line would cross Lindgren Creek and Lyons Creek using HDD methods. Site specific HDD designs will be produced.*
3. Seismic Hazard Assessment
    a. Methodology and Scope (completed for Exhibit H)
        i. Evaluated seismic hazards for well pads, injection/withdrawal pipelines, NMCS, Miller Station and Power line.
        ii. Evaluated potential presence of faults using USGS fault and fold database, and by LiDAR review.
            1. *Discussed several faults mapped by ODGC near the power line alignment. However, these faults are not included in the USGS fault and fold database and therefore we conclude that these faults likely offset older sedimentary rocks on the area and not recent alluvium and therefore are not considered active. Mr. McClaughry agreed with this assessment.*
        iii. Obtained list of recorded earthquakes within approximately 50 miles of the proposed route using USGS earthquake catalog search.
        iv. Evaluated contributing earthquake sources using USGS Probabilistic Seismic Hazard Mapping Tool.
        v. Evaluated peak ground accelerations (PGA) for 475-year (10% probability of exceedance in 50 years), 2,475-year (2% PE) and 4,975-year (1% PE) recurrence intervals using USGS unified hazard tool.
        vi. Evaluated risk of liquefaction and lateral spreading considering topographic setting and groundwater conditions of site (liquefaction/lateral spreading hazards not expected).
        vii. Evaluated potential adverse effects of earthquake shaking on infrastructure such as earthquake induced landsliding, fault

rupture, liquefaction etc. as required by OAR.

b. Site Specific Geotechnical Work (Application Phase, also discussed later in agenda)
    i. Borings completed at NMCS and Miller Station to evaluate geological conditions.
    ii. Borings included downhole seismic testing to inform seismic hazard evaluation and structural design.
    iii. Completed geotechnical reports for NMCS and Miller Station

c. Site-specific geotechnical work (pre-construction phase)
    i. No preconstruction phase site-specific seismic related geotechnical work is anticipated because it has been completed for Exhibit H and site-specific geotechnical work at NMCS and Miller Station.

4. Geologic Hazard Assessment
    a. Methodology and Scope
        i. General Geologic Hazards Evaluated
            1. Landslides
            2. Flooding
            3. Seismic (fault rupture, ground shaking. seismically induced landsliding, liquefaction, Coseismic subsidence)
            4. Steep slopes
            5. groundwater
            6. Tsunami and volcanic hazards not applicable.
        ii. Geology: Desktop Review
            1. Utilized published mapping and report to describe geologic setting, stratigraphy, geologic structure, site geology, geologic unit stability, soils and groundwater.
            2. Sources included: Oregon Geologic Data Compilation (ODGC) mapping, Statewide Landslide Information Database (SLIDO), published oil and gas investigation geologic mapping, published reports and unpublished masters theses, geoengineers reports in the area.
                a. *We discussed utilizing smaller scale mapping of an oil and gas investigation conducted in the area as a supplement to the ODGC mapping, specifically because the ODGC mapping notes a relatively large area of "landslide Deposits" in the project area that is not related to any specific landslide. Mr. McClaughry agreed that using smaller scale mapping is appropriate to better define mapped geology in the area as the ODGC mapping is a state-wide summary map of geologic conditions. The smaller scale mapping is more site specific and likely provides more accurate representation of geology in the area.*
        iii. Evaluated topographic conditions based on LiDAR generated slope maps, topographic maps and site reconnaissance.
        iv. Landsliding: Desktop review
            1. Reviewed existing reports in GeoEngineers' files. Including Exhibit H for the North Mist Pipeline project (includes existing North Mist Compressor Station), Miller Station control building

Expansion and others.

2. Reviewed GeoEngineers' landslide reports in the area. We have extensive experience in the area of the project.
3. Reviewed topographic conditions along alignment.
4. Reviewed current geologic mapping.
5. Reviewed current SLIDO.
6. Evaluated LiDAR hillshade model to map potential landslides.
7. Mapped identified landslides; those identified by DOGAMI and GeoEngineers.

v. Site Reconnaissance – landslides and soils

1. Walked injection/withdrawal pipeline routes – routed Newton to Stegosaur to avoid known landslide. Otherwise, no landslides along pipeline routes.
2. Walked Power Line Route. Route crosses a large, relict, deep-seated landslide. We looked for surface indications of activity; none observed.
3. Evaluated potential activity of nearby landslides identified from desktop review (Specifically landslides identified along Newton to Stegosaur Injection/Withdrawal Pipeline route, Medicine to Stegosaur Injection/Withdrawal Pipeline route, power line route, Miller Station)
4. Looked for surface indications of localized landsliding, particularly where pipelines routes pass near heads of 1st order streams.
5. Observed surficial soils for assessment of potential soil related hazards.
6. Observed outcrops for correlation with geologic mapping.

vi. Site Specific Geotechnical Work (Application phase).

1. Completed sitting study for Newton Well Pad.
2. Performed routing analysis and site recon along overland segment of Newton to Stegosaur pipeline route.
3. We do not see the need for site specific geotechnical work to be completed for general geologic hazard evaluation (landslides) because infrastructure avoids landslides and relict nature of landslides identified by our study.
4. We will use existing reports (completed by GeoEngineers) in the area to describe anticipated subsurface conditions and applicable landslide evaluations (Miller Station, newton well pad, power line route).
5. Site specific geotechnical work has been completed for NMCS and Miller Station (discussed later).

vii. Site Specific Geotechnical Work (pre-construction)

1. No pre-construction geotechnical work is anticipated to evaluate general geologic hazards.
2. Site-specific geotechnical work will be completed for well pad development when grading is known (discussed later)
3. Site-specific geotechnical work may be conducted for pipeline routes depending on construction corridor configuration (discussed later).
4. Site-specific geotechnical recommendations will be provided

for the overland segment of Newton to Stegosaur pipeline route to mitigate risk of erosion and landsliding during and after construction.

4. Soil Hazard Assessment
    a. Methodology and Scope
        i. Evaluated soil related hazards in accordance with Oregon Administrative Rules (OARs) for exhibit I.
        ii. Included pipelines, well pads, compressor stations, power line and laydown areas.
        iii. Evaluated wind and water erosion hazards based on National Resource Conservation Service (NRCS) mapping and Soil Survey of Columbia County
        iv. Evaluated land use.
        v. Evaluated potential adverse impacts to soil from construction, operation and retirement.
        vi. Provided recommendations for mitigation of potential adverse impacts to soils during construction, operation and retirement.
        vii. Discussed monitoring program for soil erosion.
    b. Site-specific geotechnical work (Application Phase).
        i. Prepared an erosion and sediment control plan (ESCP) for the project.
        ii. No other site-specific geotechnical work is anticipated to address soil-related hazards.
    c. Site-specific geotechnical work (pre-construction)
        i. Will revise ESCP based on proposed grading required for well pads and pipeline installation.
        ii. Implement ESCP elements (BMPs) prior to beginning earthwork.
        iii. CECSL certified person to evaluate soil erosion and BMP effectiveness during construction in accordance with NPDES 1200-C permit and final ESCP.
6. Site-specific Geotechnical Work (completed for EFSC application):
    a. NMCS
        i. 4 borings; two 100-foot deep and two 60-foot deep
        ii. Downhole seismic testing in two 100-foot borings (inform seismic study and for structural design)
        iii. Electrical resistivity testing (for piping design)
        iv. Laboratory testing for soil classification and corrosion resistant design.
        v. Prepared geotechnical report including
            1. Foundation recommendations (shallow, deep and mat foundations)
            2. Grading recommendations
            3. Retaining wall recommendations
            4. Haul road recommendations
            5. Seismic design criteria in accordance with ASCE 7-22. Obtained Seismic parameters from site specific borings (with downhole seismic) and ASCE 7 Hazard Tool
    a. Miller Station
        i. 2 borings to 80 feet

   ii. Downhole seismic testing in one of the borings (inform seismic study and for structural design)
   iii. Performed laboratory testing for soil classification and corrosion design.
   iv. Prepared geotechnical report including
      1. Foundation recommendations (shallow, deep and mat foundations)
      2. Grading recommendations
      3. Seismic design criteria in accordance with ASCE 7-22. Obtained Seismic parameters from site specific borings (with downhole seismic) and ASCE 7 Hazard Tool

b. Power line HDDs
   i. Will complete HDD design for installation of the proposed power line conduits beneath Lundgren Creek and Lyons Creek
   ii. Will include two borings at each site to depth of up to 80 feet below ground surface (4 borings total). Final boring depth to be determined based on future conceptual HDD profile design.
   iii. Will develop HDD design alignment and profile based on results of borings, and workspace layout.
   iv. HDD design reports will include HDD design plan and profile drawing, analyses of hydraulic fracture and inadvertent returns, collapse potential of HDPE conduit and installation forces, construction considerations and recommendations for potentially problematic subsurface conditions; recommendations for contractor drill plan elements to mitigate identified construction considerations.

7. Site Specific Geotechnical Work prior to construction
   a. Well pads
      i. Depending on grading requirements, may complete site specific borings and prepare geotechnical report including slope stability analyses and recommendations for grading.
   b. Injection/Withdrawal Pipelines
      i. Newton to Stegosaur Injection/Withdrawal Pipeline Overland Segment: Prepare geotechnical report for construction of temporary construction corridors including placement of pipeline (trenched into native material), placement of backfill to restore corridor to pre-existing topographic conditions and drainage recommendations.
         1. Anticipate restoration will include typical backfill details for construction on side slopes of 0-50 (2H:1V) percent, and greater than 50 percent.
      ii. Injection/Withdrawal Pipelines along road segments: if cutting into slopes is required, will complete geotechnical report to provide recommendations for temporary and permanent cut slopes, and restoration of slopes if required. No fill slopes will be required. If needed (depending on cut slope height and configuration), borings may be completed to perform slope stability analyses and provide recommendations for replacement of slopes.

Thank you for your time,

Brian C. Ranney, RG, CEG
Associate Engineering Geologist | GeoEngineers, Inc.
Telephone: 503.603.6675
Fax: 503.620.5940
Mobile: 503.730.7728
Email: branney@geoengineers.com
4000 Kruse Way Place
Bldg. 3, Suite 200
Lake Oswego, Oregon 97035
www.geoengineers.com

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Attachment H-2.
Recorded Earthquakes

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Recorded Earthquakes within 50 Miles

Year	Month	Day	Latitude	Longitude	Distance (miles)	Depth (kilometers)	Magnitude
2023	7	2	45.91533	-122.989	14.1	27.42	2.73
2023	6	21	45.6715	-122.713	34.7	20.07	2.54
2022	12	15	45.53267	-122.369	53.1	20.61	2.75
2022	7	24	45.463	-122.933	45.1	19.98	2.84
2022	6	7	45.622	-122.456	51.6	6.82	2.65
2022	2	5	46.245	-122.638	36.2	17.82	2.81
2020	11	19	46.571	-122.738	48.0	22.43	2.87
2020	9	12	46.607	-122.708	51.0	24.84	3.06
2018	9	25	45.677	-122.897	32.5	22.71	2.56
2018	5	25	46.396	-123.291	26.6	16.34	2.95
2017	10	16	45.917	-123.461	12.3	24.31	3.38
2017	7	9	46.164	-123.109	13.1	23.09	2.51
2016	11	2	46.196	-124.016	39.0	37.97	3.55
2016	7	17	45.475	-122.813	47.1	26.65	3.03
2016	5	27	46.574	-122.642	51.1	22.23	2.55
2015	11	19	46.251	-122.445	45.3	17.42	2.66
2015	11	12	45.682	-122.755	37.0	22.196	2.69
2015	6	25	45.489	-123.441	40.4	51.602	3.13
2014	12	29	45.645	-122.758	38.8	19.728	2.51
2014	9	16	46.330	-122.377	50.7	13.626	2.95
2014	4	7	45.398	-122.904	50.1	19.387	3.32
2014	2	14	46.323	-122.388	50.0	13.633	2.51
2014	1	26	45.934	-122.825	24.3	19.963	2.88
2013	11	18	46.324	-122.379	50.4	12.819	2.67
2013	8	31	46.326	-122.386	50.2	13.909	2.63
2013	8	24	46.326	-122.382	50.4	13.159	3.02
2013	8	24	46.324	-122.384	50.2	13.609	3.47
2013	8	23	46.322	-122.385	50.1	13.749	3.7
2013	8	3	46.327	-122.391	50.0	13.199	3.13
2013	5	25	45.814	-122.479	43.8	8.855	2.61
2013	1	31	45.920	-122.409	45.1	6.447	3.66
2012	12	24	45.645	-122.767	38.5	19.49	2.76
2012	11	19	45.646	-122.753	39.0	19.797	3.16
2011	10	19	45.926	-122.410	45.0	10.478	2.6

Year	Month	Day	Latitude	Longitude	Distance (miles)	Depth (kilometers)	Magnitude
2011	9	16	45.794	-122.623	37.7	15.122	2.7
2010	8	4	46.125	-122.502	40.2	16.959	2.8
2010	5	13	46.082	-122.521	38.8	13.711	2.5
2009	9	5	45.514	-122.637	50.1	17.335	2.5
2009	7	8	45.520	-122.632	49.9	16.225	2.5
2008	9	9	46.331	-122.385	50.4	14.742	2.9
2008	9	9	46.332	-122.386	50.4	14.482	2.5
2008	6	4	45.636	-122.724	40.5	18.729	2.5
2008	4	26	46.060	-122.622	33.6	17.274	2.6
2008	3	20	46.530	-122.611	49.7	18.537	2.5
2007	7	3	46.310	-123.234	20.5	9.348	2.6
2007	5	11	46.208	-122.281	52.3	0	3
2006	11	6	45.516	-122.648	49.6	15.518	2.6
2006	8	3	45.804	-122.600	38.4	12.974	2.7
2006	8	3	45.802	-122.607	38.2	14.254	3.8
2006	1	29	45.519	-122.634	49.9	15.208	2.8
2005	10	15	46.577	-122.777	47.3	23.624	2.5
2005	8	25	46.057	-123.972	35.0	35.865	2.5
2005	6	25	45.521	-122.637	49.7	14.717	2.7
2004	10	2	46.285	-122.616	38.5	15.7	2.6
2004	8	17	45.469	-122.847	46.7	26.147	2.6
2004	2	26	45.646	-122.754	38.9	18.771	3
2003	7	26	45.638	-122.735	40.0	16.788	2.8
2003	7	25	45.640	-122.736	39.9	17.228	3
2003	4	24	45.633	-122.739	40.1	17.091	3.9
2003	3	31	45.636	-122.758	39.4	16.768	2.6
1999	7	16	45.649	-122.770	38.2	17.476	3.1
1998	10	31	46.284	-122.611	38.7	21.377	2.57
1998	8	12	45.638	-122.810	37.4	1.426	2.6
1997	9	6	46.128	-122.501	40.2	18.867	2.6
1995	6	13	45.919	-122.983	17.3	23.965	3
1995	5	24	46.408	-123.848	39.6	7.323	2.5
1995	4	2	45.921	-122.975	17.6	25.015	2.7
1994	10	13	46.354	-122.395	50.7	6.841	2.5
1994	9	28	46.084	-123.288	3.8	25.203	2.6

Year	Month	Day	Latitude	Longitude	Distance (miles)	Depth (kilometers)	Magnitude
1993	8	19	45.634	-122.883	35.4	20.501	2.5
1992	3	15	46.217	-123.245	13.7	27.529	3
1991	11	3	45.622	-122.548	47.9	15.022	2.5
1991	10	21	45.631	-122.887	35.5	19.77	3
1991	10	18	45.633	-122.862	36.1	18.08	2.8
1991	10	18	45.633	-122.896	35.1	19.539	3.1
1991	7	27	45.634	-122.865	36.0	19.419	2.8
1991	7	22	45.638	-122.869	35.6	19.139	3.5
1991	3	5	45.787	-122.680	35.5	19.12	3.1
1990	9	29	46.164	-122.865	23.3	20.828	2.6
1990	9	29	46.158	-122.864	23.1	21.748	2.9
1990	6	18	45.987	-123.587	15.7	19.655	3
1990	4	6	45.469	-123.522	42.8	42.133	3.14
1989	9	22	46.384	-123.782	36.0	22.435	2.5
1989	8	28	45.855	-122.580	38.0	12.895	2.6
1989	8	1	45.609	-122.457	52.1	13.518	3.7
1987	10	2	45.734	-122.581	41.7	18.132	2.6
1986	10	12	46.353	-122.651	39.6	67.984	3.48
1986	3	11	45.942	-122.411	44.7	13.485	3.1
1986	1	2	45.900	-122.658	33.1	16.202	2.5
1985	6	7	45.684	-122.783	35.9	19.028	2.8
1984	12	11	45.478	-122.795	47.4	24.927	2.5
1984	6	4	46.214	-123.006	19.4	51.768	3.7
1983	12	30	46.350	-122.412	49.8	6.923	2.6
1983	12	29	46.263	-122.648	36.3	13.45	3
1983	5	11	45.619	-122.833	37.8	-0.859	2.6
1983	3	15	46.504	-122.729	44.4	23.168	2.7
1983	3	13	46.234	-122.629	36.2	14.427	2.9
1983	1	29	45.964	-122.982	16.0	19.351	2.6
1982	11	21	45.909	-122.879	22.3	25.335	2.7
1981	11	8	45.601	-122.469	52.0	5.618	2.5
1981	2	5	46.147	-122.276	51.7	10.398	2.6
1979	12	21	45.862	-122.726	30.9	14.536	2.5
1977	7	14	46.373	-122.481	47.6	13.605	2.7
1977	3	6	46.564	-122.779	46.5	28.047	2.5

Year	Month	Day	Latitude	Longitude	Distance (miles)	Depth (kilometers)	Magnitude
1977	2	11	46.026	-122.713	28.9	16.116	2.5
1976	12	2	46.124	-122.453	42.5	4.166	2.7
1975	6	25	45.960	-122.416	44.3	-1.144	2.5
1974	7	29	45.797	-122.583	39.4	11.795	2.8
1973	2	27	46.250	-122.429	46.1	9.541	2.5
1973	1	24	46.470	-122.525	49.9	28.756	2.7
1972	12	23	46.307	-124.097	45.8	10.668	2.5
1972	11	17	45.723	-122.680	38.0	10.175	3.1
1972	10	13	46.353	-122.400	50.4	15.064	2.6
1972	10	7	46.316	-122.502	44.6	38.828	2.5
1963	12	27	45.700	-123.400	24.9	33	4.5
1962	11	6	45.601	-122.601	46.8	15	5.16
1961	11	7	45.700	-122.400	51.0	33	4.5
Source: USGS 2023c.							

Observed Earthquakes within 50 Miles of the Site with Modified Mercalli Intensity of IV or Greater

Year	Month	Day	Epicentral Latitude	Epicentral Longitude	City Located Closest to the Epicenter	Epicenter Distance from City	MMI at City
1964	1	26	46.1	-122.4	Ariel, WA	19	IV
1964	10	1	45.7	-122.8	Portland, OR	21	V
1963	12	27	45.7	-123.4	Timber, OR	7	VI
1962	11	6	45.63	-122.67	Orchards, OR	9	V
1961	11	7	45.67	-122.87	Scappoose, OR	10	VI
1953	12	16	45.5	-122.7	Portland, OR	2	VI
1941	12	29	45.5	-122.7	Portland, OR	2	VI
1883	9	28	45.5	-122.6	Portland, OR	6	V
1841	12	3	45.6	-122.7	Vancouver, WA	5	V
Source: NOAA 2015.							

Attachment H-3.
Revised North Mist Compressor Station Resiliency Geotechnical Report

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Revised Geotechnical Engineering Report

Mist Resiliency Project
North Mist Compressor Station Resiliency
Columbia County, Oregon

for
NW Natural

November 3, 2023

GeoEngineers
Earth Science + Technology

Revised Geotechnical Engineering Report

Mist Resiliency Project
North Mist Compressor Station Resiliency
Columbia County, Oregon

for
NW Natural

November 3, 2023

GeoEngineers

4000 Kruse Way Place
Building 3, Suite 200
Lake Oswego, Oregon
503.624.9274

Revised Geotechnical Engineering Report

Mist Resiliency Project
North Mist Compressor Station Resiliency
Columbia County, Oregon

File No. 6024-308-00

November 3, 2023

Prepared for:

NW Natural
250 SW Taylor Street
Portland, Oregon 97204

Attention: Brion Larson, Andrew Bauer, RG

Prepared by:

GeoEngineers, Inc.
4000 Kruse Way Place
Building 3, Suite 200
Lake Oswego, Oregon
503.624.9274

Jerad A. Hoffman, PE
Project Engineer

Brian C. Ranney, CEG
Associate Engineering Geologist

Lyle J. Stone, PE
Associate Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
100288PE
OREGON
MAY 9, 2023
LYLE JOSEPH STONE

EXPIRES: 06/30/2024

JAH:BCR:LJS:kjb:cdb

GeoEngineers

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APPENDICES

1.0 INTRODUCTION

This report presents the results of GeoEngineers, Inc.'s (GeoEngineers') revised geotechnical engineering services for the North Mist Compressor Station (NMCS) Resiliency Area which is part of the Mist Resiliency Project in Columbia County, Oregon. The site is shown relative to surrounding physical features in the Vicinity Map, Figure 1. This report was revised from our September 22, 2023 report to address changes in the proposed site grading. We understand NW Natural is considering amending the finish site grade to 1,301 feet in elevation. We also included additional geotechnical recommendations for pile design and earthwork associated with this modification.

Development within the NMCS Resiliency Area will include constructing a new gravel pad for housing three new gas fired reciprocating engine driven compressors, a new 2-story control building and associated piping within a triangular parcel approximately 1½ acres in area adjacent to and north of the existing NMCS. We have not been provided with a layout of the compressors, buildings and piping to be constructed within the NMCS Resiliency Area parcel. However, we understand that the compressor units will weigh approximately 220 kips, and associated gas cooler and regeneration skids will weigh around 48 kips and 40 kips, respectively. Furthermore, we understand that the finish grade of the proposed NMCS Resiliency Area may be amended to match the elevation of the existing NMCS (Approximate El. 1,301 feet), which will require excavation of between 10 and 20 feet of existing site soils. NW Natural is considering placing these soils on property adjacent to the east, north and west of the NMCS Resiliency Area and NMCS.

Burns & McDonnell (the project civil/structural engineer) is considering placing equipment on drilled piers because of undocumented fill existing at the site or removing the undocumented fill and possibly placing equipment on shallow foundations and/or piers. If the fill remains in place, an approximately 20-foot-high retaining wall may be constructed between the lower elevation southern portion of the site and the higher elevation northern portion of the site. The retaining wall will be founded on the lower elevation existing NMCS portion of the site. A Site Plan showing existing site conditions and the locations of our subsurface explorations is provided in Figure 2.

GeoEngineers has completed numerous studies in the project vicinity. In completing this report, we considered data included in GeoEngineers' Geotechnical Engineering Report, North Mist Compressor Station; Columbia County, Oregon (GeoEngineers 2017) in addition to the explorations completed as part of this project.

2.0 SCOPE OF SERVICES

Our services were conducted in general accordance with our proposal dated May 5, 2023 and Change Order No. 1 dated May 26, 2023 authorized by NW Natural with Purchase Order No. 4510006216. The purpose of our geotechnical engineering services was to explore the subsurface soil and groundwater conditions and provide geotechnical engineering recommendations for designing and constructing the new compressor station equipment. Our specific scope of services for this task included the following:

1. Visited the site to mark proposed boring locations.
2. Notified the public "one-call" utility center to mark existing utilities near our proposed boring locations.
3. Subcontracted a private utility locator to locate utilities near our proposed boring locations.

4. Explored subsurface conditions by drilling four borings to depths ranging between 60 feet and 100 feet below ground surface (bgs) on the selected locations. The borings were drilled using mud rotary equipment on a track-mounted drilling rig. A 3-inch diameter PVC casing was installed and grouted in place in the 100-foot borings for subsequent downhole seismic testing. While observing the borings we:
    a. Utilized a vacuum trailer to excavate to depth of 5 feet in the proposed boring locations to clear potential unmarked shallow utilities.
    b. Completed in-situ sampling during standard penetration testing (SPT) using standard 1.5-inch samplers (SPT samplers) and obtained relatively undisturbed Shelby tube samples at representative depths. We obtained continuous rock core of rock encountered in the borings.
    c. Classified the materials encountered in the borings in general accordance with ASTM International (ASTM) Standard Practices Test Method D2488 and the American Society of Civil Engineers (ASCE) rock classifications methods.
    d. Maintained a detailed log of each exploration.
5. Performed laboratory tests on selected samples to determine index, strength or compressibility properties as necessary. The testing included:
    a. Three moisture contents in accordance with ASTM Test Method D 2937 for site soil characterization and interpretation, and to evaluate the fill suitability of the existing soils.
    b. Three Atterberg limits tests in general accordance with ASTM D4318.
    c. Eleven percent fines determinations in general accordance with ASTM D1140.
    d. Three sieve analyses in general accordance with ASTM C136.
6. Subcontracted laboratory testing for corrosion potential including: pH (EPA 9045D), Soluble Sulfates (EPA 300.0), Chloride Ion (EPA 300.0), Electrical Resistivity (AASHTO T 288), Redox Potential (Standard Method [SM] 2580B) and Sulfide (EPA 6010D).
7. Subcontracted a geophysical subconsultant to complete downhole seismic testing on a separate mobilization from drilling. GeoEngineers provided field staff to observe the downhole seismic testing.
8. Evaluated the collected data to determine the site's suitability for the proposed construction, including foundation support requirements.
9. Provided grading recommendations, including stripping depths, unsuitable soil removal, fill type for imported materials, maximum lift thicknesses compaction criteria, cut and fill slope criteria, procedures for use of on-site soils, and wet weather earthwork procedures.
10. Provided excavation recommendations, including temporary and final slope inclinations and trench excavation and backfill compaction.
11. Provided foundation recommendations for proposed structures and appurtenant facilities, including the proposed compressors and associated lightly loaded building as required. Design recommendations addressed the preferred foundation type (mat foundations, deep foundations, ground improvement, etc.), allowable bearing pressure, overturning resistance, minimum footing dimensions and embedment, and settlement behavior.

12. Provided lateral resistance recommendations for foundations, including allowable friction coefficient and passive earth pressures.
13. Provided recommendations for roadways and parking areas, including subgrade preparation and rock sections.
14. Provided recommendations for active, passive and at-rest lateral earth pressures for permanent retaining walls.
15. Evaluated site seismic hazards and recommended the appropriate zone factor and site coefficients for seismic design using conventional equivalent static lateral force methods, as well as recommendations to address seismic hazards identified at the site.
16. Provided a draft geotechnical report for review by the project team.
17. Provided this stamped geotechnical report summarizing our findings and providing geotechnical design recommendations.

This stamped report includes recommendations requested by the project team after the draft report for mechanically stabilized earth (MSE) retaining walls, load resistance factor design (LRFD) shallow foundation bearing capacity figures, LPILE soil parameters, and discussion of the Cascadia Subduction Zone earthquake relative to Code seismic design parameters.

3.0 SITE CONDITIONS

3.1. Surface Conditions

The site is located in the Oregon Coast Range approximately 5.5 miles southwest of Clatskanie, Oregon. The site is located at the north end of the existing North Mist Compressor Station facility and associated compressor equipment positioned on topographic nob. The site slopes gently to the southwest except for an approximately 20-foot-high south facing cut and fill slope that separates the existing NMCS from the proposed NMCS Resiliency Area. This slope is inclined at approximately 3H:1V. Elevations within the northern gently sloping portion of the site range between about 1,317 and 1,324 feet MSL. Elevations of the south facing slope separating the existing NMCS from the NMCS Resiliency Area range from approximately 1,299 feet MSL at the base of the slope to about 1,320 feet MSL at the top of the slope. A gravel road located on the west side of the NMCS provides vehicular access between the NMCS and the NMCS Resiliency Area. This gravel road is situated atop an east facing cut slope that ranges between approximately 5 and 16 feet high. This cut slope is also inclined at approximately 3H:1V.

Slopes on the west, north and east sides of the NMCS have been clear cut and replanted, while slopes on the south side are generally forested with mature conifer trees. A drainage headwall is located off the southeast corner of the NMCS. The associated drainage trends along the north side of the NMCS, turns to the west along the north side of the NMCS Resiliency Area and continues westward to a convergent headwall located about 500 feet northwest of the NMCS Resiliency Area. Native slopes on the west side of the NMCS are planar to convex and inclined at gradients ranging between about 10 and 20 percent to a logging road located west of the NMCS. An approximately 4-foot-high cut slope on the east side of the logging road and a 2- to 6-foot-high fill slope on the west side of the logging road. West of this logging road, slope gradients increase to about 50 to 60 percent and lead down to a north-south trending drainage. Native slopes on the north side of the NMCS are relatively planar and inclined at gradients ranging between about 10 and

30 percent within about 60 feet of the NMCS fence line, at which point slope gradients increase to about 50 percent and continue northward to the drainage that wraps around the west and north sides of the site. Native slopes on the west side of the NMCS are relatively planar and inclined at gradients ranging between approximately 30 and 50 percent.

Site topography based on site-specific topographic survey is shown in Figure 2.

3.2. Subsurface Conditions

3.2.1. Site Geology

We reviewed the geologic mapping for the site by the Oregon Department of Geology and Mineral Industries (DOGAMI, 2015) Newton and van Atta (1976) to develop an understanding of the underlying geology. The DOGAMI mapping shows the site underlain by Quaternary-aged landslide deposits. However, the landslide deposits are broadly mapped throughout the project area and do not define specific landslides. Therefore we reviewed smaller scale mapping by Newton and Van Atta (1976). Newton and Van Atta show the project underlain by Miocene-aged Columbia River Basalt (CRB). Newton and Van Atta also show sedimentary materials of middle Oligocene to lower Miocene aged predominantly Scappoose and Pittsburgh Bluff Formations, mapped within 1 mile south of the site. The CRB overlie and are interbedded with Scappoose and Pittsburg Bluff Formations. The Scappoose and Pittsburg formations are typically described as sandstone, siltstone and claystone.

Although not described in the published mapping, our experience in the Coast Range indicates that both the basalt and sedimentary bedrock are typically deeply weathered to decomposed to depths extending up to tens of feet. The residual soils formed by decomposition range from silt and clay with varying sand contents to silty gravel.

3.2.2. Subsurface Explorations

We completed four borings (NM B-1 through NM B-4) between June 9 and 15, 2023 to depths ranging between 60 and 100 feet bgs. A 3-inch diameter PVC casing was installed and grouted in place in each of the two 100-foot borings (NM B-1 and NM B-3) for subsequent in situ downhole seismic testing. The approximate locations of the borings are shown in Figure 2. Details of the subsurface exploration program and the logs of the explorations are presented in Appendix A, Field Explorations and Laboratory Testing.

Subsurface materials encountered by our borings were generally consistent with the geologic mapping by Newton and Van Atta that we reviewed, with the exception of undocumented fill capping the native materials. We characterized the subsurface material encountered by our borings into three general units: (1) fill, (2) Scappoose Formation, and (3) Columbia River Basalt (CRB). The following subsections provide a general description of subsurface materials encountered by the borings.

3.2.2.1. FILL

Fill soils typically consisted of medium stiff dark brown sandy silt to silt with organics or medium dense silty sand with organics. Organic materials typically consisted of wood chips scattered throughout the soil samples. Fill soils were encountered near the surface in each of our borings and ranged in thickness between approximately 5 feet and 9 feet at the boring locations.

3.2.2.2. SCAPOOSE FORMATION

Decomposed Scappoose Formation sediments were encountered beneath the fill in each boring. The decomposed materials typically consisted of medium dense to dense silty sand with some gravel content; however, the decomposed Scappoose Formation encountered by boring NM B-1 included layers of stiff silt and elastic silt with occasional gravels. The Scappoose Formation generally extended 25 to 50 feet bgs in our borings, except in boring NM B-1 where it was also encountered underlying the CRB and extended to the terminal depth of that boring.

3.2.2.3. CRB

The CRB encountered by the borings was typically decomposed to predominately decomposed to dense to very dense silty gravel or dense gravelly sand. However, moderately weathered to fresh, hard and very closely to closely fractured CRB with occasional decomposed zones was encountered in boring NM B-3 beneath a depth of 43 feet bgs. Except for boring NM B-1, the CRB extended to the terminal depth of the borings. The CRB in NM B-1 was interfingered into the Scappoose Formation between approximately 33 feet to 48 feet bgs.

A more detailed description of subsurface conditions is provided in boring logs in Appendix A.

3.2.3. Groundwater Conditions

Groundwater was not observed in our borings because of the presence of drilling fluid during drilling. Observations of soil moisture content and color suggest that groundwater may not have been encountered within the depth of our borings. Based on a previous subsurface exploration, well log research, and piezometer readings from the existing compressor station (GeoEngineers 2017), we anticipate the regional groundwater table is located hundreds of feet bgs at the stie. However, groundwater may be encountered seasonally perched on fine grained (silt) soils encountered by our borings. We expect groundwater conditions across the site to fluctuate due to rainfall, time of year, as well as other factors.

4.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our subsurface explorations and analyses, it is our opinion that the site can provide suitable support for the proposed compressor station additions and associated improvements, provided the recommendations in this report are incorporated into the project design and are implemented during construction. We offer the following conclusions regarding geotechnical engineering design at the site.

- Based on observations of soil samples from the borings, we anticipate that fill soils across the site may include a variable amount of organics including wood chips, roots, or other deleterious materials. To account for this variability, we recommend that GeoEngineers observe all subgrade excavations to confirm that the assumptions made developing our recommendations are consistent across the site.
- The on-site fill soils generally consist of between 5 feet and 9 feet of silt and sandy silt mixed with woody debris. The presence of woody debris in the fill soils makes them unsuitable for use as structural fill.
- Shallow foundations supported by native soil subgrade prepared as recommended in this report should be adequate to support the proposed equipment at the site. However, this would require excavation and removal of the fill soils to depths of up to 9 feet or more.

- If on-site fill soils are to be left in place, cast-in-place drilled concrete piers up to 42 inches in diameter can be used to transfer loads through soft/loose fill soils and to deeper, more competent soil layers. Cast-in-place drilled concrete piers can also be used to transfer loads to more consistent bearing material to reduce the potential for differential settlement between foundation elements.
- Static groundwater is expected to be a hundred or more feet below the ground surface. Zones of perched groundwater may be encountered during earthwork operations.
- The on-site native soils are generally above the optimum moisture content needed to be compacted as structural fill. Based on our experience working on the North Mist Compressor Station site it will be extremely difficult if not impossible to moisture condition the native soils and reuse it because of the wet weather in the Mist area and the lack of a suitable area for drying the material. If the soil cannot be properly moisture conditioned, we recommend using imported granular material for structural fill.
- If soils excavated to match the existing NMCS site grade will be placed on the east, north and west sides of the NMCS Resiliency Area and NMCS, removal of existing vegetation (trees, brush), stripping of organic topsoil and grubbing to remove tree stumps from slopes within the proposed fill area will be required.
- Fill placed on the east, north and west sides of the NMCS Resiliency Area and NMCS should be placed on slopes with gradients of less than 40 percent. If soils will be placed on existing slopes that are inclined between 20 and 40 percent, keying, benching and drainage will be required prior to placing the fill. Additional fill placement recommendations are included in Section 5.6.

5.0 EARTHWORK RECOMMENDATIONS

5.1. Site Preparation

Initial site preparation and earthwork operations will include grading the site and excavating for utilities and foundations. All existing utilities in the construction area should be identified prior to excavation. Live utility lines identified beneath proposed structures should be relocated. Abandoned utility lines beneath structures should be completely removed or filled with grout in order to reduce potential settlement of new structures. Soft or loose soil encountered in utility line excavations should be removed and replaced with structural fill where loose or soft soil is located within structural areas.

Excavations resulting from removing foundations, utilities, or other subsurface elements should be replaced with structural fill. The bottoms of the excavations should be excavated to expose firm native subgrade, as approved in the field by a qualified geotechnical engineer. All structural fill used during site preparation should meet the criteria in Sections 5.6 and 5.7 of this report.

5.2. Subgrade Preparation and Evaluation

Following stripping, the site should be cut (rough graded) to establish planned subgrade elevations. Rough grading and stripping will expose the on-site materials at final subgrade elevations. Fill subgrades with slopes in excess of 5H:1V (horizontal to vertical) should be properly benched.

The lateral limits of subgrade preparation should extend at least 5 feet beyond the compressor station equipment area and other areas to receive fill. A representative from GeoEngineers should evaluate the need for overexcavation of fill soils with organic materials at the time of construction. Subgrade stabilization may be required for some areas of the site, particularly to protect heavily traveled areas such as haul roads and construction entrances. The typical method of subgrade stabilization consists of granular structural fill placed over a geotextile.

Soft soil and soils with excessive organic material encountered at access road subgrade elevation should be removed to medium stiff/medium dense material or as recommended by the geotechnical engineer during construction. Fill encountered in foundations excavations should be removed such that the foundation can bear on medium stiff/medium dense or better native soils. We recommend that soil exposed at planned subgrade elevations be evaluated by GeoEngineers by either proof-rolling or probing prior to placing fill. The contractor should use construction equipment that can travel on the subgrade areas without causing damage to the subgrade until the subgrade can be stabilized or covered.

Subgrade stabilization within access road areas consisting of Imported Select Granular Fill over geotextile may be required for some areas of the site. Geotextile fabric should have a minimum Mullen burst strength of 500 pounds per square inch (psi) and an apparent opening size (AOS) between a U.S. Standard No. 70 and U.S. Standard No. 100. Mirafi 500x is a fabric that meets these specifications.

For access road planning purposes, particularly to protect heavily traveled areas such as haul roads and construction entrances, we recommend a minimum total fill thickness of 24 inches consisting of a minimum of 18 inches of Imported Select Granular Fill overlain by a minimum 6-inch thickness of Aggregate Wearing Surface material described in Section 6.0. As described below in more detail elsewhere in this report the existing fill is not suitable for structural fill due to the presence of organics.

5.3. Excavation

Based on the materials encountered in our borings, it is our opinion that conventional earthmoving equipment in proper working condition should be capable of making necessary general excavations at anticipated subgrade elevations (20 feet or less bgs). We recommend that excavators with smooth edge buckets (i.e. no teeth) be used wherever practical to limit disturbance to the subgrade. The earthwork contractor should be responsible for reviewing this report, including the boring log, providing their own assessments and providing equipment and methods needed to excavate the site soils while protecting subgrades.

5.4. Dewatering

As discussed in the "Groundwater" section of this report, the regional groundwater was not observed in our explorations and is not likely to be encountered. If perched groundwater is encountered, saturated/wet soils should be dewatered. Sump pumps are expected to adequately address groundwater encountered in shallow excavations. In addition to groundwater seepage, surface water inflow to the excavations during the wet season can be problematic. Provisions for surface water control during earthwork and excavations should be included in the project plans and should be installed prior to commencing earthwork.

5.5. Shoring and Sloping

All excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. The soil OSHA soil type and associated slopes must be determined by the contractor's designated competent person. We recommend the contractor plan on OSHA Type C soils. Excavations deeper than 4 feet should be shored or laid back at an inclination of 1.5H:1V (horizontal to vertical) or flatter if workers are required to enter. Excavations should be laid back or shored at the surface as necessary to prevent soil from falling into excavations.

In our opinion, the contractor is in the best position to observe subsurface conditions continuously throughout the construction process and to respond to the soil and groundwater conditions. Construction site safety is the sole responsibility of the contractor, who also is solely responsible for the means, methods, and sequencing of the construction operations and choices regarding excavations and shoring. Under no circumstances should the information provided by GeoEngineers be interpreted to mean that GeoEngineers is assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

5.6. Non-Structural Fill

For the purposes of this discussion, non-structural fill is considered to be soils that are excavated from the NMCS Resiliency Area and placed on slopes to the west, north and east sides of the existing NMCS and NMCS Resiliency Area that will not support structures in the future. If future structures may be constructed on the fill area, the fill should be placed and compacted in accordance with recommendations for Structural Fill in Sections 5.7 through 5.9.

We recommend segregating the undocumented fill from the underlying native soils during construction. The native soils can be placed as non-structural fill on slopes surrounding the site, provided the existing slopes are prepared and the fill is placed and compacted as recommended in this report. The undocumented fill could be mixed with stripped topsoil and placed as a thin (1-foot or less) topsoil layer above the native soils to promote vegetation growth.

Prior to placing the non-structural fill, vegetation and stumps should be removed from the ground surface. The topsoil should then be stripped to the underlying non-organic native soils and stockpiled for use as topsoil placed over the non-structural fill or hauled off site to an approved disposal location. Any slopes greater than 20 percent that are to receive the non-structural fill should be benched, and a keyway excavated as shown in the attached Typical Fill Slope Detail, Figure 3.

Non-structural fill should be placed in uniform, horizontal lifts and compacted with appropriate equipment. The appropriate lift thickness will vary depending on the material and compaction equipment used. Fill material should be compacted to a minimum of 85 percent of the maximum dry density (MDD) as determined by ASTM Test Method D1557. It is the contractor's responsibility to select appropriate compaction equipment and place the material in lifts that are thin enough to meet these criteria. However, in no case should the loose lift thickness exceed 12 inches.

Final fill slopes constructed of non-structural fill should be no steeper than 3H:1V.

5.7. Structural Fill

Structural fill must be placed directly beneath foundations and beyond the edge of the foundations to a distance equal to the depth of the fill.

All structural fill soils should be free of debris, clay balls, roots, organic matter, frozen soil, man-made contaminants, particles with greatest dimension exceeding 4 inches and other deleterious materials. The suitability of soil for use as structural fill will depend on the gradation and moisture content of the soil. As the amount of fines in the soil matrix increases, the soil becomes increasingly more sensitive to small changes in moisture content and achieving the required degree of compaction becomes more difficult or impossible. Recommendations for suitable fill material are provided in the following sections.

5.7.1. On-Site Soils

5.7.1.1. FILL SOILS

The on-site fill soils generally consist of silt and sandy silt mixed with woody debris and other organics. The presence of organics in the fill soils makes them unsuitable for use as structural fill.

5.7.1.2. NATIVE SOILS

The moisture content of the native on-site soil is likely greater than optimum moisture for adequate compaction. The native silt and silty sand soils are sensitive to small changes in moisture and will be suitable for use as structural fill only if the soil can be properly moisture conditioned and compacted. Based on our experience working on the North Mist Compressor Station site it will be extremely difficult if not impossible to moisture condition the native soils and reuse it because of the wet weather in the Mist area and the lack of a suitable area for drying the material. If the soil cannot be properly moisture conditioned, we recommend using imported granular material for structural fill.

5.7.2. Imported Select Granular Fill

Select imported granular material may be used as structural fill. The imported material should consist of pit or quarry run rock, crushed rock, or crushed gravel and sand that is fairly well graded between coarse and fine sizes (approximately 25 to 65 percent passing the U.S. No. 4 sieve). It should have less than 5 percent passing the U.S. No. 200 sieve.

5.7.3. Aggregate Base

Aggregate base material under foundations should consist of imported clean, durable, crushed angular rock. Such rock should be well graded, have a maximum particle size of ¾ inch and have less than 5 percent passing the U.S. No. 200 sieve. In addition, aggregate base shall have a minimum of 75 percent fractured particles according to American Association of State Highway and Transportation Officials (AASHTO) TP-61 and a sand equivalent of not less than 30 percent based on AASHTO T-176.

5.7.4. Aggregate Wearing Surface

Aggregate Wearing Surface material should consist of material such as Oregon Department of Transportation (ODOT) approved shoulder aggregate meeting the requirements of Section 02640 of the ODOT Standard Specifications for Construction. This material generally consists of 1" – 0 or ¾" – 0 crushed rock, including sand, that is uniformly graded from coarse to fine.

5.7.5. Granular Wall Backfill

Fill placed to provide a drainage zone behind retaining walls should consist of free-draining sand and gravel or crushed rock with a maximum particle size of 1-inch and less than 3 percent passing the U.S. No. 200 sieve. In our opinion, "Granular Wall Backfill" as described in Section 00510.12 of the ODOT Standard Specifications for Construction can be considered. Fill material and placement recommendations provided in Section 6.6.5 of this report should also be considered.

5.7.6. Trench Backfill

The on-site soils are not suitable for use as trench backfill in structural areas. Unless different requirements are specified by the pipe manufacturer, trench backfill for the utility pipe base and pipe zone should consist of well-graded granular material having a maximum particle size of ¾ inch and less than 8 percent passing the U.S. No. 200 sieve. The material should be free of organic matter and other deleterious materials. Above the pipe zone, crushed aggregate should be used as described above. The pipe bedding and backfill above the pipe zone should be placed and compacted as recommended in the "Fill Placement and Compaction" section of this report.

5.8. Structural Fill Placement and Compaction

Structural fill and aggregate base should be compacted at moisture contents that are within 3 percent of the optimum moisture content as determined by ASTM International (ASTM) Standard Practices Test Method D1557 (Modified Proctor). The optimum moisture content varies with gradation and should be evaluated during construction. Material that is not near the optimum moisture content should be moisture conditioned prior to compaction.

Fill and backfill material should be placed in uniform, horizontal lifts and compacted with appropriate equipment. The appropriate lift thickness will vary depending on the material and compaction equipment used. Fill material should be compacted to a minimum of 95 percent of the maximum dry density (MDD) in as determined by ASTM Test Method D1557. It is the contractor's responsibility to select appropriate compaction equipment and place the material in lifts that are thin enough to meet these criteria. However, in no case should the loose lift thickness exceed 12 inches.

A representative from GeoEngineers should evaluate compaction of each lift of fill. Compaction should be evaluated by compaction testing unless other methods are proposed for oversized materials and are approved by GeoEngineers during construction. These other methods typically involve procedural placement and compaction specifications together with verifying requirements such as probing or proof-rolling.

5.8.1. Area Fills and Bases

Fill placed to raise site grades and aggregate base materials under foundations, slabs, and pavements should be placed on a prepared subgrade that consists of firm, inorganic native soils or compacted fill. Fill should be compacted to at least 95 percent of the maximum dry density determined by ASTM Test Method D 1557 (modified Proctor).

Fill placed on slopes should be placed on a keyed and benched surface. Typically, a minimum 4-foot-wide by 2-foot-deep keyway is excavated into competent (medium stiff/medium dense or better) native soils at the base of the fill. The slope of the downslope edge of this excavation should not be greater than 1H:1V. After excavation of the keyway, the slope to receive fill should be benched with the benches being excavated into medium stiff/medium dense or better native soils. The keyway and benching should be observed by the geotechnical engineer or their representative during construction to verify that the keyway and benches were excavated into competent soils.

5.8.2. Trench Backfill

Pipe bedding and fill in the pipe zone should be compacted to 90 percent of the maximum density as determined by ASTM Test Method D 1557, or as recommended by the pipe manufacturer.

In nonstructural areas, trench backfill above the pipe zone should be compacted to at least 85 percent of the maximum dry density as determined by ASTM Test Method D 1557. Suitable native soils that are moisture-conditioned or select granular soils are acceptable in nonstructural areas.

Within structural areas, trench backfill placed above the pipe zone should be compacted to at least 92 percent of the maximum dry density as determined by ASTM Test Method D 1557 at depths greater than 2 feet below the finished subgrade, and to 95 percent within 2 feet of finished subgrade. Trench backfill in structural areas should consist of select granular fill or aggregate base as described in “Structural Fill” section of this report.

5.9. Permanent Slopes

Permanent cut or fill slopes constructed of structural fill should not exceed a gradient of 2H:1V. Fill slopes should be overbuilt by at least 12 inches and trimmed back to the required slope to maintain a firm face.

Slopes should be planted with appropriate vegetation to provide protection against erosion as soon as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

If seepage is encountered at the face of permanent or temporary slopes, it might be necessary to flatten the slopes or install a subdrain to collect the water to prevent long term instability. We should be contacted to evaluate such conditions on a case-by-case basis.

5.10. Compressor Station Surface and Gravel Surfaced Roads

We recommend a minimum total fill thickness of 24 inches along gravel access roads within the existing facility consisting of a minimum of 18 inches of Imported Select Granular Fill overlain by a minimum 6-inch thickness of Aggregate Wearing Surface material for haul roads within the proposed site. This assumes the subgrade consists of medium stiff/medium dense or better soils and that construction will be completed during an extended period of dry weather and with subgrade soils prepared as described elsewhere in this report. Wet weather construction may require an increased thickness of aggregate and isolated over-excavation of soft, wet, or otherwise disturbed material. Subgrade stabilization may be required for some areas of the site, particularly to protect heavily traveled areas such as haul roads and construction entrances. The typical method of subgrade stabilization consists of granular structural fill placed over a geotextile.

We recommend a minimum total fill thickness of 12 inches within light staging areas consisting of a minimum of 6 inches of Imported Select Granular Fill overlain by a minimum 6-inch thickness of Aggregate Wearing Surface material for light staging areas within the proposed site. This assumes the subgrade consists of medium stiff/medium dense or better soils. A representative from GeoEngineers should evaluate the need for overexcavation of fill soils with organic materials and fill compaction at the time of construction in accordance with recommendations outlined in Section 5.2 above.

Prior to placing the gravel surfacing material, the subgrade should be proof-rolled using a fully-loaded dump truck or probed. We recommend that GeoEngineers observe the proof-rolling or conduct probing because of our familiarity with the site and subsurface conditions.

We recommend that a separation geotextile, such as Geotex 104F or approved alternate, be placed between the subgrade and aggregate base layers. The Granular Structural Fill and Aggregate Wearing Surface materials should be placed in lifts and compacted to at least 95 percent of the maximum dry density determined in accordance with ASTM D 1557.

6.0 FOUNDATION SUPPORT RECOMMENDATIONS

6.1. General

We have not been provided a layout of the compressor, buildings and piping to be constructed within the NMCS Resiliency Area. **Due to the variability of site soils (fill and organic material overlying weathered bedrock) and unknown loading conditions, we provide general foundation support recommendations in the sections below. Once structural loads and foundation locations have been finalized we should review and modify our recommendations as needed.**

We understand proposed compressor units weigh approximately 220 kips, building envelop column loads are about 133 kips, and associated gas cooler and regeneration skids weigh around 48 kips and 40 kips, respectively. Because of potential settlement of the upper fill soils and organic material, we understand the project team is considering drilled piers for support of the compressor units and associated structures. Alternatively, the undocumented fill could be removed, and the structures founded on shallow foundations and/or drilled pier foundations. Proposed retaining walls, if utilized, are anticipated to be founded at elevations within native weathered bedrock; therefore, shallow foundation support is expected. Some removal and replacement of loose soils and/or organic material could be required (if encountered) as described below.

6.2. Drilled Pier Support

6.2.1. Drilled Pier Design Considerations

Cast-in-place drilled concrete piers up to 42 inches in diameter can be considered to transfer loads through soft/loose fill soils and to deeper, more competent soil layers. Cast-in-place drilled concrete piers can also be used to transfer loads to more consistent bearing material (i.e. the Columbia River Basalt) to reduce the potential for differential settlement between foundation elements. Drilled piers should be constructed per the most current version of the American Concrete Institute (ACI) Standard Specifications for Construction of Drilled Piers–336.1, or comparable specifications. Concrete for the piers must be placed directly against the side walls of the excavation, not in a casing or sonotube. If casing is required to keep the holes open, it must be removed after the wet concrete is placed but before the concrete has set to allow the concrete to directly contact undisturbed soil. The bottom of the excavation must be kept free of loose or disturbed soil.

Drilled piers should extend through overlying soft/loose and organic-rich soils and be embedded in the underlying native medium stiff/medium dense or denser weathered bedrock deposits. We recommend piers extend a minimum of 5 feet into these denser deposits, which were encountered at depths of about 5 to 9 feet bgs in the borings completed at the site. For uniform performance across the station footprint, we recommend piles be installed into similar end bearing material as determined by the geotechnical engineer in the field.

The structural characteristics of pile materials and structural connections may impose additional limitations on pile capacities and should be evaluated by the structural engineer.

6.2.2. Resistance Estimates

In addition to the minimum depths discussed above, piers should extend deep enough as required for axial and lateral resistance. For uniform performance across the structure footprint, we recommend piles be installed into similar end bearing material.

We anticipate downward axial resistance will be primarily developed from end bearing and skin friction in the medium stiff/medium dense or better weathered rock deposits. Uplift capacity will be developed from side frictional resistance. Estimated resistances for 12-inch through 42-inch-diameter drilled piers are presented in Figures 4 through 9 are based on subsurface materials encountered in boring B-4. Because of the complex stratigraphy and variability of soils encountered, allowable resistances should be used for designing the drilled piers. Allowable resistance estimates were evaluated based on allowable stress design (ASD) and are for combined dead plus long-term live loads. Allowable resistances may be increased by one-third when considering design loads of short duration, such as seismic or wind forces. Allowable axial resistances include a factor of safety (FS) equal to 3 for end bearing, 2 for downward skin friction, and 2 for uplift.

We understand that Burns & McDonnell is considering a range of pier diameters up to 42 inches for building and equipment support. Pile capacities for drilled piers prepared in accordance with Section 6.2.1 up to 42-inch diameter may be estimated utilizing the ultimate skin friction and end bearing parameters in Table 1 below.

TABLE 1. RECOMMENDED ULTIMATE SKIN FRICTION AND END BEARING PARAMETERS

USCS Soil Type	Approx. Layer Depth Range (ft bgs)	Skin Friction[1] (ksf/ft)	End Bearing[2] (ksf/ft)	Max. End Bearing (ksf)
Fill (Silts & Silty Sands w/ Organics)	0 to 9	N/A	N/A	N/A
Scappoose Formation (Silty Sands)	9 up to 52.5	0.075	1.79	35
Columbia River Basalt (Sands and Gravels)	Varies	0.050	1.97	50

Notes:

[1] Skin friction units are kips per square foot per linear foot of pile.

[2] End bearing units are kips per square foot per foot of depth below ground surface.

The lateral load performance of the proposed piers may be evaluated using the computer software program LPILE produced by Ensoft, Inc. LPILE is appropriate for flexible piles; relatively short piles which act as a rigid element could require alternative analysis methods. Recommended LPILE soil parameters are presented in Table 2 below.

TABLE 2. RECOMMENDED LPILE SOIL PARAMETERS

USCS Soil Type	Approx. Layer Depth Range (ft bgs)	Soil Model (p-y Curve Model)	Total Unit Weight (pcf)	Friction Angle (degree)	k (pci)
Fill (Silts & Silty Sands w/ Organics)	0 to 9	Sand (Reese)	115	28	25
Decomposed Scappoose Formation Sediments (Silty Sands)	9 to 52.5	Sand (Reese)	120	32	90
Columbia River Basalt (Decomposed)	52.5+	Sand (Reese)	135	36	225

pci = pounds per cubic inch

In our opinion, no reduction of estimated vertical or lateral resistances is needed if center-to-center pier spacing is at least 5 diameters or greater. The structural characteristics of pier materials and structural connections may impose limitations on capacities and should be evaluated by the structural engineer.

6.2.3. Vertical Settlement Estimates

Vertical settlement under static axial loading is not expected to exceed about 1 inch, while differential settlement between comparably loaded piers is not expected to exceed about ½ inch. Settlement estimates are provided at the top of pier (no stickup) and include the elastic shortening of the pier and the soil reaction for the length of the embedded portion of the pier. Most of this settlement will occur rapidly as loads are applied. Settlement estimates consider axial loading consistent with axial resistances as provided in this report. If the anticipated settlement must be reduced to the maximum extent practical, the piers could be socketed approximately 5 feet into weathered Columbia River Basalt (CRB) at the site.

We estimate vertical settlement under the design seismic event to be less than 1 inch. Because of the unpredictable nature of earthquakes and variability of on-site soil conditions, differential settlement between piers under earthquake conditions could be similar to the total settlement.

6.3. Shallow Foundation Support

6.3.1. Subgrade Preparation

We recommend shallow foundations not bear directly on relatively soft, loose, and/or organic-rich soils to limit the potential for excessive settlement. Depending on thickness of relatively soft, loose and/or organic soils at proposed footing locations, removal and replacement (overexcavation) may be necessary as discussed below.

We recommend that shallow foundations be founded below the local frost line of 12 inches (OSSC 2022) on medium stiff/medium dense or better soils. Soft and/or organic fill soils will require over excavation to firm native soils. The specific depth of overexcavation should be confirmed in the field by GeoEngineers during construction. The width of the overexcavation and placement of Structural Fill should extend beyond the edge of the footing a distance equal to the depth of the overexcavation below the base of the footing. Compaction of aggregate base rock should be performed as described above in the "Fill Placement and Compaction" Section 5.8.

Foundation bearing surface elevations are expected to be above the groundwater table. However, if water infiltrates and pools in the foundation excavations, the water along with any disturbed soil, should be removed before placing the aggregate base rock and reinforcing steel.

We recommend GeoEngineers observe all foundation excavations before placing concrete forms and reinforcing steel or precast foundations to determine that bearing surfaces have been adequately prepared and the soil conditions are consistent with those observed during our explorations.

6.3.2. Spread Footing Design Parameters

We recommend footings have a minimum width of 24 inches and the bottom of the exterior footings be founded at least 12 inches below the lowest adjacent grade. The recommended minimum footing depth is equal to the anticipated frost depth.

We recommend conventional spread footings be proportioned using the bearing resistance values shown in Figure 10, which includes bearing resistances for the Extreme, Strength, and Service Limit States (elastic settlement of 1 and 2 inches). The bearing values presented in Figure 10 are appropriate for foundations bearing on flat ground (i.e., horizontal and level foundation subgrade) with a minimum vertical embedment of 12 inches.

6.3.3. Estimated Spread Footing Settlement

We anticipate that the soft/loose fill soils underlying the site will have at least a moderate settlement potential relative to the proposed structural loads and therefore should be removed from foundation areas. Strip footings and column footings designed and constructed as recommended are expected to experience settlements of approximately 1 inch or less. Differential settlements of up to one half of the total settlement magnitude can be expected between adjacent foundation elements supporting comparable loads.

6.3.4. Lateral Resistance

Lateral loads on shallow and mat foundations can be resisted by passive earth pressures on the sides of footings and by friction on the bearing surface. We recommend that passive earth pressures be calculated using an equivalent fluid unit weight of 200 pounds per cubic foot (pcf) for foundations confined by existing fill consisting of medium dense or denser silty gravel, or 350 pcf if confined by imported granular fill extending two times the depth beyond the edge of the footing.

We recommend using an ultimate friction coefficient 0.45 for foundations bearing on a minimum 2-foot thickness of Structural Fill. The passive earth pressure and friction components may be combined, provided the passive component does not exceed two-thirds of the total.

The passive earth pressure value is based on the assumptions that the adjacent grade is level and groundwater remains below the base of the footing throughout the year. The top 1 foot of soil should be neglected when calculating passive lateral earth pressures, unless the adjacent area is covered with pavement. The lateral resistance values do not include safety factors. We recommend a minimum factor of safety of 1.5 or as determined by the project structural engineer.

6.4. Floor Slabs and Mat Foundations

Concrete floor slabs and mat foundations may be supported on subgrades prepared in accordance with Section 5.2. We recommend that floor slabs be underlain by at least 6 inches of Aggregate Base to aid as a capillary break. Mat foundations and Floor slabs supported on subgrades prepared in Accordance with Section 5.2 can be designed using a coefficient of subgrade reaction modulus (k1) of 150 pounds per cubic inch (pci). This value is for a 1-foot by 1-foot square plate. The coefficient of subgrade reaction for a foundation varies based on its minimum width according to the following equation:

$$k_s = k_{s1}[(B+1)/2B]^2$$

Where k_s is the coefficient of subgrade reaction, k_{s1} is the coefficient of subgrade reaction for a 1-foot by 1-foot plate, and B is the minimum width or lateral dimension of the mat. Based on a total dead load of 350 psf, we estimate that floor slabs constructed as recommended will settle approximately 1 inch or less. Differential settlements less than ½ inch across no less than 25 feet can be expected.

We recommend mat foundations be proportioned using a maximum allowable bearing pressure of 750 psf. This bearing pressure applies to the total of dead and long-term live loads and may be increased by one-third when considering seismic or wind loads. This is a net bearing pressure. The weight of the footing and overlying backfill can be ignored in calculating footing sizes. Vapor barriers are not required under floor slabs. However, vapor barriers are often required by flooring manufacturers to protect flooring and flooring adhesives. A typical vapor barrier consists of plastic sheeting covered with 2 inches of sand. Many flooring manufacturers will warrant their product only if a vapor barrier is installed according to their recommendations. Selection and design of an appropriate vapor barrier, if needed, should be based on discussions among members of the design team. We can provide additional information to assist you with your decision.

6.5. Mechanically Stabilized Earth (MSE) Retaining Walls

We understand current plans are to construct an approximately 20-foot-high retaining wall between the lower elevation southern portion of the site and the higher elevation northern portion of the site. The retaining wall will be founded on the lower elevation existing NMCS portion of the site, expected to be within native weathered bedrock.

We recommend that the guidelines in Section 6-13.GR6 Structural Earth Walls of the Washington State Department of Transportation (WSDOT) General Special Provisions or similar accepted design criteria be used in designing these wall systems. We also recommend that we review all contractor submitted retaining wall plans to confirm that our recommendations were interpreted as intended. We recommend that structural earth walls include a drainpipe at the back of the reinforced section. Other drainage details should be determined by the wall designer.

6.5.1. Bearing Capacity

We provide a set of bearing pressure plots for services states 1 inch and 2 inches of settlement as well as strength and extreme event limit states, as shown in Figure 10. All wall foundations should be embedded a minimum of 2 feet below adjacent grade. The provided bearing pressures assume foundations are supported on medium dense to very dense or stiff to very stiff soils derived from weathered rock.

6.5.2. Structural Earth Wall Design Parameters

We recommend contractor designed or propriety wall systems be designed using the methods in the WSDOT Geotechnical Design Manual and the geotechnical design parameters provided in Table 3. We recommend that gravel borrow consisting of material meeting requirements referenced in Section 5.7 for Granular Wall Backfill be used for the reinforced and backfill sections of the wall. The fill should be compacted to at least 95 percent of the MDD as determined by ASTM D1557 (Modified Proctor).

TABLE 3. RETAINING WALLS DESIGN PARAMETERS (CUT AREAS)

Soil Properties	Wall Backfill		Retained Soil		Foundation Soil	
	Imported Fill[1]	On-site Fill[2]	On-site Fill[2]	Scappoose[2]	Scappoose[2]	CRB[2]
Unit Weight (pcf)	135	115	115	120	120	135
Friction Angle (deg)	38	28	28	32	32	36
Cohesion (psf)	0	0	0	0	0	0

Note:

[1] Assumes Imported Fill consists Granular Wall Backfill (Section 5.7) compacted to at least 95 percent of the MDD determined by ASTM D1557.

[2] Soil units are described in Section 3.2.2 and in detail in the boring logs in Appendix A. Values in Table 3 assume On-site Fill = fill and native soils are weathered bedrock consisting of medium dense or denser granular soils or stiff or stiffer cohesive soils derived from the Scappoose Formation (Scapoose) and/or Columbia River Basalt (CRB).

Walls should be designed to accommodate differential settlement of 1 inch per 100 feet of wall length. Based on the anticipated wall heights on the order of 20 feet, we anticipate that static settlement will be on the order of 1 to 2 inches depending on the height and weight of the wall (see Figure 10). For seismic loading, walls should be designed for a horizontal seismic acceleration coefficient k_h of 0.28g and a vertical seismic acceleration coefficient k_v of 0. This assumes that the wall is free to yield somewhat during a seismic event.

6.6. Cast in Place Retaining Walls

Retaining walls on the order of 10 feet in height can be founded on subgrades prepared in accordance with Section 5.2. Recommended lateral earth pressures are provided in the sections below. Except as noted, the recommended pressures assume the ground surface within about 25 feet laterally of the retaining structures will be level or near level. Drainage systems must be included in the design in accordance with the recommendations presented in the “Wall Drainage” section below. If drainage systems are not feasible, we should be contacted to provide the appropriate undrained lateral soil pressures. In our experience, cantilevered gravity walls are suitable for wall heights up to about 10 feet (based on a level front slope from the base of the wall). Greater wall heights could require reinforcements, bracing, or other alternative wall types. We should be contacted as wall design is advanced to determine if a more detailed global stability analysis is required.

6.6.1. Wall Backfill

As previously discussed, it is our opinion that on-site soils are generally unsuitable for use as structural fill, due to the presence of woody debris and/or moisture sensitive fine-grained soils. We therefore assume wall backfill will consist of Granular Wall Backfill as described in Section 5.7.5.

6.6.2. Recommended Lateral Earth Pressures

We anticipate retaining walls at the site will retain native weathered bedrock deposits and be backfilled with structural fill. For imported wall backfill and drained conditions, we recommend walls and subsurface structures as described above be designed using the following lateral earth pressures.

- Active soil pressure may be estimated using an equivalent fluid density of 40 pcf for the level backfill condition. For walls with backfill sloping upward behind the wall at 2H:1V, an equivalent fluid density of 65 pcf should be used. If the slope is shallower than 2H:1V, the active lateral earth pressures can be linearly interpolated between the two values above using the slope angle in degrees.
- At-rest soil pressure may be estimated using an equivalent fluid density of 60 pcf for the level backfill condition.
- For seismic considerations, a uniform lateral pressure surcharge of 10*H psf (where H is the height of the retaining structure or the depth of a structure below ground surface) should be added to the lateral earth pressure.

The active soil pressure condition assumes the top of the wall is not structurally restrained and is free to rotate and deflect a distance of at least 0.001*H (where H is the wall height). The at-rest condition is applicable where walls are restrained against deflection (not allowed to rotate). The above recommended lateral soil pressures do not include the effects of sloping backfill surfaces or surcharge loads, except as described.

Overcompaction of fill placed directly behind retaining walls or below-grade structures must be avoided to limit lateral pressures placed on the wall. We recommend use of hand-operated compaction equipment and maximum 6-inch loose lift thickness when compacting fill within about 5 feet of retaining walls and below-grade structures.

6.6.3. Surcharge Loads

The above recommended lateral soil pressures do not include surcharge loads. We recommend surcharge effects be considered if surcharge loads are applied closer than one-half of the retaining structure height from the wall face. A typical traffic surcharge of 250 psf should be included if vehicles are allowed to operate within one-half the height of the retaining walls. This traffic surcharge can be estimated by calculating lateral loads with an assumed additional 2 feet of retained fill. Other surcharge loads should be considered on a case-by-case basis.

6.6.4. Wall Foundation Support

Wall foundations may be supported on shallow foundations designed and prepared in accordance with recommendations presented in Section 6.3 above. We estimate settlement of retaining structures will be similar to the values previously presented for structure foundations.

6.6.5. Wall Drainage

Lateral earth pressures provided above are based on drained conditions. A positive drainage system behind walls and below-grade structures must be constructed to collect water and prevent the buildup of hydrostatic pressure against the wall or structure. We recommend a zone of free-draining material behind the retaining structure with perforated pipes to collect seepage water. For this condition, the drainage zone should extend horizontally at least 18 inches from the back of the retaining structure. Other systems and materials may also be considered, where appropriate and as approved by the project engineer.

Site soils encountered in the explorations contain a significant percentage of fines. Fine-grained soils are susceptible to particle migration, potentially clogging the drainage. A filter fabric designed for separation should be placed between the wall backfill and any native site soils or common borrow fill to prevent soil migration.

A perforated, smooth-walled, rigid polyvinyl chloride (PVC) pipe with a minimum diameter of 4 inches should be placed at the bottom of the drainage zone along the entire length of the retaining structure with the pipe invert at or below the elevation of the base of the footing. The drainpipes should collect water and direct it to a tightline leading to an appropriate disposal system. Cleanouts should be incorporated into the design of the drains in order to provide access for regular maintenance. Roof downspouts, perimeter drains, or other types of drainage systems must not be connected to drain systems for retaining walls or below-grade structures. Weep holes and other through-wall drainage systems may also be considered.

6.7. Dynamic Soil Properties

We recommend using a unit weight of 125 pcf, a Poisson's Ratio of 0.3, a dynamic shear modulus of 940 ksf and a damping ratio of 0.05 for granular structural fill placed in accordance with the recommendations given in Section 5.0 of this report.

6.8. Design Spectrum Parameters

We understand that seismic design of the project will be completed in accordance with 2021 International Building Code (IBC). We further understand that the buildings and foundations will be designed assuming Risk Category IV. The design seismic motion recommended in the IBC is based on seismic hazard maps developed by the United States Geological Survey (USGS) and a return period of 2,475 years (about a 2 percent probability of exceedance in 50 years). The USGS maps are developed by compiling known and documented earthquake sources, their distance from the site in question, and other seismological and geological information to predict potential maximum expected ground motions at a site over a particular period of time. The resulting design criteria is a composite or an average of these events, not an expected ground motion from a specific seismic source.

We understand that there is also a desire to make the on-site infrastructure resilient to a Cascadia Subduction Zone (CSZ) event. As described, the IBC design event is not a representation of a specific seismic event and completing a CSZ event specific evaluation would require completing a deterministic site-specific response analysis which is beyond the scope of our work. However, the USGS has published estimates of peak ground accelerations for the region based on a magnitude Mw 9.1 megathrust event on the CSZ. According to the USGS event scenario, predicted peak ground acceleration (PGA) values in the Mist area could be about 0.5g. Based on this information we expect that the PGA at the site associated with a CSZ event could be lower than the IBC event (design PGA = 0.558g).

Based on our understanding of the project and our qualitative comparison of the IBC and CSZ events, it is our opinion that designing the structure for the IBC event will also provide similar resilience against a CSZ event.

Parameters provided in Table 3 are based on the conditions encountered during our subsurface exploration program and the procedure outlined in the 2021 International Building Code (IBC), which references the 2016 Minimum Design Loads for Buildings and Other Structures (American Society of Civil Engineers [ASCE] 7-16). The average of downhole seismic testing results from casing NM B-1 and NM B-3 indicates Site Class C. Parameters listed in Table 4 below are code level parameters and may be used to determine design ground motions for structural design.

TABLE 4. SEISMIC DESIGN PARAMETERS

Parameter	Recommended Value[1]
Site Class	C
Mapped Spectral Response Acceleration at Short Period (S_S)	0.983 g
Mapped Spectral Response Acceleration at 1 Second Period (S_1)	0.508 g
Site Modified Peak Ground Acceleration (PGA_M)	0.558 g
Site Amplification Factor at 0.2 second period (F_a)	1.200
Site Amplification Factor at 1.0 second period (F_v)	1.492
Design Spectral Acceleration at 0.2 second period (S_{DS})	0.786 g
Design Spectral Acceleration at 1.0 second period (S_{D1})	0.505 g

Notes:

[1] Parameters developed based on Latitude 46.047795° and Longitude -123.298327° using the ASCE 7 Hazard Tool.

6.9. Liquefaction

Liquefaction refers to the condition when vibration or ground shaking, usually from earthquake forces, results in the development of excess pore pressures in saturated soils with subsequent loss of strength in the soil deposit affected. In general, soils that are susceptible to liquefaction include very loose to medium dense clean to silty sands and low plasticity silts. For liquefaction to occur, soils must be saturated.

As discussed in the Groundwater Conditions section of this report, groundwater may perch on layers of cohesive soils across portions of the site. We anticipate perched groundwater occurring in loose to medium dense sand to silty sand encountered on site will be relatively thin, transient, and relatively quick draining. Therefore, it is our opinion a design level earthquake presents a low risk of liquefaction of the on-site soils.

6.10. Construction Observation

Satisfactory earthwork performance depends to a large degree on quality of construction. Sufficient monitoring of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during the subsurface exploration. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

We recommend that GeoEngineers be retained to observe construction at the site to confirm that subsurface conditions are consistent with the site explorations, and to confirm that the intent of project plans and specifications relating to earthwork construction are being met

7.0 LIMITATIONS

This report has been prepared for the exclusive use of NW Natural and their authorized agents and/or regulatory agencies for the North Mist Compressor Station Resiliency project associated with Mist Resiliency project in Columbia County, Oregon. This report is not intended for use by others, and the information contained herein is not applicable to other sites. No other party may rely on the product of our services unless we agree in advance and in writing to such reliance.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with generally accepted geotechnical engineering practices in the area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

Please refer to Appendix C, Report Limitations and Guidelines for Use, for additional information pertaining to use of this report.

8.0 REFERENCES

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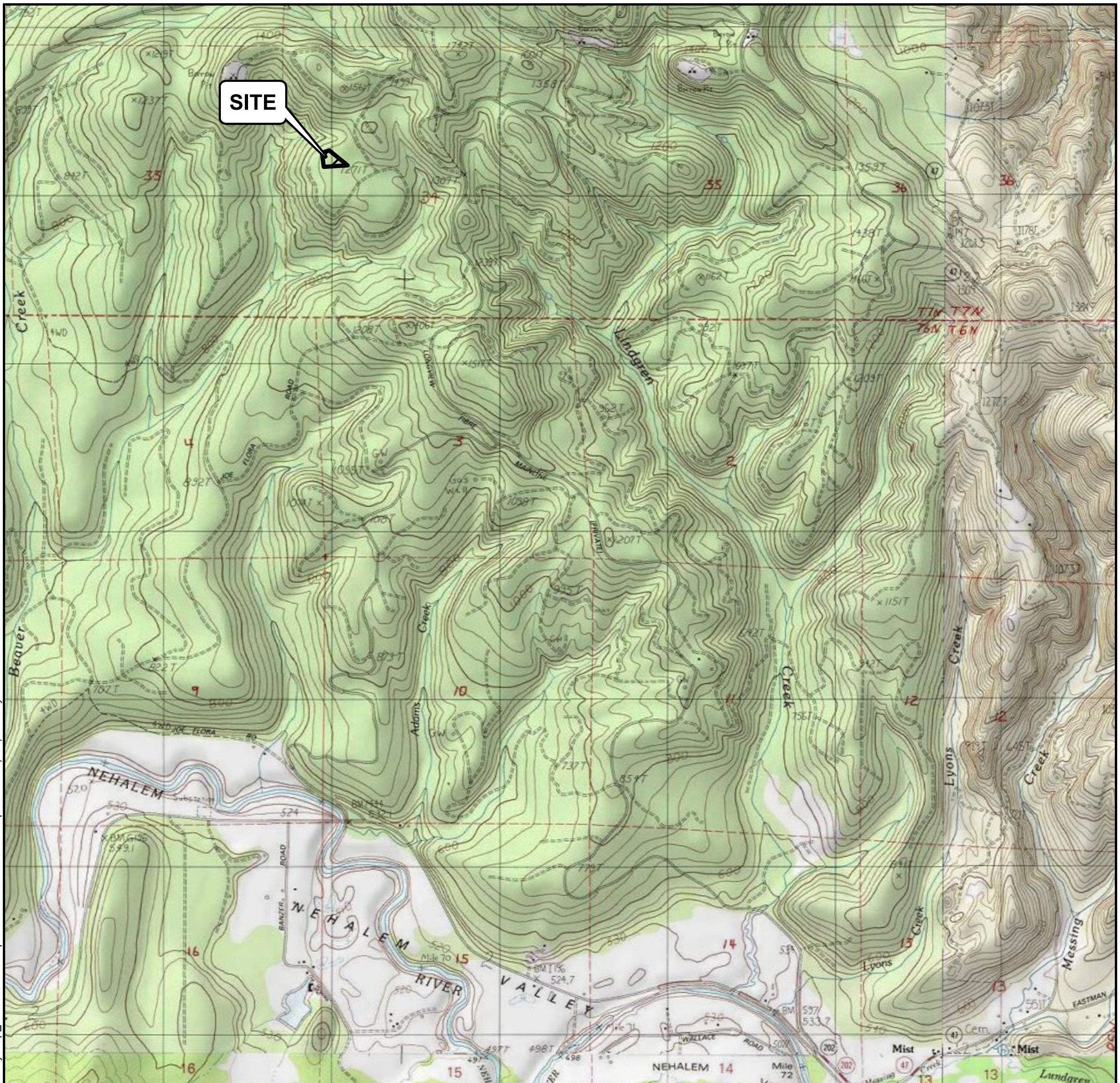
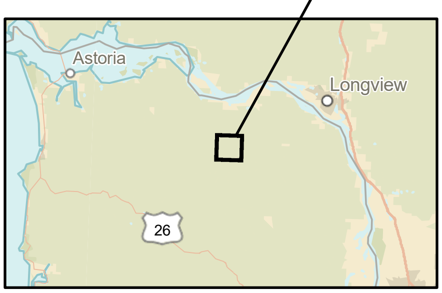
FIGURES

GeoEngineers





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Feet

Vicinity Map

Mist Resiliency Project
North Mist Compressor Station Resiliency
Columbia County, Oregon

GeoEngineers

Figure 1

Source(s):
• ESRI

Coordinate System: NAD 1983 UTM Zone 10N

Disclaimer: This figure was created for a specific purpose and project. Any use of this figure for any other project or purpose shall be at the user's sole risk and without liability to GeoEngineers. The locations of features shown may be approximate. GeoEngineers makes no warranty or representation as to the accuracy, completeness, or suitability of the figure, or data contained therein. The file containing this figure is a copy of a master document, the original of which is retained by GeoEngineers and is the official document of record.

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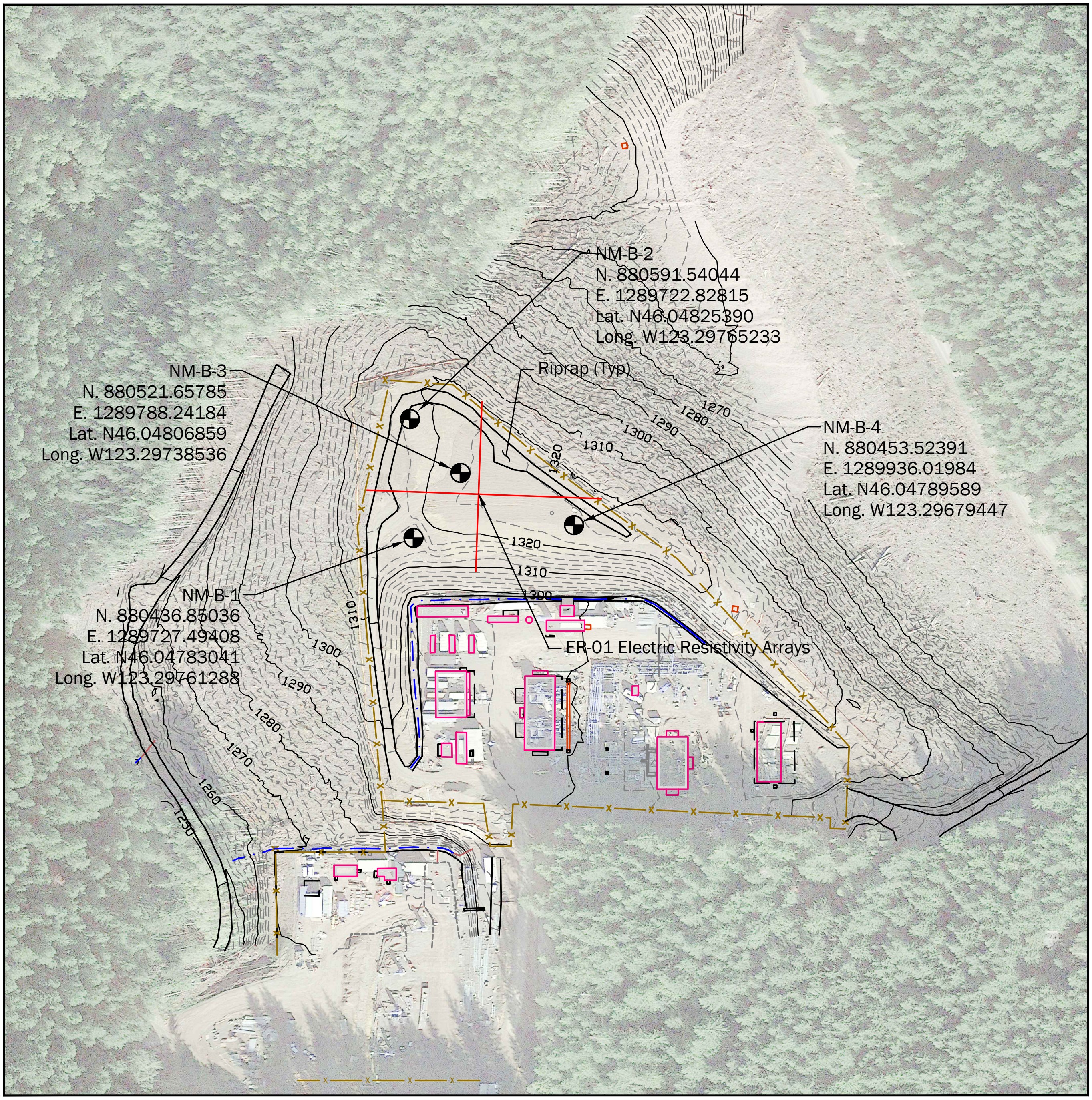


Legend

- Boring Location
- Edge of Gravel, Riprap
- Building Footprint Line
- Fence
- Storm Flowline
- Major Contour - 10' Interval
- Minor Contour - 2' Interval

N

0 200

Feet

Note(s):

1. GeoEngineers, Inc. has not verified the field location of the existing utilities.

Source(s):

- Aerial from Google Earth Pro, dated 10/12/18.
- Survey file provided from Westlake Consultants Inc. by email on 07/27/23.

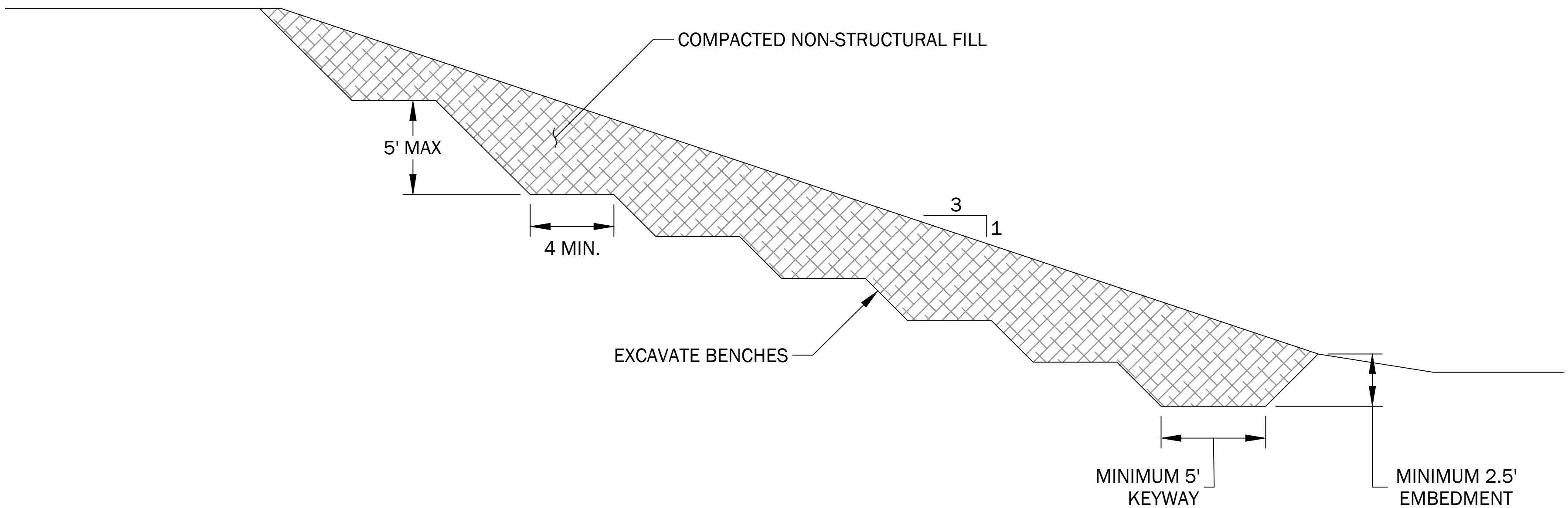
Coordinate System: Oregon State Plane, North Zone, NAD27, US Foot

Boring Location Map

Mist Resiliency Project
North Mist Compressor Station Expansion
Columbia County, Oregon

GEOENGINEERS

Figure 2



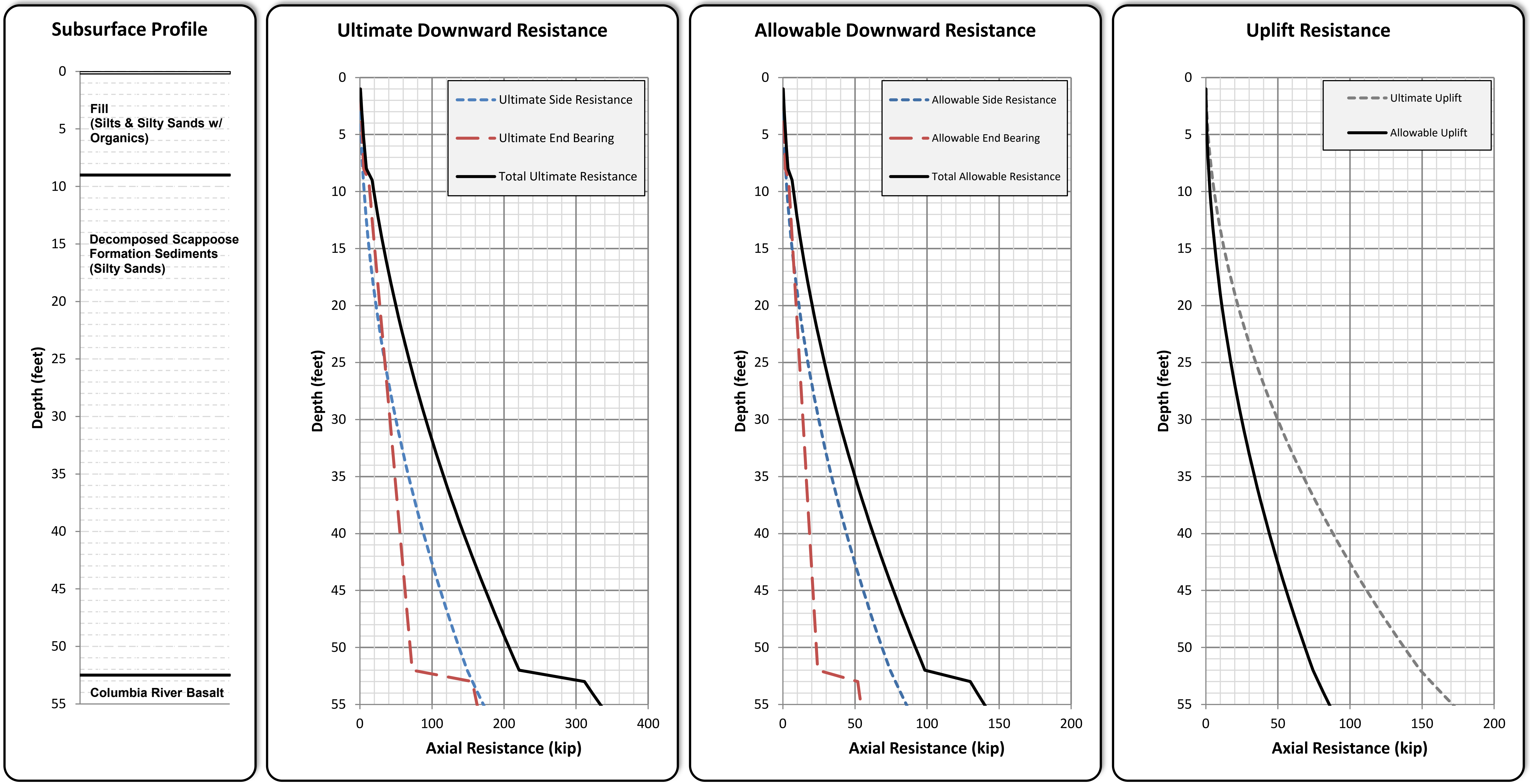
Note:
THIS FIGURE IS A GENERAL REPRESENTATION OF SOIL CONDITIONS FOR INFORMATIONAL PURPOSES ONLY AND SHOULD NOT BE USED FOR BIDDING PURPOSES. REFER TO THE ATTACHED GEOTECHNICAL REPORT AND EXPLORATION LOGS FOR MORE DETAILED INFORMATION.

TYPICAL FILL SLOPE DETAIL

NMCS RESILIENCY AREA
MIST, OR

GeoEngineers

Figure 3



Notes:

1. Resistances consider groundwater depths greater than 100 feet below ground surface.
2. Resistances are based on a single pile and do not consider group effects of closely spaced piles.
3. Allowable resistances include a Factor of Safety (FS) of 2 for Side Friction, 3 for End Bearing and 2 for Uplift.
4. In our opinion, the risk of liquefaction at the site is low. Therefore, seismic liquefaction effects and downdrag loads are not considered.
   i) See report text for additional discussion.

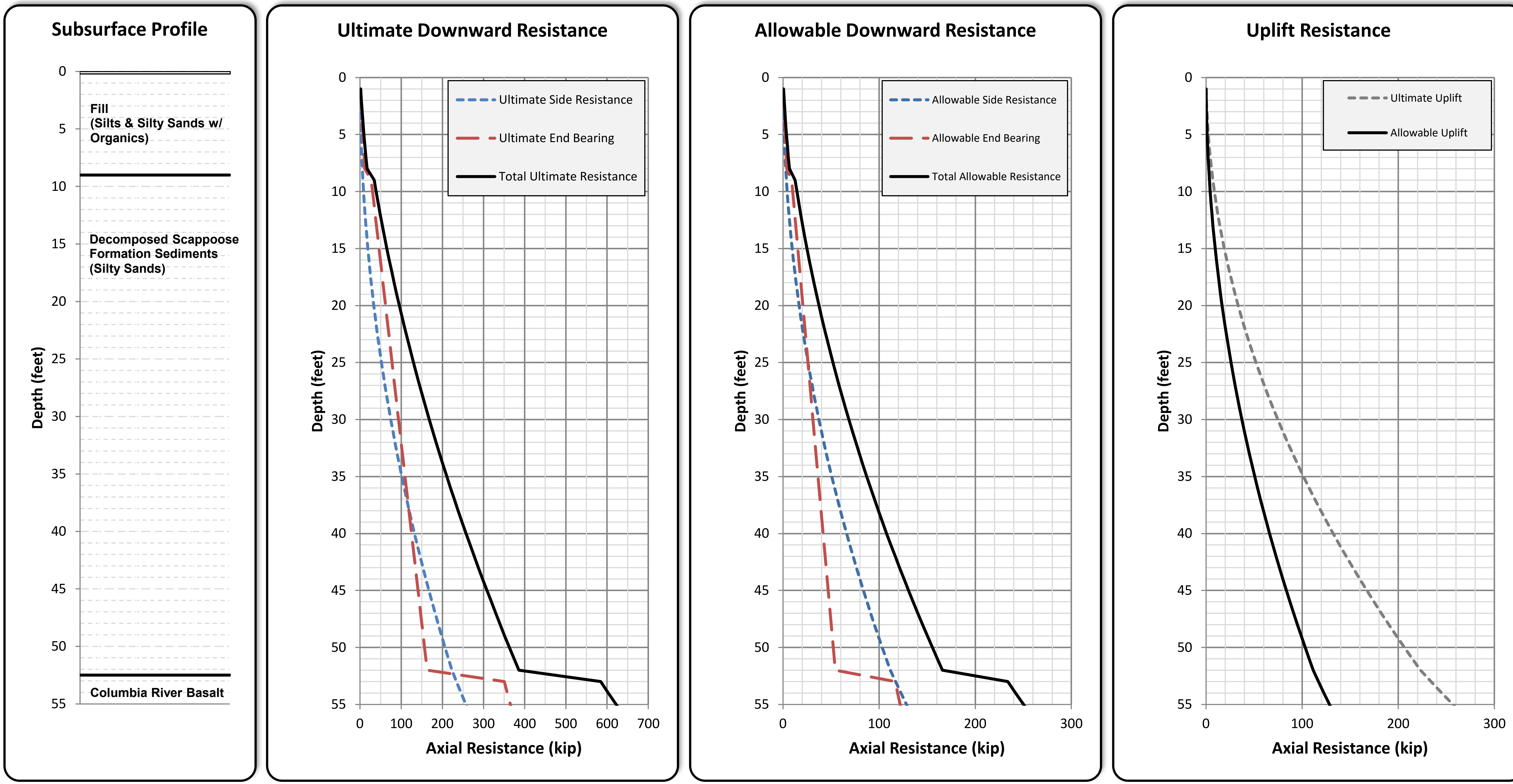
Estimated Axial Resistance
12-inch Diameter Drilled Pier

North Mist Compressor Station Resiliency
Columbia County, Oregon

GeoEngineers

Figure 4

06024-308-00 Date: 9/19/2023



Notes:

1. Resistances consider groundwater depths greater than 100 feet below ground surface.
2. Resistances are based on a single pile and do not consider group effects of closely spaced piles.
3. Allowable resistances include a Factor of Safety (FS) of 2 for Side Friction, 3 for End Bearing and 2 for Uplift.
4. In our opinion, the risk of liquefaction at the site is low. Therefore, seismic liquefaction effects and downdrag loads are not considered.
   i) See report text for additional discussion.

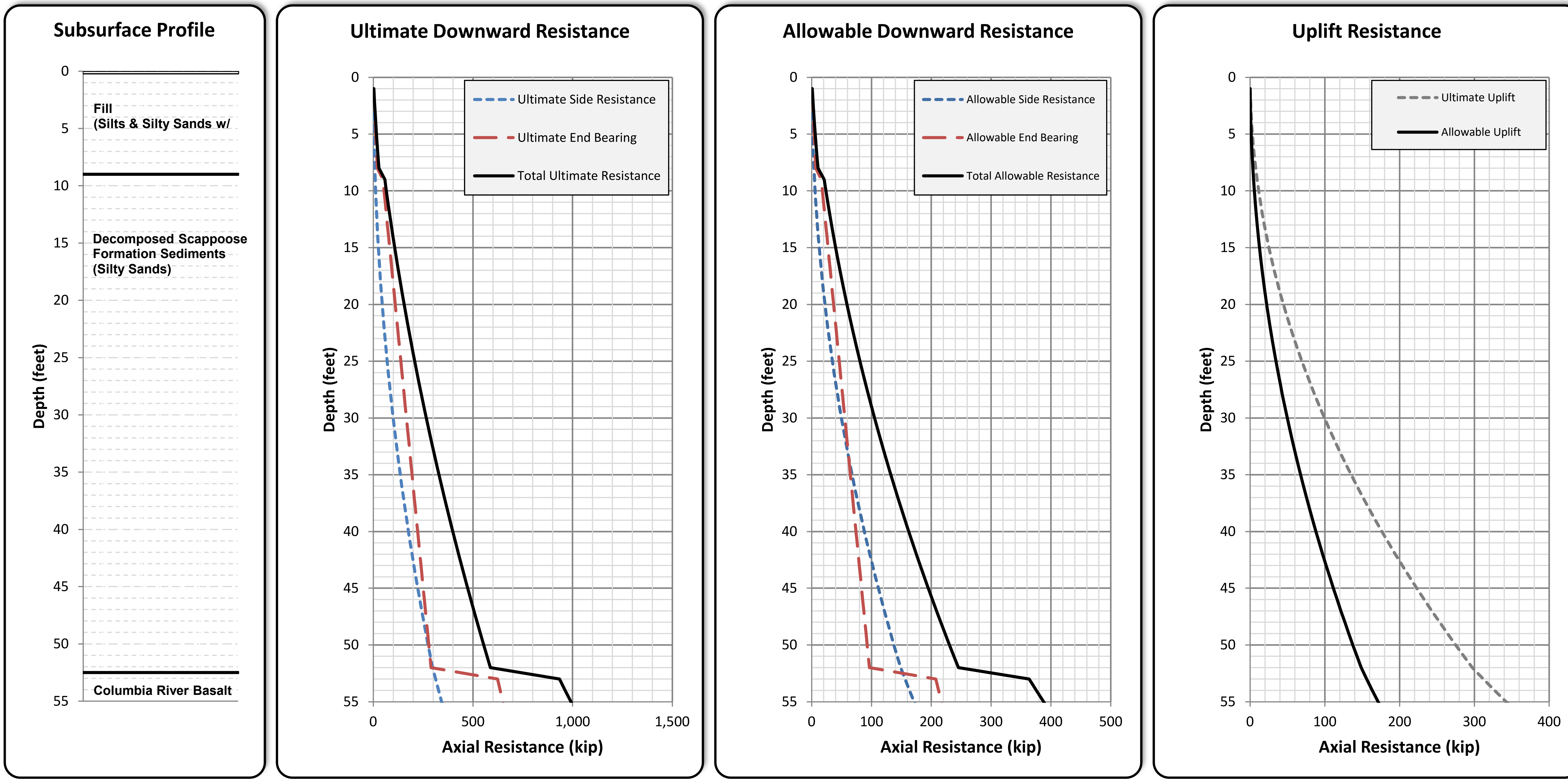
Estimated Axial Resistance
18-inch Diameter Drilled Pier

North Mist Compressor Station Resiliency
Columbia County, Oregon

GEOENGINEERS

Figure 5

06024-308-00 Date: 9/19/2023



Notes:

1. Resistances consider groundwater depths greater than 100 feet below ground surface.
2. Resistances are based on a single pile and do not consider group effects of closely spaced piles.
3. Allowable resistances include a Factor of Safety (FS) of 2 for Side Friction, 3 for End Bearing and 2 for Uplift.
4. In our opinion, the risk of liquefaction at the site is low. Therefore, seismic liquefaction effects and downdrag loads are not considered.
   i) See report text for additional discussion.

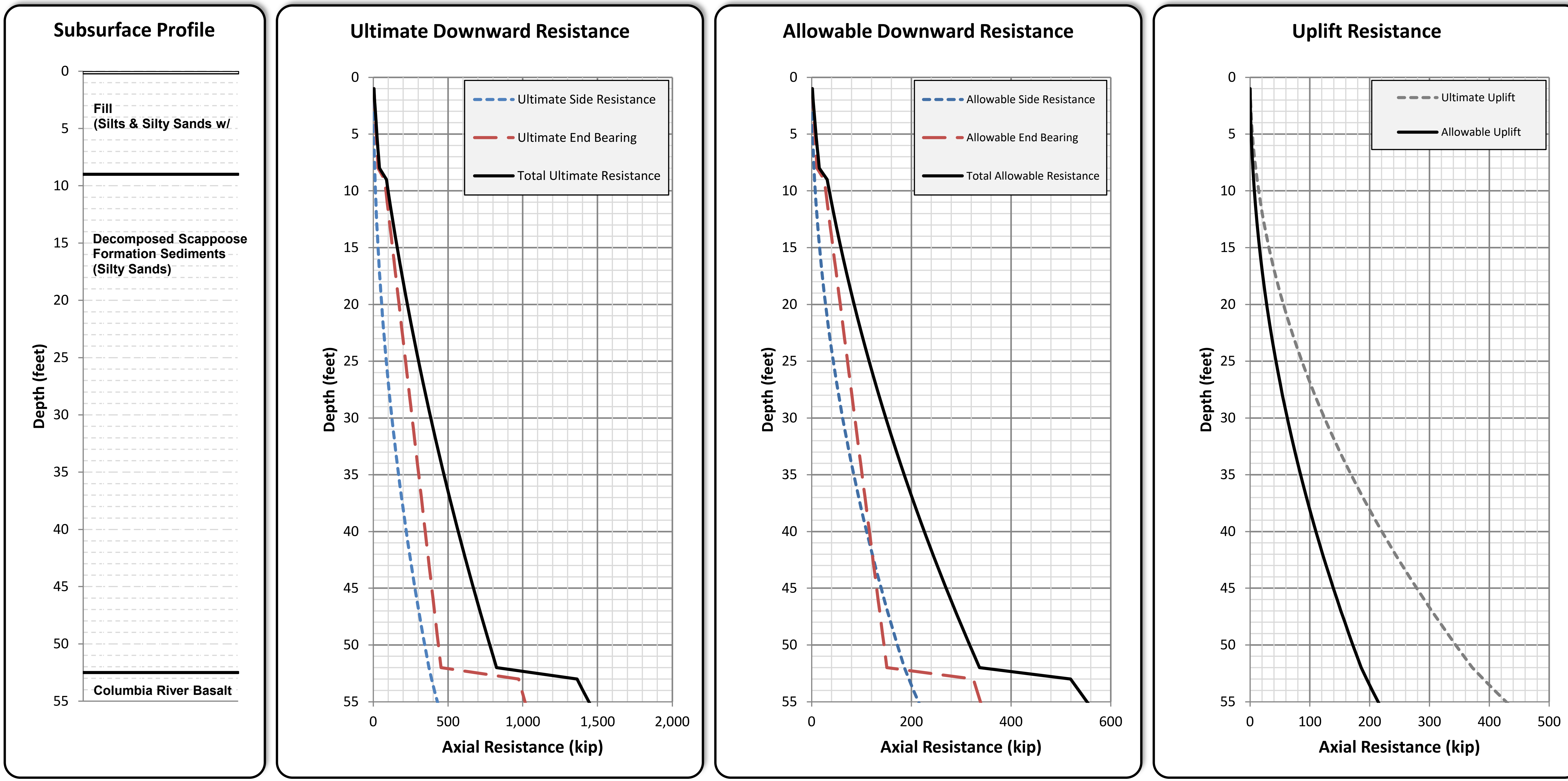
Estimated Axial Resistance
24-inch Diameter Drilled Pier

North Mist Compressor Station Resiliency
Columbia County, Oregon

GeoEngineers

Figure 6

06024-308-00 Date: 11/3/2023



Notes:

1. Resistances consider groundwater depths greater than 100 feet below ground surface.
2. Resistances are based on a single pile and do not consider group effects of closely spaced piles.
3. Allowable resistances include a Factor of Safety (FS) of 2 for Side Friction, 3 for End Bearing and 2 for Uplift.
4. In our opinion, the risk of liquefaction at the site is low. Therefore, seismic liquefaction effects and downdrag loads are not considered.
   i) See report text for additional discussion.

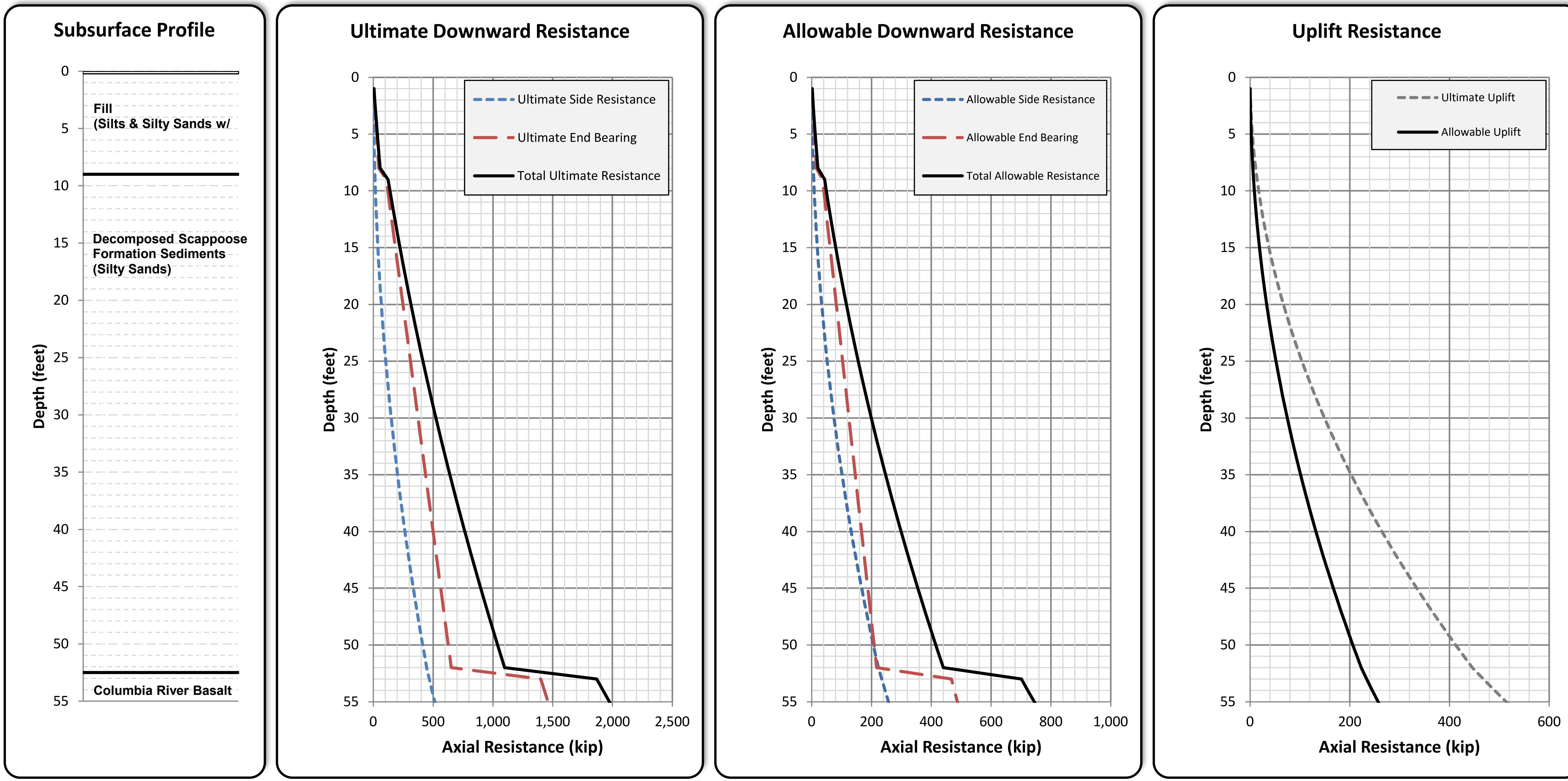
Estimated Axial Resistance
30-inch Diameter Drilled Pier

North Mist Compressor Station Resiliency
Columbia County, Oregon

GeoEngineers

Figure 7

06024-308-00 Date: 11/3/2023



Notes:

1. Resistances consider groundwater depths greater than 100 feet below ground surface.
2. Resistances are based on a single pile and do not consider group effects of closely spaced piles.
3. Allowable resistances include a Factor of Safety (FS) of 2 for Side Friction, 3 for End Bearing and 2 for Uplift.
4. In our opinion, the risk of liquefaction at the site is low. Therefore, seismic liquefaction effects and downdrag loads are not considered.
   i) See report text for additional discussion.

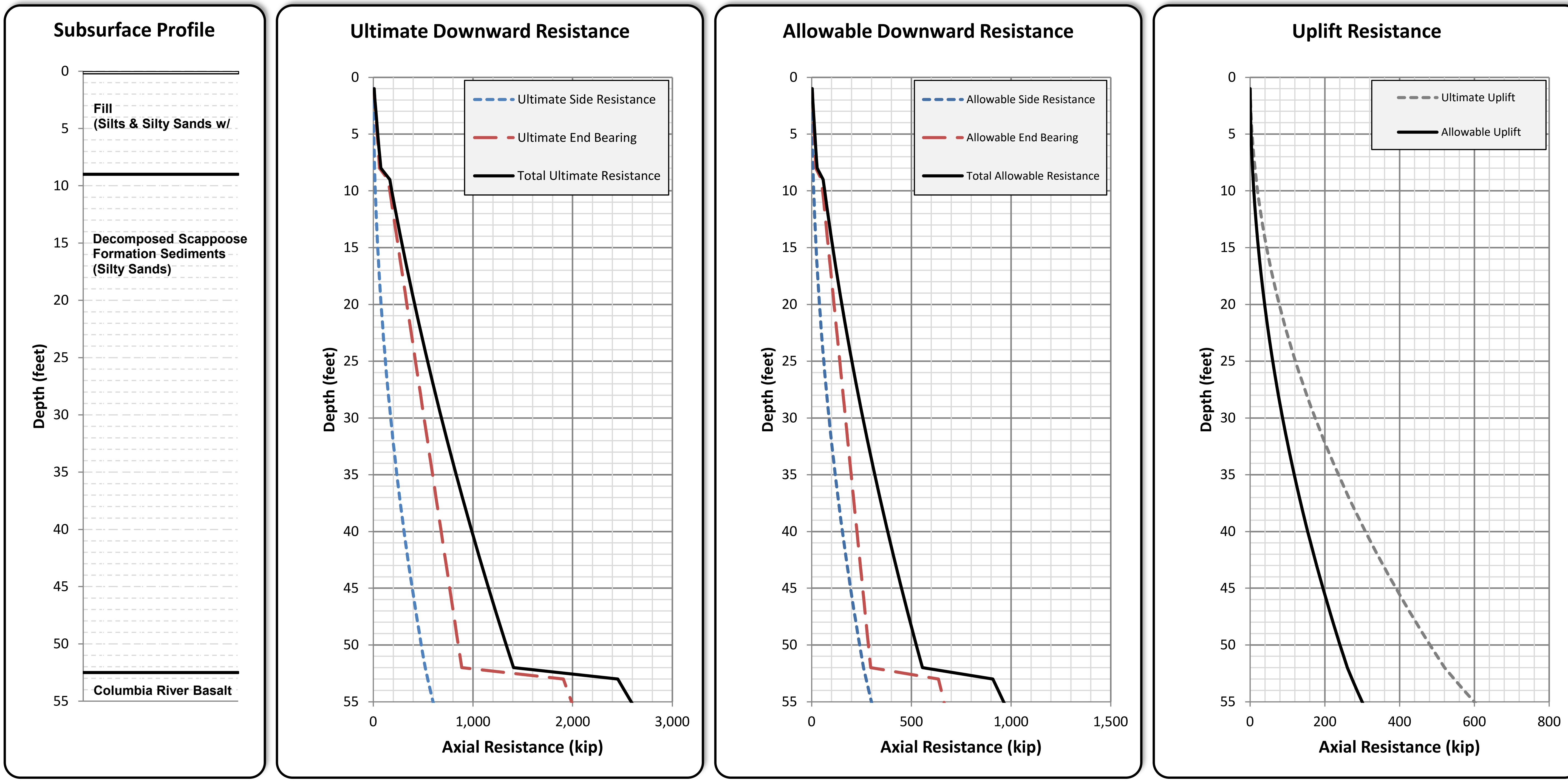
Estimated Axial Resistance
36-inch Diameter Drilled Pier

North Mist Compressor Station Resiliency
Columbia County, Oregon

GeoEngineers

Figure 8

06024-308-00 Date: 11/3/2023



Notes:

1. Resistances consider groundwater depths greater than 100 feet below ground surface.
2. Resistances are based on a single pile and do not consider group effects of closely spaced piles.
3. Allowable resistances include a Factor of Safety (FS) of 2 for Side Friction, 3 for End Bearing and 2 for Uplift.
4. In our opinion, the risk of liquefaction at the site is low. Therefore, seismic liquefaction effects and downdrag loads are not considered.
   i) See report text for additional discussion.

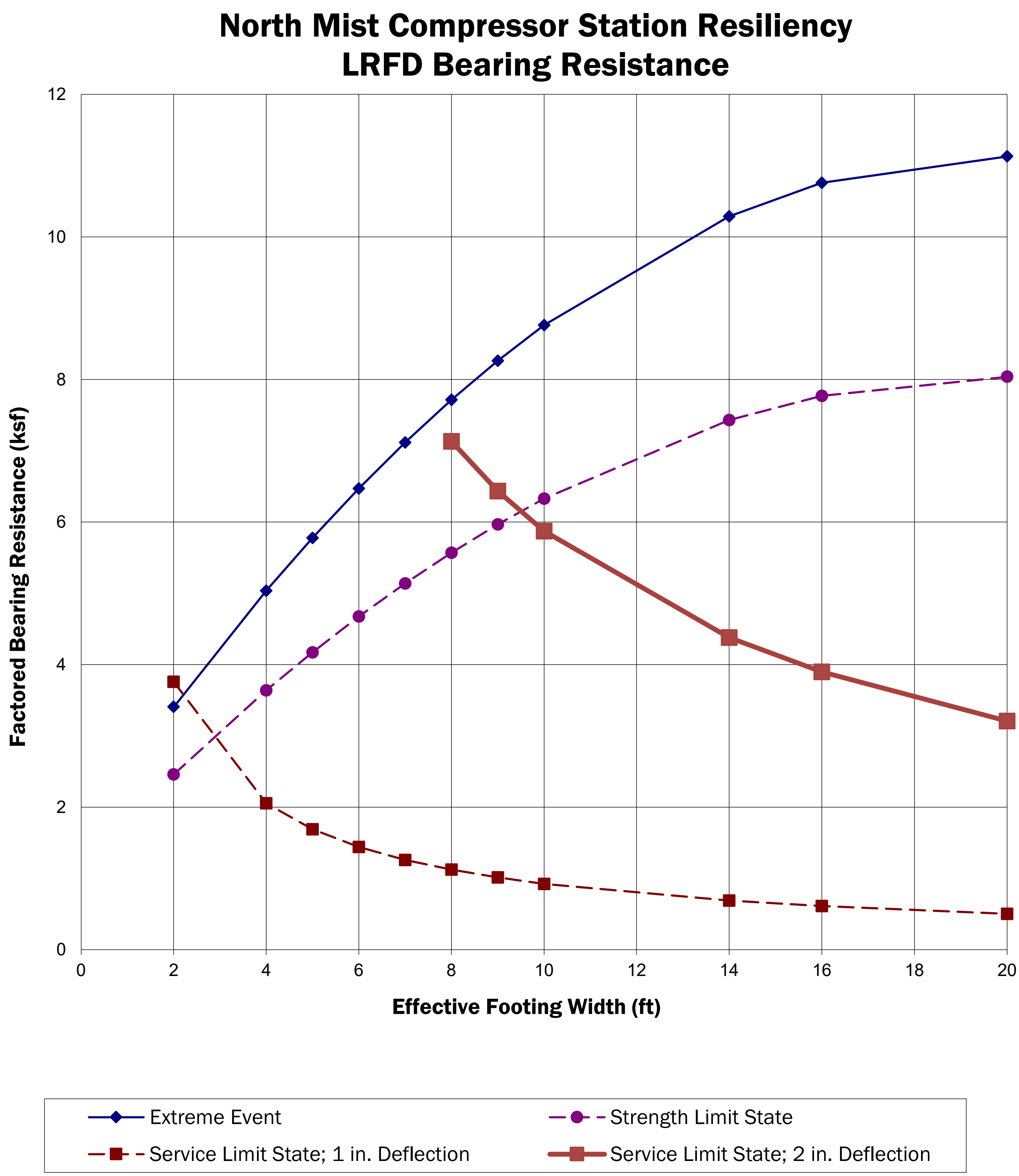
Estimated Axial Resistance
42-inch Diameter Drilled Pier

North Mist Compressor Station Resiliency
Columbia County, Oregon

GeoEngineers

Figure 9

06024-308-00 Date: 11/3/2023



Notes:

1) Strength limit state incorporates a resistance factor of 0.65.

2) Extreme Event incorporates resistance factor of 0.9.

LRFD Shallow Foundation Design
Bearing Capacity

North Mist Compressor Station Resiliency
Columbia County, Oregon

GEOENGINEERS

Figure 10

Project Number: 06024-308-00 Date Exported: 09/18/2023

APPENDICES

APPENDIX A
Field Explorations and Laboratory Testing

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

We completed four borings (NM B-1 through NM B-4) between June 9 and 15, 2023 at the approximate locations shown in Figure 2. The borings were completed to depths ranging between 60 feet bgs and 100 feet bgs with a track mounted drill rig using mud rotary and rock coring techniques. Western States Soil Conservation, Inc. of Hubbard, Oregon drilled the borings. A representative from our Portland, Oregon office observed field activities, classified soil and rock encountered, obtained representative soil samples and rock cores, observed groundwater conditions and prepared a log of each exploration. The borings were backfilled with ⅜-inch bentonite chips at the conclusion of each exploration.

Vacuum excavation was utilized to depth of 5 feet bgs in order to ensure there were no utilities present at the boring locations. Soil samples were obtained from the borings at representative intervals using split spoon samplers and Shelby tubes. Soils encountered in the borings were classified in the field by a GeoEngineers representative in general accordance with ASTM International (ASTM) Standard Practices Test Method D2488, the Standard Practice for the Classification of Soils (Visual-Manual Procedure) which is described in Figure A-1. Rock samples were obtained on a continuous basis and classified in general accordance with the American Society of Civil Engineers Manual on Engineering Practice No. 56 (ASCE 1976), which is briefly described in Figure A-2. The boring logs are presented in Figures A-3 through A-6. Soil/rock classifications and sampling intervals are shown in the boring logs. Inclined lines at the material contacts shown on the log indicate uncertainty as to the exact contact elevation, rather than the inclination of the contact itself.

Standard penetration tests (SPTs) were performed during soil sampling in general accordance with ASTM Test Method D1586. The sampler was driven with a 140-pound hammer falling 30 inches. The number of blows required to drive the sampler 1 foot, or as otherwise indicated, into the soils is shown adjacent to the sample symbols on the boring log. Disturbed samples were obtained from the split spoon sampler for subsequent classification and index testing. Bedrock core samples were obtained using a 2.5-inch I.D. HQ core barrel.

The relative density of the SPT samples recovered at each interval was evaluated based on correlations with lab and field observations in general accordance with the values outlined in Table A-1 below.

TABLE A-1. CORRELATION BETWEEN BLOW COUNTS AND RELATIVE DENSITY *

Cohesive Soils (Clay/Silt)						
Parameter	Very Soft	Soft	Medium Stiff	Stiff	Very Stiff	Hard
Blows, N	< 2	2 – 4	4 - 8	8 – 16	16 - 32	>32

Cohesionless Soils (Gravel/Sand/Silty Sand) **					
Parameter	Very Loose	Loose	Medium Dense	Dense	Very Dense
Blows, N	0 – 4	4 – 10	10 – 30	30 - 50	> 50

Notes:

* After Terzaghi, K and Peck, R.B., "Soil Mechanics in Engineering Practice," John Wiley & Sons, Inc., 1962.

** Classification applies to soils containing additional constituents; that is, organic clay, silty or clayey sand, etc.

The Rock Quality Designation (RQD) of the rock core samples recovered from each core run was evaluated and is presented in the exploration logs in Figure A-5. Rock quality descriptions are correlated to RQD (%) as outlined in Table A-2, below. Photographs of the rock cores are presented in Figures A-10 through A-12.

TABLE A-2. ROCK QUALITY DESIGNATION

RQD (%)	Rock Quality Description
0–25	Very Poor
26–50	Poor
51–75	Fair
76–90	Good
91–100	Excellent

Geotechnical Laboratory Testing

General

All soil samples obtained from the borings were visually classified in the field and/or in our laboratory using a system based on the USCS and ASTM classification methods. ASTM Test Method D 2488 was used to visually classify the soil samples, while ASTM D 2487 was used to classify the soils based on laboratory tests results. These classification procedures are incorporated in the boring logs shown in Figures A-3 through A-6.

Moisture Content Determinations

Moisture contents were determined in general accordance with ASTM D 2216 for selected samples obtained from the borings. The results of these tests are presented in the boring logs at the sample depths.

Percent Passing U.S. No. 200 Sieve

Selected samples were "washed" through the U.S. No. 200 mesh sieve to determine the relative percentage of coarse- and fine-grained particles in the soil. The percent passing value represents the percentage by weight of the sample finer than the U.S. No. 200 sieve. These tests were conducted in general accordance with ASTM D 1140, and the results are shown in the boring logs at the sample depths.

Atterberg Limits Determinations

Selected samples were tested to determine their Atterberg limits in general accordance with ASTM D 4318. The results of these tests are shown in the boring logs at the sample depths and presented in this Appendix in Figure A-7.

Sieve Analysis

Sieve analyses were performed on selected soil samples in general accordance with ASTM C 136. The results of the sieve analyses were plotted and classified in general accordance with the USCS and are presented in Figures A-8 and A-9. The percentage passing the U.S. No. 200 sieve is shown on the boring logs.

Corrosion Potential Testing

General

Soil samples obtained from the borings between depths of 5 feet bgs and 20 feet bgs were selected and sent to Gerhart Cole Laboratory in Salt Lake City, Utah to conduct corrosion potential testing. The corrosion potential testing included the following tests. Results of the testing are included at the end of this appendix.

Electrical Resistivity Testing

Electrical resistivity tests were performed on the selected soil samples in accordance with AASHTO T 288.

Chloride Ion

Selected soil samples were tested for Chloride Ion in accordance with EPA 300.0.

Soluble Sulfates

Selected soil samples were tested for soluble sulfate in accordance with EPA 300.0.

Sulfite Ion Content

Selected soil samples were tested for total sulfur content in accordance with EPA 6010D. Sulfite ion content can subsequently be determined with the following equation:

TOTAL SULFUR – SOLUBLE SULFATE = SULFITE ION CONTENT

pH Testing

Selected soil samples were tested for pH in general accordance with EPA 9045D.

Carbonate and Bicarbonate

Selected soil samples were tested for carbonates and bicarbonates in general accordance with Standard Method (SM) 2320B.

Redox Potential

Selected soil samples were tested for oxidation/reduction (redox) potential in general accordance with SM 2580B.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS — MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS — MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS — MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS — MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

ADDITIONAL MATERIAL SYMBOLS

GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
	AC	Asphalt Concrete
	CC	Cement Concrete
	CR	Crushed Rock/ Quarry Spalls
	SOD	Sod/Forest Duff
	TS	Topsoil

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Laboratory / Field Tests

%F	Percent fines
%G	Percent gravel
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DD	Dry density
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
Mohs	Mohs hardness scale
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PL	Point load test
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
UU	Unconsolidated undrained triaxial compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen

Sampler Symbol Descriptions

2.4-inch I.D. split barrel / Dames & Moore (D&M)

Standard Penetration Test (SPT)

Shelby tube

Piston

Direct-Push

Bulk or grab

Continuous Coring

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

"P" indicates sampler pushed using the weight of the drill rig.

"WOH" indicates sampler pushed using the weight of the hammer.

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

Key to Exploration Logs

GeoEngineers

Figure A-1

Explanation of Bedrock Terms

Scale of Relative Rock Weathering[1]

Designation	Field Identification
Fresh	Crystals are bright. Discontinuities may show some minor surface staining. No discoloration in rock fabric.
Slightly Weathered	Rock mass is generally fresh. Discontinuities are stained and may contain clay. Some discoloration in rock fabric. Decomposition extends up to 1 inch into rock.
Moderately Weathered	Rock mass is decomposed 50% or less. Significant portions of rock show discoloration and weathering effects. Crystals are dull and show visible chemical alteration. Discontinuities are stained and may contain secondary mineral deposits.
Predominantly Decomposed	Rock mass is more than 50% decomposed. Rock can be excavated with geologist's pick. All discontinuities exhibit secondary mineralization. Complete discoloration of rock fabric. Surface of core is friable and usually pitted due to washing out of highly altered minerals by drilling water.
Decomposed	Rock mass is completely decomposed. Original rock "fabric" may be evident. May be reduced to soil with hand pressure.

Scale of Relative Rock Hardness[1]

Term	Hardness Designation	Field Identification	Approximate Unconfined Compressive Strength
Extremely Soft	R0	Can be indented with difficulty by thumbnail. May be moldable or friable with finger pressure.	< 100 psi
Very Soft	R1	Crumbles under firm blows with point of a geology pick. Can be peeled by a pocket knife. Scratched with fingernail.	100-1000 psi
Soft	R2	Can be peeled by a pocket knife with difficulty. Cannot be scratched with fingernail. Shallow indentation made by firm blow of geology pick.	1000-4000 psi
Medium Hard	R3	Can be scratched by knife or pick. Specimen can be fractured with a single firm blow of hammer/geology pick.	4000-8000 psi
Hard	R4	Can be scratched with knife or pick only with difficulty. Several hard hammer blows required to fracture specimen.	8000-16000 psi
Very Hard	R5	Cannot be scratched by knife or sharp pick. Specimen requires many blows of hammer to fracture or chip. Hammer rebounds after impact.	> 16000 psi

Discontinuity Spacing[1]

Description for Bedding, Foliation, or Flow Banding	Spacing	Description of Joints, Faults, or Other Fractures
Very Thick	>10 ft	Very Widely Spaced
Thick	3 ft – 10 ft	Widely Spaced
Medium	1 ft – 3 ft	Moderately Spaced
Thin	2 in – 1 ft	Closely Spaced
Very Thin	<2 in	Very Closely Spaced

Rock Quality Designation (RQD)[1, 2]

RQD (Percent)	Description of Rock Quality
0 – 25	Very Poor
25 – 50	Poor
50 – 75	Fair
75 – 90	Good
90 – 100	Excellent

Notes:

1. Based on ASCE Manual on Engineering Practice No. 56,1976.
2. RQD is a modified core recovery measurement which expresses the number of hard and sound rock pieces of 4" or more in size as a percentage of the total length of core run.

Explanation of Bedrock Terms

North Mist Compressor Station Resiliency
Columbia County, Oregon

GEOENGINEERS

Figure A-2

Drilled	Start 6/9/2023	End 6/12/2023	Total Depth (ft)	101.5	Logged By WCS / Checked By AB	Driller	Western States Soil Conservation, Inc.	Drilling Method	Mud Rotary

Surface Elevation (ft) / Vertical Datum	1315 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	CME 55 Track Rig
Latitude / Longitude	46.047664 / -123.298845	System Datum	NAD83 (feet)	Groundwater not observed at time of exploration	

Notes: 3-inch-diameter PVC casing installed to full depth of boring upon completion.

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
1315	0				ML	Brown to dark brown sandy silt with organic matter (wood chips) (medium stiff, moist) (fill)			Vacuum excavation to approximately 5 feet
1310	5	6	9	1	ML	Brown to dark brown silt with sand (stiff, moist) (decomposed Scappoose formation)			
		12	10	2					
1305	10	12	6	3		Becomes medium stiff			
		13	8	4 %F; AL			46	78	AL (LL = 48, PI = 17)
1300	15	1	7	5					
					MH	Brown sandy elastic silt (stiff, moist)			
1295	20	18	10	6					
1290	25	15	18	7 %F		Becomes very stiff	51	58	
1285	30	15	18	8 SA		Becomes gravelly with sand	60	63	%G = 19
					GM	Brown silty gravel (very dense, moist) (decomposed to predominantly decomposed Columbia River basalt)			
1280	35								

Note: See Figure A-1 for explanation of symbols; Figure A-2 for explanation of bedrock terms.
Coordinates Data Source: Horizontal approximated based on GPS. Vertical approximated based on Topographic Survey.

Log of Boring NM B-1

GEOENGINEERS

Project: North Mist Compressor Station Expansion
Project Location: Columbia County, Oregon
Project Number: 6024-308-00

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		18	51		9						
1275	40		18	35		10			Becomes dense with decreased gravel content			
1270	45		18	79		11			Becomes very dense			
								MH	Brown elastic silt with gravel (stiff, moist) (decomposed Scappoose formation)			
1265	50		17	9		12 / SA				51	72	%G = 25
								SM	Tannish brown sand with silt (medium dense)			
1260	55		18	19		13 / %F				41	38	
1255	60		18	31		14			Becomes dense			
1250	65		18	20		15 / %F			Becomes medium dense	35	30	
1245	70		15	34		16			Becomes dense			
1240	75		14	22		17			Becomes medium dense			

Elevation (feet)	Depth (feet)	FIELD DATA: Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
1235	80		14	7		18			Becomes loose			
			24			19						
1230	85		18	21		20 / %F			Becomes tan and medium dense	39	24	
1225	90		17	35		21			Becomes brown and dense			
1220	95		18	40		22			Becomes with orange and red streaks			
1215	100		18	57		23 / MC			Becomes very dense	36		

Log of Boring NM B-1 (continued)

Drilled	Start	End	Total Depth (ft)	Logged By	Checked By	Driller	Drilling Method
	6/14/2023	6/14/2023	61.5	WCS	AB	Western States Soil Conservation, Inc.	Mud Rotary

Surface Elevation (ft) Vertical Datum	1319 NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	CME 55 Track Rig
Latitude Longitude	46.04808731 -123.2988849	System Datum	NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							ML	Brown to dark brown silt with organic matter (wood chips) (medium stiff, moist) (fill)			Vacuum excavation to approximately 5 feet
1315	5		0	2		1						
			18	13		2 / AL		SM	Tan to brown silty sand (medium dense, moist) (decomposed Scappoose formation)	48		AL (non-plastic)
1310			14	18		3						
	10		14	9		4 / %F			Becomes red streaked and loose	74	40	
			17	12		5			Becomes medium dense			
1305			18	13		6						
			17	13		7			Becomes orange			
	15		15	11		8 / MC			Becomes brown and with trace gravel	70		
1300	20		17	9		9 / %F			Becomes loose	56	35	
1295	25		10	5		10						
1290	30		11	16		11			Becomes medium dense			
1285								GM	Brown silty gravel with sand (medium dense, moist) (decomposed to predominantly decomposed Columbia River basalt)			
	35											

Note: See Figure A-1 for explanation of symbols; Figure A-2 for explanation of bedrock terms.
Coordinates Data Source: Horizontal approximated based on GPS. Vertical approximated based on Topographic Survey.

Log of Boring NM B-2

GEOENGINEERS

Project: North Mist Compressor Station Expansion
Project Location: Columbia County, Oregon
Project Number: 6024-308-00

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		4	16		12						
1280												
	40		18	46		13 / SA			Becomes dense	32	27	%G = 57
1275												
	45		8	35		14						
1270												
	50		9	37		15						
1265												
	55		10	30		16						
1260												
	60		12	51		17			Becomes very dense			

Log of Boring NM B-2 (continued)

GeoEngineers

Project: North Mist Compressor Station Expansion
Project Location: Columbia County, Oregon
Project Number: 6024-308-00

Figure A-4
Sheet 2 of 2

Drilled	Start 6/13/2023	End 6/15/2023	Total Depth (ft)	101.5	Logged By WCS Checked By AB	Driller	Western States Soil Conservation, Inc.	Drilling Method: Mud Rotary

Surface Elevation (ft) 1322 Vertical Datum NAVD88	Hammer Data: Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	CME 55 Track Rig
Latitude 46.047902 Longitude -123.298618	System Datum: NAD83 (feet)	Groundwater not observed at time of exploration	

Notes: 3-inch-diameter PVC casing installed to full depth of boring upon completion.

Elevation (feet)	Depth (feet)	Interval Recovered Soil (in)/Rock (%)	Blows/foot	Sample/Run Testing	RQD %	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
1322	0						ML	Brown to dark brown silt with organic matter (wood chips) (medium stiff, moist) (fill)			Vacuum excavation to approximately 5 feet
1320											
	5	4	5	1 MC					64		
1315		0	5	2							
		2	3	3				Becomes dark brown and soft			
							SM	Tan silty sand with orange mottling (medium dense, moist) (decomposed Scappoose formation)			
	10	17	11	4							
1310		18	30	5				Becomes dense			
		17	44	6							
		17	38	7				Becomes with gravel			
	15	12	44	8							
1305											
	20	12	39	9 %F					32	18	
1300											
							GM	Brown-red silty gravel with sand (very dense, moist) (decomposed to predominantly decomposed Columbia River basalt)			
	25	12	95	10 SA					23	12	%G = 63
1295											
	30	1.5	50/5"	11				Becomes gray			
1290											
	35										

Note: See Figure A-1 for explanation of symbols; Figure A-2 for explanation of bedrock terms.
Coordinates Data Source: Horizontal approximated based on GPS. Vertical approximated based on Topographic Survey.

Date:11/2/23 Path:P:\6\6024308\GINT\602430800.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_SOIL_ROCK_%F_NO_GW

Log of Boring NM B-3

GeoEngineers

Project: North Mist Compressor Station Expansion
Project Location: Columbia County, Oregon
Project Number: 6024-308-00

Elevation (feet)	Depth (feet)	Recovered Soil (in)/Rock (%)	Blows/foot	Sample/Run Testing	RQD %	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35	3	55	12						
	40	4	50/4"	13			Becomes moderately weathered			
		88		Run 1		Basalt	Dark gray and light black basalt, slightly weathered to fresh, very closely spaced fractured, very poor quality and hard (Columbia River basalt)			Switched to HQ coring
		85		Run 2	24		Becomes closely spaced fractured			
	50	60		Run 3	0		Become moderately weathered			
		100		Run 4	33		Becomes slightly weathered and poor quality			
	55	72		Run 5	13		Becomes closely to very closely spaced fractured and very poor quality			
	60	80		Run 6	15		Becomes grayish tan			
	65	91		Run 7	11					
		70		Run 9	0		Becomes dark gray and predominately decomposed weathered			
		100		Run 8	9		Becomes slightly weathered and very closely spaced fractured			
	70	93		Run 10	0		Becomes moderately spaced fractured			
	75	12		14			Becomes very closely fractured and highly weathered			Driller notes very closely fractured bedrock Switched to Mud Rotary

Elevation scale: 1285, 1280, 1275, 1270, 1265, 1260, 1255, 1250

Log of Boring NM B-3 (continued)

GeoEngineers

Project: North Mist Compressor Station Expansion
Project Location: Columbia County, Oregon
Project Number: 6024-308-00

Figure A-5
Sheet 2 of 3

Date:11/2/23 Path:P:\6\602430800\GINT\602430800.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_SOIL_ROCK_%F_NO_GW

Elevation (feet)	Depth (feet)	Interval	Recovered Soil (in)/Rock (%)	Blows/foot	Sample/Run Testing	RQD %	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
1245												
	80		0	100/1"	15							
1240												
	85		0	100/1"	16							
1235												
	90		0	100/1"	17							
1230												
	95		0	100/0"	18							
1225												
	100		0	100/1"	19							

Log of Boring NM B-3 (continued)

GEOENGINEERS

Project: North Mist Compressor Station Expansion
Project Location: Columbia County, Oregon
Project Number: 6024-308-00

Drilled	Start 6/13/2023	End 6/13/2023	Total Depth (ft) 61.5	Logged By WCS / Checked By AB	Driller: Western States Soil Conservation, Inc. / Drilling Method: Mud Rotary
Surface Elevation (ft) / Vertical Datum	1324 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	CME 55 Track Rig
Latitude / Longitude	46.04772944 / -123.2980275	System Datum	NAD83 (feet)	Groundwater not observed at time of exploration	

Notes:

Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
0							SM	Brown-black silty sand with organic matter (medium dense, moist) (fill)			Vacuum excavation to approximately 5 feet
5	X	4	3		1		MH	Brown elastic silt with sand (soft, moist) (decomposed Scappoose formation)			
	X	18	6		2			Becomes medium stiff			
	X	14	10		3 / AL			Becomes stiff and wet	67		AL (LL = 60, PI = 24)
10	X	14	6		4						
	X	7	7		5		SM	Brown silty sand (loose, moist)			
	X	8	5		6						
	X	9	7		7						
15	X	8	5		8						
20	X	9	6		9 / %F				55	40	
25	X	18	42		10			Becomes dense with gravel			
30	X	16	18		11 / %F			Becomes medium dense	51	14	
35											

Elevation (feet): 1320, 1315, 1310, 1305, 1300, 1295, 1290

Note: See Figure A-1 for explanation of symbols; Figure A-2 for explanation of bedrock terms.
Coordinates Data Source: Horizontal approximated based on GPS. Vertical approximated based on Topographic Survey.

Log of Boring NM B-4

GEOENGINEERS

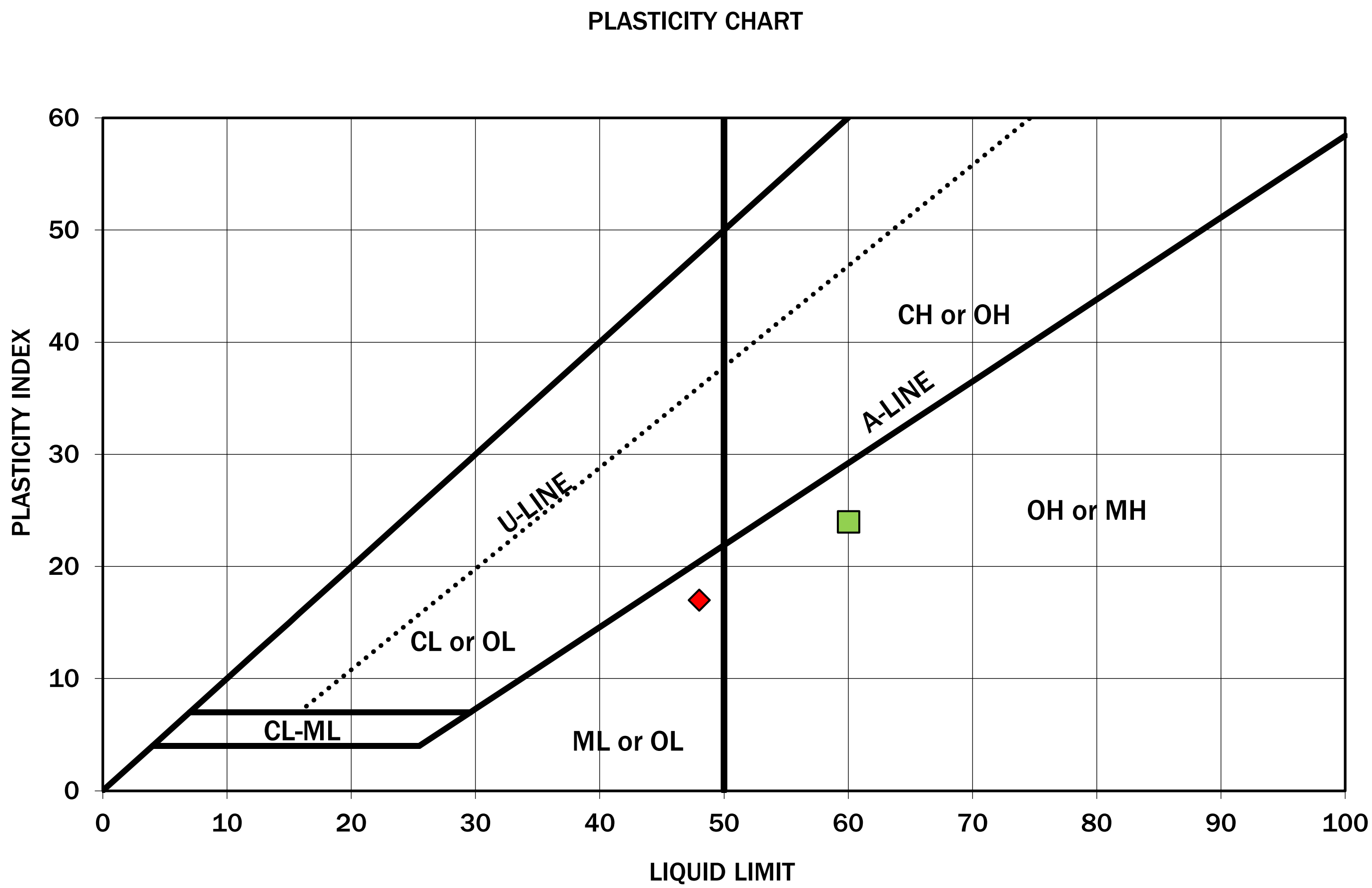
Project: North Mist Compressor Station Expansion
Project Location: Columbia County, Oregon
Project Number: 6024-308-00

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		17	19		12						
1285												
	40		18	24		13						
1280												
	45		15	24		14 / SA				50	18	%G = 32
1275												
	50		11	23		15						
								SP	Gray gravelly sand (dense, moist) (decomposed Columbia River basalt)			
1270												
	55		17	32		16 / %F				42	23	
1265												
	60			40		17						

Log of Boring NM B-4 (continued)

GEOENGINEERS

Project: North Mist Compressor Station Expansion
Project Location: Columbia County, Oregon
Project Number: 6024-308-00



Symbol	Boring Number	Depth (feet)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Soil Description
◆	NMB-1	12.5	46	48	17	Silt (ML)
■	NMB-4	8	67	60	24	Elastic Silt (MH)

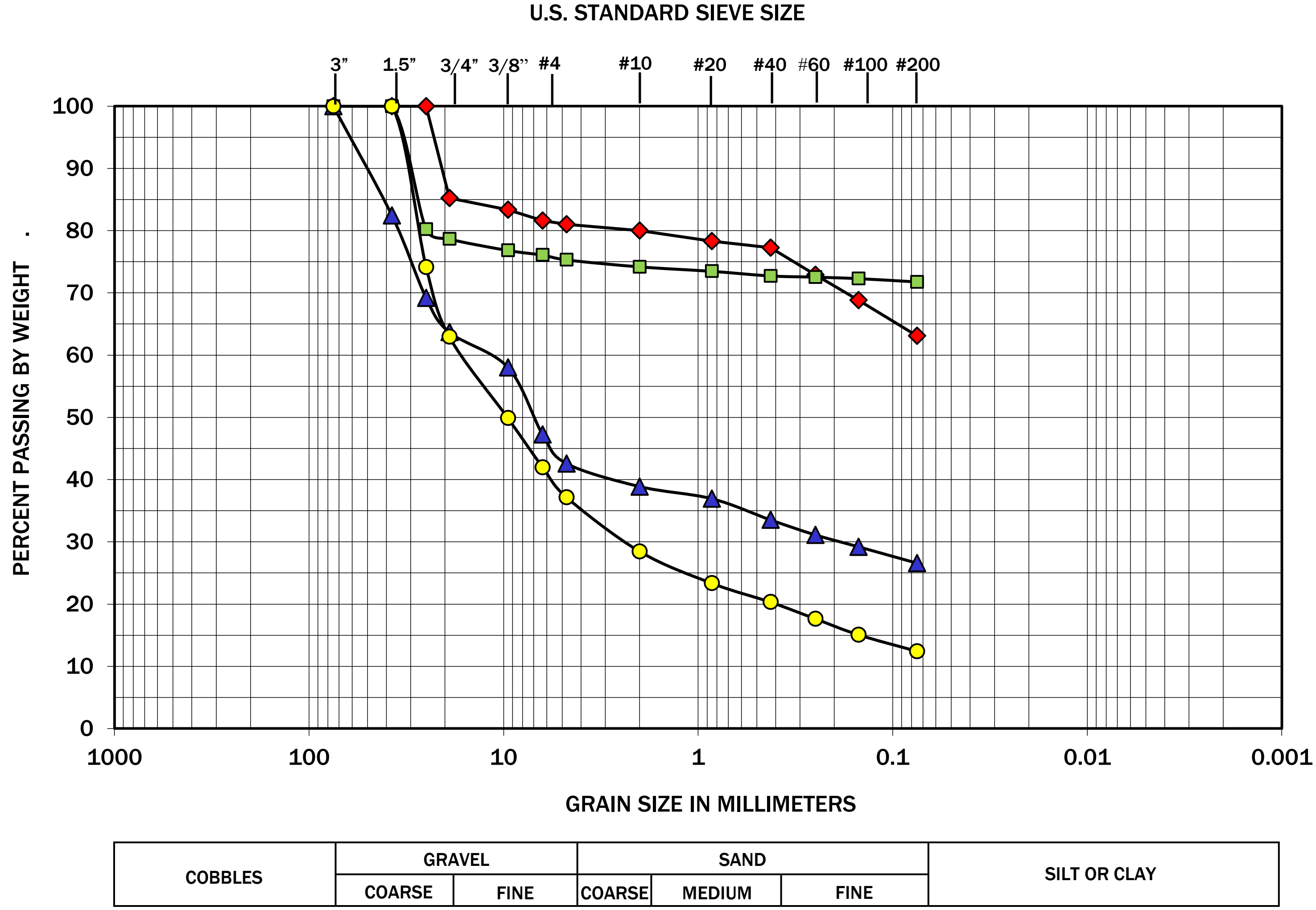
The liquid limit and plasticity index were obtained in general accordance with ASTM D 4318.

Atterberg Limits Test Results

Mist Resiliency Project
North Mist Compressor Station Resiliency
Columbia County, Oregon

GeoEngineers

Figure A-7



Symbol	Boring Number	Sample Depth (feet)	Moisture Content (%)	Gravel (%)	Sand (%)	Fines (%)	Soil Description (USCS)
◆	NMB-1	30	60	19	18	63	Gravelly Silt with Sand (ML)
■	NMB-1	50	51	25	3	72	Silt with Gravel (ML)
▲	NMB-2	40	32	57	16	27	Silty Gravel with Sand (GM)
●	NMB-3	25	23	63	25	12	Silty Gravel with Sand (GM)

Note: This report may not be reproduced, except in full, without written approval of GeoEngineers, Inc. Test results are applicable only to the specific sample on which they were performed, and should not be interpreted as representative of any other samples obtained at other times, depths or locations, or generated by separate operations or processes. The grain size analysis results were obtained in general accordance with ASTM C 136.

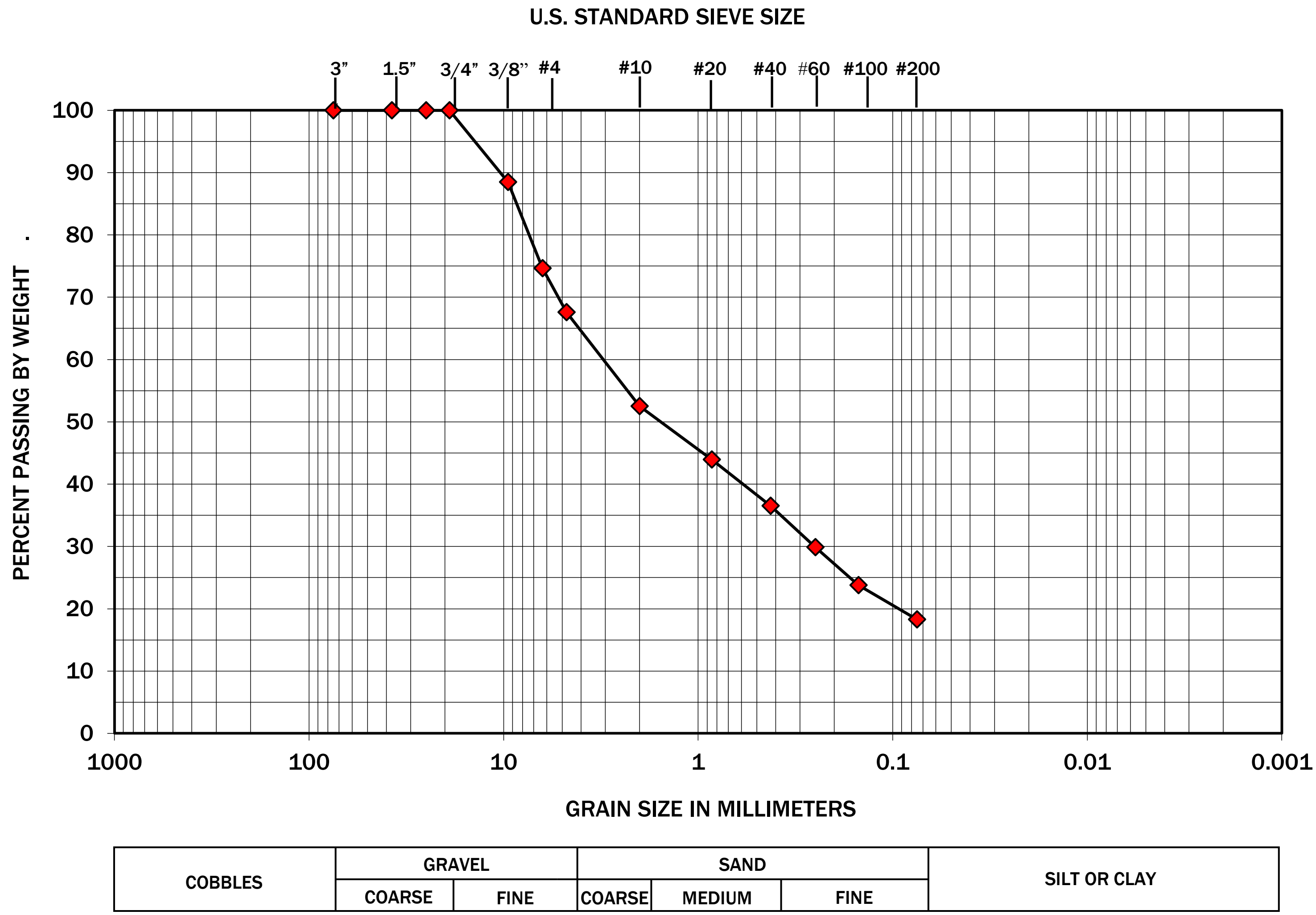
Sieve Analysis Results

Mist Resiliency Project
North Mist Compressor Station Resiliency
Columbia County, Oregon

GeoEngineers

Figure A-8

6024-308-00 Date Exported: 11/03/2023



Symbol	Boring Number	Sample Depth (feet)	Moisture Content (%)	Gravel (%)	Sand (%)	Fines (%)	Soil Description (USCS)
◆	NMB-4	45	50	32	49	18	Silty Sand with Gravel (SM)

Sieve Analysis Results

Mist Resiliency Project
North Mist Compressor Station Resiliency
Columbia County, Oregon

GeoEngineers

Figure A-9

 The grain size analysis results were obtained in general accordance with ASTM C 136.

6024-308-00 Date Exported: 11/03/2023

Photograph 1. Boring B-1: 43 feet to 51 feet

Photograph 2. Boring B-1: 51 feet to 55 feet

Rock Core Photograph

Mist Resiliency Project
North Mist Compressor Station Resiliency
Columbia County, Oregon

GEOENGINEERS

Figure A-10

*Note: Fractures in photographs may be mechanical, and do not necessarily reflect in-tact rock core character.

6024-308-00 Date Exported: 08/31/2023

Photograph 3. Boring B-1: 55 feet to 65 feet

Photograph 4. Boring B-1: 65 feet to 75 feet

Rock Core Photograph	
Mist Resiliency Project North Mist Compressor Station Resiliency Columbia County, Oregon	
GeoEngineers	Figure A-11

*Note: Fractures in photographs may be mechanical, and do not necessarily reflect in-tact rock core character.

6024-308-00 Date Exported: 08/31/2023

Photograph 5. Boring B-1: 75 feet to 76.5 feet

*Note: Fractures in photographs may be mechanical, and do not necessarily reflect in-tact rock core character.

6024-308-00 Date Exported: 08/31/2023

Rock Core Photograph

Mist Resiliency Project
North Mist Compressor Station Resiliency
Columbia County, Oregon

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Figure A-12

PO Box 880
Midvale, Utah 84047
801.849.0055
gerhartcole.com

To:	Mr. Jarad Hoffman GeoEngineers, Inc 4000 Kruse Way Place Bldg. 3 Suite 200 Lake Oswego, Oregon 97035
Project:	Mist Resiliency
Gerhart Cole Project Number:	23-1681
Client Project Number	6024-308-00
Report Date:	August 24, 2023

Attached are the requested laboratory test results. These laboratory results are for the addressee and must be presented in their entirety without alteration, except with permission.

Gerhart Cole's Laboratory is accredited through the AASHTO Accreditation Program. The results provided were tested in general accordance with the referenced standards. Any test methods reported in this document outside the scope of this accreditation are marked with an asterisk in the header of the individual test report.

Gerhart Cole will dispose of samples two (2) weeks after the date above. Please contact us for storage pricing.

Approved By:____
Zach Gibbs, PE (UT, MI) Laboratory Director

AASHTO ACCREDITED

Moisture Content Determination

ASTM D2937 / D2216

Project: **Mist Resiliency**

No: **6024-308-00 (23-1681)**

Location: **-**

X:\PROJECTS\23-1681 Mist Resiliency (6024-308-00)\[2023-08-07_MC.xlsx]1

Sample Info.	Sample:	**MMB-1**	**MMB-2**	**NMB-1**	**NMB-2**	**NMB-3**	**NMB-4**
	Depth:	**6.5-8.5 ft**	**11.5-13.5 ft**	**Composite 5-20 ft**	**Composite 5-14 ft**	**Composite 5-14 ft**	**Composite 5-12.5 ft**
	Date Sampled:	**-**	**-**	**-**	**-**	**-**	**-**
	Date tested:	**09-Aug-23**	**09-Aug-23**	**09-Aug-23**	**09-Aug-23**	**09-Aug-23**	**09-Aug-23**
	Laboratory sample description:	**tn - gy clay**	**lt bn - lt ol bn clay**	**bn - dk bn clay**	**lt ol bn - lt bn silt**	**bn - dk bn clay**	**dk yl bn - lt bn clay**
Moisture	Wet soil + tare (g)	445.21	422.31	1176.68	497.10	688.50	805.90
	Dry soil + tare (g)	336.92	300.69	893.25	353.91	483.31	618.31
	Tare (g)	117.33	118.10	119.26	150.66	117.02	197.82
	Moisture content, w (%)	49.3	66.6	36.6	70.5	56.0	44.6
Comments							
QC/QA	Tested By:	JC	JC	JC	JC	JC	JC
	Reduced By:	JC	JC	JC	JC	JC	JC
	Reviewed By:	DBW	DBW	DBW	DBW	DBW	DBW

Minimum Laboratory Soil Resistivity*

After AASHTO T 288

Project: Mist Resiliency

No: 6024-308-00 (23-1681)

Location: -

Section								
Sample Info.	Test Hole/Pit:	MMB-1			MMB-2			
	Depth (ft):	6.5-8.5 ft			11.5-13.5 ft			
	Date Sampled:	-			-			
	Date Tested:	8/9/2023			8/9/2023			
	Laboratory sample description:	tn - gy clay			lt bn - lt ol bn clay			
	Meter Serial Number	4385			4777			
Soil Data	Wet soil + tare (g)	497.13			513.5			
	Air dry soil + tare (g)	383.06			365.44			
	Tare (g)	117.19			117.22			
	Moisture Content, w (%)	42.9			59.6			
	Dry Soil after #10 sieve (g)	246.64			212.01			
Moisture	Water for 10% initial MC (mL)	24.66			21.20			
	% / Step (%)	5			5			
	Water for % / Step (mL)	12.3			10.6			
Resistivity data	Trial	Soil condition (%)	Resistivity (Ω-cm)	Soil condition (%)	Resistivity (Ω-cm)	Soil condition (%)	Resistivity (Ω-cm)	Soil condition (%)	Resistivity (Ω-cm)
	1	Initial	58,000			Initial	1,080,000		
	2	Initial +5	17,000			Initial +5	170,000		
	3	Initial +10	8,000			Initial +10	104,000		
	4	Initial +15	5,700			Initial +15	40,500		
	5	Initial +20	4,500			Initial +20	14,000		
	6	Initial +25	4,300			Initial +25	15,500		
	7	Initial +30	4,300			Initial +30	5,700		
	8	Initial +35	4,800			Initial +35	5,700		
	9	Initial +40	4,500			Initial +40	6,300		
	10					Initial +45	6,350		
	Minimum resistivity (Ω-cm)	4,300				5,700			
QC/QA	Tested By:	JWS				CJS			
	Reduced By:	JC				JC			
	Reviewed By:	DBW				DBW			

Comments:

X:\PROJECTS\23-1681 Mist Resiliency (6024-308-00)\[2023-08-07_Resistivity.xlsx]1

Minimum Laboratory Soil Resistivity*

After AASHTO T 288

Project: Mist Resiliency
No: 6024-308-00 (23-1681)
Location: -

Sample Info.	Test Hole/Pit:	NMB-1				NMB-2			
	Depth (ft):	Composite 5-20 ft				Composite 5-14 ft			
	Date Sampled:	-				-			
	Date Tested:	8/9/2023				8/9/2023			
	Laboratory sample description:	bn - dk bn clay				lt ol bn - lt bn silt			
	Meter Serial Number	4777				4777			
Soil Data	Wet soil + tare (g)	573.6				677.11			
	Air dry soil + tare (g)	488.07				520.89			
	Tare (g)	196.72				198.02			
	Moisture Content, w (%)	29.4				48.4			
	Dry Soil after #10 sieve (g)	282.17				260.76			
Moisture	Water for 10% initial MC (mL)	28.22				26.08			
	% / Step (%)	5				5			
	Water for % / Step (mL)	14.1				13.0			
Resistivity data	Trial	Soil condition (%)	Resistivity (Ω-cm)	Soil condition (%)	Resistivity (Ω-cm)	Soil condition (%)	Resistivity (Ω-cm)	Soil condition (%)	Resistivity (Ω-cm)
	1	Initial	990,000			Initial	500,000		
	2	Initial +5	270,000			Initial +5	74,000		
	3	Initial +10	110,000			Initial +10	47,000		
	4	Initial +15	49,000			Initial +15	29,000		
	5	Initial +20	22,000			Initial +20	21,000		
	6	Initial +25	23,000			Initial +25	14,000		
	7	Initial +30	24,000			Initial +30	7,500		
	8					Initial +35	7,700		
	9					Initial +40	8,000		
	10								
	Minimum resistivity (Ω-cm)	22,000				7,500			
QC/QA	Tested By:	TK				TK			
	Reduced By:	JC				JC			
	Reviewed By:	DBW				DBW			

Comments:

X:\PROJECTS\23-1681 Mist Resiliency (6024-308-00)\[2023-08-07_Resistivity.xlsx]2

Minimum Laboratory Soil Resistivity*

After AASHTO T 288

Project: Mist Resiliency

No: 6024-308-00 (23-1681)

Location: -

Sample Info.	Test Hole/Pit:	NMB-3				NMB-4			
	Depth (ft):	Composite 5-14 ft				Composite 5-12.5 ft			
	Date Sampled:	-				-			
	Date Tested:	8/9/2023				8/9/2023			
	Laboratory sample description:	bn - dk bn clay				dk yl bn - lt bn clay			
	Meter Serial Number	4777				4777			
Soil Data	Wet soil + tare (g)	699.22				636.28			
	Air dry soil + tare (g)	553.47				519.09			
	Tare (g)	196.58				196.16			
	Moisture Content, w (%)	40.8				36.3			
	Dry Soil after #10 sieve (g)	251.56				241.99			
Moisture	Water for 10% initial MC (mL)	25.16				24.20			
	% / Step (%)	5				5			
	Water for % / Step (mL)	12.6				12.1			
Resistivity data	Trial	Soil condition (%)	Resistivity (Ω-cm)	Soil condition (%)	Resistivity (Ω-cm)	Soil condition (%)	Resistivity (Ω-cm)	Soil condition (%)	Resistivity (Ω-cm)
	1	Initial	510,000			Initial	875,000		
	2	Initial +5	89,000			Initial +5	235,000		
	3	Initial +10	36,000			Initial +10	85,500		
	4	Initial +15	24,000			Initial +15	39,000		
	5	Initial +20	15,000			Initial +20	23,000		
	6	Initial +25	9,200			Initial +25	19,000		
	7	Initial +30	8,900			Initial +30	19,000		
	8	Initial +35	9,300			Initial +35	19,500		
	9	Initial +40	9,900			Initial +40	20,000		
	10								
	Minimum resistivity (Ω-cm)	8,900				19,000			
QC/QA	Tested By:	TK				TK			
	Reduced By:	JC				JC			
	Reviewed By:	DBW				DBW			

Comments:

X:\PROJECTS\23-1681 Mist Resiliency (6024-308-00)\[2023-08-07_Resistivity.xlsx]3

8/16/2023

Work Order: 23H0932
Project: 6024-308-00 (23-1681)

Gerhart Cole, Inc.
Attn: Zach Gibbs
7657 South Holden Street
Midvale, UT 84047

Client Service Contact: 801.262.7299

The analyses presented on this report were performed in accordance with the National Environmental Laboratory Accreditation Program (NELAP) unless noted in the comments, flags, or case narrative. If the report is to be used for regulatory compliance, it should be presented in its entirety, and not be altered.

Approved By:

Reed Hendricks, Director of Operations

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Certificate of Analysis

Gerhart Cole, Inc.
Zach Gibbs
7657 South Holden Street
Midvale, UT 84047

PO#:
Receipt: **8/9/23 16:45 @ 24.9** °C
Date Reported: 8/16/2023
Project Name: **6024-308-00 (23-1681)**

Sample ID: **MMB-1 / 6.5-8.5 ft**

Matrix: **Solid**
Lab ID: **23H0932-01**
Date Sampled: **8/7/23 0:00**
Sampled By: **client**

	Result	Units	Minimum Reporting Limit	Method	Preparation Date/Time	Analysis Date/Time	Flag(s)
Inorganic							
Chloride, Soluble (IC)	ND	mg/kg dry	15	EPA 300.0	8/10/23	8/10/23	
pH	5.0	pH Units	0.1	EPA 9045D	8/10/23	8/10/23	
Sulfate, Soluble (IC)	ND	mg/kg dry	15	EPA 300.0	8/10/23	8/10/23	
Total Solids	68.0	%	0.1	CTF8000	8/10/23	8/11/23	
Metals							
Sulfur, Total	58.5	mg/kg dry	0.25	EPA 6010D	8/11/23	8/14/23	

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PO#:
Receipt: **8/9/23 16:45 @ 24.9** °C
Date Reported: 8/16/2023
Project Name: **6024-308-00 (23-1681)**

Sample ID: **MMB-2 / 11.5-13.5 ft**

Matrix: **Solid**
Lab ID: **23H0932-02**
Date Sampled: **8/7/23 0:00**
Sampled By: **client**

	Result	Units	Minimum Reporting Limit	Method	Preparation Date/Time	Analysis Date/Time	Flag(s)
Inorganic							
Chloride, Soluble (IC)	ND	mg/kg dry	17	EPA 300.0	8/10/23	8/10/23	
pH	5.0	pH Units	0.1	EPA 9045D	8/10/23	8/10/23	
Sulfate, Soluble (IC)	ND	mg/kg dry	17	EPA 300.0	8/10/23	8/10/23	
Total Solids	59.0	%	0.1	CTF8000	8/10/23	8/11/23	
Metals							
Sulfur, Total	86.3	mg/kg dry	0.22	EPA 6010D	8/11/23	8/14/23	

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PO#:
Receipt: **8/9/23 16:45 @ 24.9** °C
Date Reported: 8/16/2023
Project Name: **6024-308-00 (23-1681)**

Sample ID: **NMB-1 / Composite 5-20 ft**

Matrix: **Solid**
Lab ID: **23H0932-03**
Date Sampled: **8/7/23 0:00**
Sampled By: **client**

	Result	Units	Minimum Reporting Limit	Method	Preparation Date/Time	Analysis Date/Time	Flag(s)
Inorganic							
Chloride, Soluble (IC)	ND	mg/kg dry	14	EPA 300.0	8/10/23	8/10/23	
pH	5.4	pH Units	0.1	EPA 9045D	8/10/23	8/10/23	
Sulfate, Soluble (IC)	ND	mg/kg dry	14	EPA 300.0	8/10/23	8/10/23	
Total Solids	69.6	%	0.1	CTF8000	8/10/23	8/11/23	
Metals							
Sulfur, Total	145	mg/kg dry	0.18	EPA 6010D	8/11/23	8/14/23	

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Sandy, UT 84070
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PO#:
Receipt: **8/9/23 16:45 @ 24.9** °C
Date Reported: 8/16/2023
Project Name: **6024-308-00 (23-1681)**

Sample ID: **NMB-2 / Composite 5-14 ft**

Matrix: **Solid**

Lab ID: **23H0932-04**

Date Sampled: **8/7/23 0:00**

Sampled By: **client**

	Result	Units	Minimum Reporting Limit	Method	Preparation Date/Time	Analysis Date/Time	Flag(s)
Inorganic							
Chloride, Soluble (IC)	ND	mg/kg dry	18	EPA 300.0	8/10/23	8/10/23	
pH	4.8	pH Units	0.1	EPA 9045D	8/10/23	8/10/23	
Sulfate, Soluble (IC)	ND	mg/kg dry	18	EPA 300.0	8/10/23	8/10/23	
Total Solids	56.5	%	0.1	CTF8000	8/10/23	8/11/23	
Metals							
Sulfur, Total	159	mg/kg dry	0.40	EPA 6010D	8/11/23	8/14/23	

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Certificate of Analysis

Gerhart Cole, Inc.
Zach Gibbs
7657 South Holden Street
Midvale, UT 84047

PO#:
Receipt: **8/9/23 16:45 @ 24.9** °C
Date Reported: 8/16/2023
Project Name: **6024-308-00 (23-1681)**

Sample ID: **NMB-3 / Composite 5-14 ft**
Matrix: **Solid**
Lab ID: **23H0932-05**
Date Sampled: **8/7/23 0:00**
Sampled By: **client**

	Result	Units	Minimum Reporting Limit	Method	Preparation Date/Time	Analysis Date/Time	Flag(s)
Inorganic							
Chloride, Soluble (IC)	ND	mg/kg dry	15	EPA 300.0	8/10/23	8/10/23	
pH	5.1	pH Units	0.1	EPA 9045D	8/10/23	8/10/23	
Sulfate, Soluble (IC)	ND	mg/kg dry	15	EPA 300.0	8/10/23	8/10/23	
Total Solids	65.5	%	0.1	CTF8000	8/10/23	8/11/23	
Metals							
Sulfur, Total	51.3	mg/kg dry	0.43	EPA 6010D	8/11/23	8/14/23	

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Zach Gibbs
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Midvale, UT 84047

PO#:
Receipt: **8/9/23 16:45 @ 24.9** °C
Date Reported: 8/16/2023
Project Name: **6024-308-00 (23-1681)**

Sample ID: **NMB-4 / Composite 5-12.5**

Matrix: **Solid**
Lab ID: **23H0932-06**
Date Sampled: **8/7/23 0:00**
Sampled By: **client**

	Result	Units	Minimum Reporting Limit	Method	Preparation Date/Time	Analysis Date/Time	Flag(s)
Inorganic							
Chloride, Soluble (IC)	ND	mg/kg dry	14	EPA 300.0	8/10/23	8/10/23	
pH	5.4	pH Units	0.1	EPA 9045D	8/10/23	8/10/23	
Sulfate, Soluble (IC)	ND	mg/kg dry	14	EPA 300.0	8/10/23	8/10/23	
Total Solids	70.3	%	0.1	CTF8000	8/10/23	8/11/23	
Metals							
Sulfur, Total	172	mg/kg dry	0.34	EPA 6010D	8/11/23	8/14/23	

Certificate of Analysis

Gerhart Cole, Inc.
Zach Gibbs
7657 South Holden Street
Midvale, UT 84047

PO#:
Receipt: **8/9/23 16:45 @ 24.9** °C
Date Reported: 8/16/2023
Project Name: **6024-308-00 (23-1681)**

Report Footnotes

Abbreviations

ND = Not detected at the corresponding Minimum Reporting Limit (MRL).
1 mg/L = one milligram per liter or 1 mg/kg = one milligram per kilogram = 1 part per million.
1 ug/L = one microgram per liter or 1 ug/kg = one microgram per kilogram = 1 part per billion.
1 ng/L = one nanogram per liter or 1 ng/kg = one nanogram per kilogram = 1 part per trillion.
On calculated parameters, there may be a slight difference between summing the rounded values shown on the report vs the unrounded values used in the calculation.

CHEMTECH - FORD ANALYTICAL LABORATORY

CHAIN OF CUSTODY

COMPANY: Gerhart Cole
ADDRESS: 7657 s. Holden St.
CITY/STATE/ZIP: Midvale, Utah 84047
PHONE #: 801.791.3641 FAX:
CONTACT: Zach Gibbs PROJECT: 6024-308-00 (23-1681)
EMAIL: zachg@gerhartcole.com

BILLING ADDRESS:
BILLING CITY/STATE/ZIP:
PURCHASE ORDER #:

CHEMTECH-FORD LABORATORIES

TURNAROUND REQUIRED:*
* Expedited turnaround subject to additional charge

W-

Sample Receipt Conditions:
- ( ) Custody Seals Present
- (✓) Containers Intact
- (✓) COC and Labels Match
- (✗) Received on Ice
- (✓) Correct Containers
- (✓) COC Included
- (✓) COC Complete
- (✓) Sufficient Sample Volume
- ( ) Headspace Present (VOC)
- ( ) Temperature Blank
- (✓) Received within Hold

Checked by: AW

23H0932 AW

Lab Use Only	#	LOCATION / IDENTIFICATION	DATE	TIME	MATRIX	Field: Residual Chlorine	pH	Sulfates and Chlorides	Total Sulphur	Total Coliform + E. coli (Present/Absent)	Total Coliform + E. coli (Enumerated)	HPC (Plate Count)	E. Coli Only
01	1.	MMB-1 / ~~5 &~~ 6.5-8.5 ft	8/7/2023				×	×	×				
02	2.	MMB-2 / ~~10 &~~ 11.5-13.5 ft	8/7/2023				×	×	×				
03	3.	NMB-1 / Composite 5-20 ft	8/7/2023				×	×	×				
04	4.	NMB-2 / Composite 5-14 ft	8/7/2023				×	×	×				
05	5.	NMB-3 / Composite 5-14 ft	8/7/2023				×	×	×				
06	6.	NMB-4 / Composite 5-12.5	8/7/2023				×	×	×				
	7.												
	8.												
	9.						Q						
	10.												

Sampled by: [print]
Sampled by: [signature]

Special Instructions:

ON ICE | NOT ON ICE (circled) | Temp (C°): 24.9

Samples received outside the EPA recommended temperature range of 0-6 C° may be rejected.

Relinquished by: [signature]	Date/Time	Received by: [signature]	Date/Time
Alicia Walburger	8/9/23 16:45	[signature]	8/9/23 16:45

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Payment Terms are net 30 days OAC. 1.5% interest charge per month (18% per annum). Client agress to pay collection costs and attorney's fees.

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Lenexa, KS 66219
(913)599-5665

August 23, 2023

Zach Gibbs
Gerhart Cole
7657 South Holden Street
Midvale, UT 84047

RE: Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

Dear Zach Gibbs:

Enclosed are the analytical results for sample(s) received by the laboratory on August 10, 2023. The results relate only to the samples included in this report. Results reported herein conform to the applicable TNI/NELAC Standards and the laboratory's Quality Manual, where applicable, unless otherwise noted in the body of the report.

The test results provided in this final report were generated by each of the following laboratories within the Pace Network:

- Pace Analytical Services - Kansas City

If you have any questions concerning this report, please feel free to contact me.

Sincerely,

Heather Wilson
heather.wilson@pacelabs.com
1(913)563-1407
Project Manager

Enclosures

CERTIFICATIONS

Project:	MIST RESILIENCY 23-1681
Pace Project No.:	60435134

Pace Analytical Services Kansas

9608 Loiret Boulevard, Lenexa, KS 66219
Missouri Inorganic Drinking Water Certification #: 10090
Arkansas Drinking Water
Arkansas Certification #: 88-00679
Illinois Certification #: 2000302023-5
Iowa Certification #: 118
Kansas/NELAP Certification #: E-10116
Louisiana Certification #: 03055
Nevada Certification #: KS000212023-1
Oklahoma Certification #: 2022-057
Florida: Cert E871149 SEKS WET
Texas Certification #: T104704407-22-16
Utah Certification #: KS000212022-12
Illinois Certification #: 004592
Kansas Field Laboratory Accreditation: # E-92587
Missouri SEKS Micro Certification: 10070

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SAMPLE SUMMARY

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

Lab ID	Sample ID	Matrix	Date Collected	Date Received
60435134001	**MMB-1**	Solid	08/09/23 08:00	08/10/23 08:50
60435134002	**MMB-2**	Solid	08/09/23 08:00	08/10/23 08:50
60435134003	**NMB-1**	Solid	08/09/23 08:00	08/10/23 08:50
60435134004	**NMB-2**	Solid	08/09/23 08:00	08/10/23 08:50
60435134005	**NMB-3**	Solid	08/09/23 08:00	08/10/23 08:50
60435134006	**NMB-4**	Solid	08/09/23 08:00	08/10/23 08:50

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SAMPLE ANALYTE COUNT

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

Lab ID	Sample ID	Method	Analysts	Analytes Reported	Laboratory
60435134001	**MMB-1**	ASTM D2974	DWC	1	PASI-K
		SM 2320B	BMT	3	PASI-K
		SM 2580B	TML	1	PASI-K
60435134002	**MMB-2**	ASTM D2974	DWC	1	PASI-K
		SM 2320B	BMT	3	PASI-K
		SM 2580B	TML	1	PASI-K
60435134003	**NMB-1**	ASTM D2974	DWC	1	PASI-K
		SM 2320B	BMT	3	PASI-K
		SM 2580B	TML	1	PASI-K
60435134004	**NMB-2**	ASTM D2974	DWC	1	PASI-K
		SM 2320B	BMT	3	PASI-K
		SM 2580B	TML	1	PASI-K
60435134005	**NMB-3**	ASTM D2974	DWC	1	PASI-K
		SM 2320B	BMT	3	PASI-K
		SM 2580B	TML	1	PASI-K
60435134006	**NMB-4**	ASTM D2974	DWC	1	PASI-K
		SM 2320B	BMT	3	PASI-K
		SM 2580B	TML	1	PASI-K

PASI-K = Pace Analytical Services - Kansas City

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ANALYTICAL RESULTS

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

Sample: MMB-1 **Lab ID: 60435134001** Collected: 08/09/23 08:00 Received: 08/10/23 08:50 Matrix: Solid

Results reported on a "dry weight" basis and are adjusted for percent moisture, sample size and any dilutions.

Parameters	Results	Units	Report Limit	DF	Prepared	Analyzed	CAS No.	Qual
Percent Moisture	Analytical Method: ASTM D2974 Pace Analytical Services - Kansas City							
Percent Moisture	**32.9**	%	0.50	1		08/11/23 10:41		
2320B Alkalinity	Analytical Method: SM 2320B Preparation Method: SM 2320B Pace Analytical Services - Kansas City							
Alkalinity,Bicarbonate (CaCO3)	ND	mg/kg	292	1	08/21/23 07:58	08/21/23 13:40		
Alkalinity,Carbonate (CaCO3)	ND	mg/kg	292	1	08/21/23 07:58	08/21/23 13:40		
Alkalinity, Total as CaCO3	ND	mg/kg	292	1	08/21/23 07:58	08/21/23 13:40		
Oxidation/Reduction Potential	Analytical Method: SM 2580B Pace Analytical Services - Kansas City							
Oxidation/Reduction Potential	**272.2**	mV	1.0	1		08/14/23 15:07		

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ANALYTICAL RESULTS

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

Sample: MMB-2 **Lab ID: 60435134002** Collected: 08/09/23 08:00 Received: 08/10/23 08:50 Matrix: Solid

Results reported on a "dry weight" basis and are adjusted for percent moisture, sample size and any dilutions.

Parameters	Results	Units	Report Limit	DF	Prepared	Analyzed	CAS No.	Qual
Percent Moisture	Analytical Method: ASTM D2974 Pace Analytical Services - Kansas City							
Percent Moisture	**37.2**	%	0.50	1		08/11/23 10:41		
2320B Alkalinity	Analytical Method: SM 2320B Preparation Method: SM 2320B Pace Analytical Services - Kansas City							
Alkalinity,Bicarbonate (CaCO3)	ND	mg/kg	304	1	08/21/23 07:58	08/21/23 13:48		
Alkalinity,Carbonate (CaCO3)	ND	mg/kg	304	1	08/21/23 07:58	08/21/23 13:48		
Alkalinity, Total as CaCO3	ND	mg/kg	304	1	08/21/23 07:58	08/21/23 13:48		
Oxidation/Reduction Potential	Analytical Method: SM 2580B Pace Analytical Services - Kansas City							
Oxidation/Reduction Potential	**221.4**	mV	1.0	1		08/14/23 15:23		

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ANALYTICAL RESULTS

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

Sample: NMB-1 **Lab ID: 60435134003** Collected: 08/09/23 08:00 Received: 08/10/23 08:50 Matrix: Solid

Results reported on a "dry weight" basis and are adjusted for percent moisture, sample size and any dilutions.

Parameters	Results	Units	Report Limit	DF	Prepared	Analyzed	CAS No.	Qual
Percent Moisture	Analytical Method: ASTM D2974 Pace Analytical Services - Kansas City							
Percent Moisture	**26.5**	%	0.50	1		08/11/23 10:41		
2320B Alkalinity	Analytical Method: SM 2320B Preparation Method: SM 2320B Pace Analytical Services - Kansas City							
Alkalinity,Bicarbonate (CaCO3)	ND	mg/kg	266	1	08/21/23 07:58	08/21/23 13:51		
Alkalinity,Carbonate (CaCO3)	ND	mg/kg	266	1	08/21/23 07:58	08/21/23 13:51		
Alkalinity, Total as CaCO3	ND	mg/kg	266	1	08/21/23 07:58	08/21/23 13:51		
Oxidation/Reduction Potential	Analytical Method: SM 2580B Pace Analytical Services - Kansas City							
Oxidation/Reduction Potential	**236.8**	mV	1.0	1		08/14/23 15:31		

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ANALYTICAL RESULTS

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

Sample: NMB-2 **Lab ID: 60435134004** Collected: 08/09/23 08:00 Received: 08/10/23 08:50 Matrix: Solid

Results reported on a "dry weight" basis and are adjusted for percent moisture, sample size and any dilutions.

Parameters	Results	Units	Report Limit	DF	Prepared	Analyzed	CAS No.	Qual
Percent Moisture	Analytical Method: ASTM D2974 Pace Analytical Services - Kansas City							
Percent Moisture	**42.7**	%	0.50	1		08/11/23 10:41		
2320B Alkalinity	Analytical Method: SM 2320B Preparation Method: SM 2320B Pace Analytical Services - Kansas City							
Alkalinity,Bicarbonate (CaCO3)	ND	mg/kg	327	1	08/21/23 07:58	08/21/23 13:54		
Alkalinity,Carbonate (CaCO3)	ND	mg/kg	327	1	08/21/23 07:58	08/21/23 13:54		
Alkalinity, Total as CaCO3	ND	mg/kg	327	1	08/21/23 07:58	08/21/23 13:54		
Oxidation/Reduction Potential	Analytical Method: SM 2580B Pace Analytical Services - Kansas City							
Oxidation/Reduction Potential	**269.8**	mV	1.0	1		08/14/23 15:35		

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ANALYTICAL RESULTS

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

Sample: NMB-3 **Lab ID: 60435134005** Collected: 08/09/23 08:00 Received: 08/10/23 08:50 Matrix: Solid

Results reported on a "dry weight" basis and are adjusted for percent moisture, sample size and any dilutions.

Parameters	Results	Units	Report Limit	DF	Prepared	Analyzed	CAS No.	Qual
Percent Moisture	Analytical Method: ASTM D2974 Pace Analytical Services - Kansas City							
Percent Moisture	**39.7**	%	0.50	1		08/11/23 10:42		
2320B Alkalinity	Analytical Method: SM 2320B Preparation Method: SM 2320B Pace Analytical Services - Kansas City							
Alkalinity,Bicarbonate (CaCO3)	ND	mg/kg	333	1	08/21/23 07:58	08/21/23 13:57		
Alkalinity,Carbonate (CaCO3)	ND	mg/kg	333	1	08/21/23 07:58	08/21/23 13:57		
Alkalinity, Total as CaCO3	ND	mg/kg	333	1	08/21/23 07:58	08/21/23 13:57		
Oxidation/Reduction Potential	Analytical Method: SM 2580B Pace Analytical Services - Kansas City							
Oxidation/Reduction Potential	**258.1**	mV	1.0	1		08/14/23 15:36		

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ANALYTICAL RESULTS

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

Sample: NMB-4 **Lab ID: 60435134006** Collected: 08/09/23 08:00 Received: 08/10/23 08:50 Matrix: Solid

Results reported on a "dry weight" basis and are adjusted for percent moisture, sample size and any dilutions.

Parameters	Results	Units	Report Limit	DF	Prepared	Analyzed	CAS No.	Qual
Percent Moisture	Analytical Method: ASTM D2974 Pace Analytical Services - Kansas City							
Percent Moisture	**30.1**	%	0.50	1		08/11/23 10:42		
2320B Alkalinity	Analytical Method: SM 2320B Preparation Method: SM 2320B Pace Analytical Services - Kansas City							
Alkalinity,Bicarbonate (CaCO3)	**295**	mg/kg	273	1	08/21/23 07:58	08/21/23 14:00		
Alkalinity,Carbonate (CaCO3)	ND	mg/kg	273	1	08/21/23 07:58	08/21/23 14:00		
Alkalinity, Total as CaCO3	**295**	mg/kg	273	1	08/21/23 07:58	08/21/23 14:00		
Oxidation/Reduction Potential	Analytical Method: SM 2580B Pace Analytical Services - Kansas City							
Oxidation/Reduction Potential	**256.1**	mV	1.0	1		08/14/23 15:39		

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QUALITY CONTROL DATA

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

QC Batch:	860144	Analysis Method:	ASTM D2974
QC Batch Method:	ASTM D2974	Analysis Description:	Dry Weight/Percent Moisture
		Laboratory:	Pace Analytical Services - Kansas City

Associated Lab Samples: 60435134001, 60435134002, 60435134003, 60435134004, 60435134005, 60435134006

METHOD BLANK: 3405992 Matrix: Solid

Associated Lab Samples: 60435134001, 60435134002, 60435134003, 60435134004, 60435134005, 60435134006

Parameter	Units	Blank Result	Reporting Limit	Analyzed	Qualifiers
Percent Moisture	%	ND	0.50	08/11/23 10:41	

SAMPLE DUPLICATE: 3405993

Parameter	Units	60435134001 Result	Dup Result	RPD	Max RPD	Qualifiers
Percent Moisture	%	32.9	33.1	1	20	

Results presented on this page are in the units indicated by the "Units" column except where an alternate unit is presented to the right of the result.

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QUALITY CONTROL DATA

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

QC Batch: 861257
QC Batch Method: SM 2320B
Analysis Method: SM 2320B
Analysis Description: 2320BS Analysis
Laboratory: Pace Analytical Services - Kansas City
Associated Lab Samples: 60435134001, 60435134002, 60435134003, 60435134004, 60435134005, 60435134006

METHOD BLANK: 3410620 Matrix: Solid
Associated Lab Samples: 60435134001, 60435134002, 60435134003, 60435134004, 60435134005, 60435134006

Parameter	Units	Blank Result	Reporting Limit	Analyzed	Qualifiers
Alkalinity, Total as CaCO3	mg/kg	ND	200	08/21/23 13:31	
Alkalinity,Bicarbonate (CaCO3)	mg/kg	ND	200	08/21/23 13:31	
Alkalinity,Carbonate (CaCO3)	mg/kg	ND	200	08/21/23 13:31	

LABORATORY CONTROL SAMPLE: 3410621

Parameter	Units	Spike Conc.	LCS Result	LCS % Rec	% Rec Limits	Qualifiers
Alkalinity, Total as CaCO3	mg/kg	5000	5030	101	90-110	

SAMPLE DUPLICATE: 3410622

Parameter	Units	60435134001 Result	Dup Result	RPD	Max RPD	Qualifiers
Alkalinity, Total as CaCO3	mg/kg	ND	ND		20	
Alkalinity,Bicarbonate (CaCO3)	mg/kg	ND	ND		20	
Alkalinity,Carbonate (CaCO3)	mg/kg	ND	ND		20	

Results presented on this page are in the units indicated by the "Units" column except where an alternate unit is presented to the right of the result.

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QUALITY CONTROL DATA

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

QC Batch: 860452
QC Batch Method: SM 2580B
Analysis Method: SM 2580B
Analysis Description: Oxidation/Reduction Potential
Laboratory: Pace Analytical Services - Kansas City
Associated Lab Samples: 60435134001, 60435134002, 60435134003, 60435134004, 60435134005, 60435134006

SAMPLE DUPLICATE: 3407377

Parameter	Units	60434923001 Result	Dup Result	RPD	Max RPD	Qualifiers
Oxidation/Reduction Potential	mV	201.6	201.6			

Results presented on this page are in the units indicated by the "Units" column except where an alternate unit is presented to the right of the result.

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QUALIFIERS

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

DEFINITIONS

DF - Dilution Factor, if reported, represents the factor applied to the reported data due to dilution of the sample aliquot.

ND - Not Detected at or above adjusted reporting limit.

TNTC - Too Numerous To Count

J - Estimated concentration above the adjusted method detection limit and below the adjusted reporting limit.

MDL - Adjusted Method Detection Limit.

PQL - Practical Quantitation Limit.

RL - Reporting Limit - The lowest concentration value that meets project requirements for quantitative data with known precision and bias for a specific analyte in a specific matrix.

S - Surrogate

1,2-Diphenylhydrazine decomposes to and cannot be separated from Azobenzene using Method 8270. The result for each analyte is a combined concentration.

Consistent with EPA guidelines, unrounded data are displayed and have been used to calculate % recovery and RPD values.

LCS(D) - Laboratory Control Sample (Duplicate)

MS(D) - Matrix Spike (Duplicate)

DUP - Sample Duplicate

RPD - Relative Percent Difference

NC - Not Calculable.

SG - Silica Gel - Clean-Up

U - Indicates the compound was analyzed for, but not detected.

N-Nitrosodiphenylamine decomposes and cannot be separated from Diphenylamine using Method 8270. The result reported for each analyte is a combined concentration.

Reported results are not rounded until the final step prior to reporting. Therefore, calculated parameters that are typically reported as "Total" may vary slightly from the sum of the reported component parameters.

Pace Analytical is TNI accredited. Contact your Pace PM for the current list of accredited analytes.

TNI - The NELAC Institute.

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QUALITY CONTROL DATA CROSS REFERENCE TABLE

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

Lab ID	Sample ID	QC Batch Method	QC Batch	Analytical Method	Analytical Batch
60435134001	**MMB-1**	ASTM D2974	860144		
60435134002	**MMB-2**	ASTM D2974	860144		
60435134003	**NMB-1**	ASTM D2974	860144		
60435134004	**NMB-2**	ASTM D2974	860144		
60435134005	**NMB-3**	ASTM D2974	860144		
60435134006	**NMB-4**	ASTM D2974	860144		
60435134001	**MMB-1**	SM 2320B	861257	SM 2320B	861388
60435134002	**MMB-2**	SM 2320B	861257	SM 2320B	861388
60435134003	**NMB-1**	SM 2320B	861257	SM 2320B	861388
60435134004	**NMB-2**	SM 2320B	861257	SM 2320B	861388
60435134005	**NMB-3**	SM 2320B	861257	SM 2320B	861388
60435134006	**NMB-4**	SM 2320B	861257	SM 2320B	861388
60435134001	**MMB-1**	SM 2580B	860452		
60435134002	**MMB-2**	SM 2580B	860452		
60435134003	**NMB-1**	SM 2580B	860452		
60435134004	**NMB-2**	SM 2580B	860452		
60435134005	**NMB-3**	SM 2580B	860452		
60435134006	**NMB-4**	SM 2580B	860452		

WO#:60435134

60435134

Pace ANALYTICAL SERVICES

DC#_Title: ENV-FRM-LENE-0009_Sample Condition Upon Receipt (SCUR)		
Revision: 2	Effective Date: 01/12/2022	Issued By: Lenexa

Client Name: Gerhart Cole

Courier: FedEx ☑ UPS ☐ VIA ☐ Clay ☐ PEX ☐ ECI ☐ Pace ☐ Xroads ☐ Client ☐ Other ☐

Tracking #: 7822 8933 2096 Pace Shipping Label Used? Yes ☐ No ☑

Custody Seal on Cooler/Box Present: Yes ☑ No ☐ Seals intact: Yes ☑ No ☐

Packing Material: Bubble Wrap ☑ Bubble Bags ☐ Foam ☐ None ☐ Other ☐

Thermometer Used: T289 **Type of Ice:** Wet Blue None

Cooler Temperature (°C): **As-read** 2.0 **Corr. Factor** +0.2 **Corrected** 2.2

Date and initials of person examining contents: AF 8/10

Temperature should be above freezing to 6°C

Chain of Custody present:	☑Yes	☐No	☐N/A	
Chain of Custody relinquished:	☑Yes	☐No	☐N/A	
Samples arrived within holding time:	☑Yes	☐No	☐N/A	
Short Hold Time analyses (<72hr):	☐Yes	☑No	☐N/A	
Rush Turn Around Time requested:	☐Yes	☑No	☐N/A	
Sufficient volume:	☑Yes	☐No	☐N/A	
Correct containers used:	☑Yes	☐No	☐N/A	
Pace containers used:	☐Yes	☑No	☐N/A	
Containers intact:	☑Yes	☐No	☐N/A	
Unpreserved 5035A / TX1005/1006 soils frozen in 48hrs?	☐Yes	☐No	☑N/A	
Filtered volume received for dissolved tests?	☐Yes	☐No	☑N/A	
Sample labels match COC: Date / time / ID / analyses	☑Yes	☐No	☐N/A	
Samples contain multiple phases? Matrix: SL	☐Yes	☑No	☐N/A	
Containers requiring pH preservation in compliance? (HNO_3, H_2SO_4, HCl<2; NaOH>9 Sulfide, NaOH>10 Cyanide) (Exceptions: VOA, Micro, O&G, KS TPH, OK-DRO) **LOT#:**	☐Yes	☐No	☑N/A	List sample IDs, volumes, lot #'s of preservative and the date/time added.
Cyanide water sample checks:				
Lead acetate strip turns dark? (Record only)	☐Yes	☐No		
Potassium iodide test strip turns blue/purple? (Preserve)	☐Yes	☐No		
Trip Blank present:	☐Yes	☐No	☑N/A	
Headspace in VOA vials (>6mm):	☐Yes	☐No	☑N/A	
Samples from USDA Regulated Area: State:	☐Yes	☑No	☐N/A	
Additional labels attached to 5035A / TX1005 vials in the field?	☐Yes	☐No	☑N/A	

Client Notification/ Resolution: Copy COC to Client? Y / N Field Data Required? Y / N

Person Contacted: ________ Date/Time: ________

Comments/ Resolution: ________

Project Manager Review: ________ Date: ________

Chain of Custody

GC Set Number 23-1681

Company Name:	Gerhart Cole
Project Name:	Mist Resiliency
Project Number:	23-1681
Company Address:	7657 South Holden Street, Midvale, Utah 84047
Contact Name:	Zach Gibbs
Phone Number:	801-791-3641
Email:	zachg@gerhartcole.com

GERHART COLE

7657 South Holden Street
Midvale, Utah 84047
(801) 849-0055

Relinquished By: (Signature)	Date	Time	Received By: (Signature)	Date	Time
[signature]	8/9/23	15:00	[signature]	8/9/23	15:00
Relinquished By: (Print)			**Received By: (Print)**		
Zach Gibbs / [signature] PISCCUT 8/9/23	8/9/23	17:00	[signature]	8/10	850

2.2

601739134

Sample Information				Test Description and Information							
Test Hole / Test Pit #	Depth Range (ft)	Sample Type	Sample Idendificatio	Redox	Carbonates	Bicarbonates					Special instructions / comments
MMB-1	6.5-8.5	SH	-	1	1	1					Need carbonates and bicarbonates results separate on all samples
MMB-2	11.5-13.5	SH	-	1	1	1					
NMB-1	5-20	SPT/D&M	-	1	1	1					
NMB-2	5-14	SPT/D&M	-	1	1	1					
NMB-3	5-14	SPT/D&M	-	1	1	1					
NMB-4	5-12.5	SPT/D&M	-	1	1	1					

Client: Gerhart Cole

Profile # 15655-1

Site: Mist Resiliency

Notes

COC Line Item	Matrix	VG9H	DG9H	DG9Q	VG9U	DG9U	DG9M	DG9B	BG1U	AG1H	AG1U	AG2U	AG3S	AG4U	AG5U	JGFU	WGKU	WGDU	BP1U	BP2U	BP3U	BP1N	BP3N	BP3F	BP3S	BP3C	BP3Z	WPDU	ZPLC	Other		
1	SL																												1			
2																													1			
3																													1			
4																													1			
5																													1			
6																													1			
7																																
8																																
9																																
10																																
11																																
12																																

Container Codes

Glass			
DG9B	40mL bisulfate clear vial	WGKU	8oz clear soil jar
DG9H	40mL HCl amber voa vial	WGFU	4oz clear soil jar
DG9M	40mL MeOH clear vial	WG2U	2oz clear soil jar
DG9Q	40mL TSP amber vial	JGFU	4oz unpreserved amber wide
DG9S	40mL H2SO4 amber vial	AG0U	100mL unores amber glass
DG9T	40mL Na Thio amber vial	AG1H	1L HCl amber glass
DG9U	40mL amber unpreserved	AG1S	1L H2SO4 amber glass
VG9H	40mL HCl clear vial	AG1T	1L Na Thiosulfate clear/amber glass
VG9T	40mL Na Thio. clear vial	AG1U	1liter unpres amber glass
VG9U	40mL unpreserved clear vial	AG2N	500mL HNO3 amber glass
BG1S	1liter H2SO4 clear glass	AG2S	500mL H2SO4 amber glass
BG1U	1liter unpres glass	AG3S	250mL H2SO4 amber glass
BG3H	250mL HCL Clear glass	AG2U	500mL unpres amber glass
BG3U	250mL Unpres Clear glass	AG3U	250mL unpres amber glass
WGDU	16oz clear soil jar	AG4U	125mL unpres amber glass
		AG5U	100mL unpres amber glass

Plastic	
BP1C	1L NAOH plastic
BP1N	1L HNO3 plastic
BP1S	1L H2SO4 plastic
BP1U	1L unpreserved plastic
BP1Z	1L NaOH, Zn Acetate
BP2C	500mL NAOH plastic
BP2N	500mL HNO3 plastic
BP2S	500mL H2SO4 plastic
BP2U	500mL unpreserved plastic
BP2Z	500mL NaOH, Zn Acetate
BP3C	250mL NaOH plastic
BP3F	250mL HNO3 plastic - field filtered
BP3N	250mL HNO3 plastic
BP3U	250mL unpreserved plastic
BP3S	250mL H2SO4 plastic
BP3Z	250mL NaOH, Zn Acetate
BP4U	125mL unpreserved plastic
BP4N	125mL HNO3 plastic
BP4S	125mL H2SO4 plastic
WPDU	16oz unpresserved plstic

Misc.	
I	Wipe/Swab
SP5T	120mL Coliform Na Thiosulfate
ZPLC	Ziploc Bag
AF	Air Filter
C	Air Cassettes
R	Terracore Kit
U	Summa Can

Matrix	
WT	Water
SL	Solid
NAL	Non-aqueous Liquid
OL	OIL
WP	Wipe
DW	Drinking Water

Work Order Number: 60435134

APPENDIX B

Downhole Seismic Testing Results and Field Resistivity Testing Results

Report on

Downhole Seismic Explorations
Northwest Natural Gas North Mist Compressor Station

Mist, Oregon

Report Date: July 24, 2023

Prepared for:

GeoEngineers, Inc
4000 Kruse Way Place
Lake Oswego, OR 97035

Prepared by:

EARTH DYNAMICS LLC
2284 N.W. Thurman St.
Portland, OR 97210
(503) 227-7659
Project No. 23215

1.0 INTRODUCTION

This report presents the results of downhole seismic explorations at the Northwest Natural Gas North Mist Compressor station near Mist, Oregon. This work was requested and authorized by Jerad Hoffman of GeoEngineers. The exploration consisted of downhole measurements of compressional and shear wave velocity at three locations.

The geophysical data were acquired on June 20 and 21, 2023 under the supervision of Mr. Daniel Lauer of Earth Dynamics LLC. This report describes the methodology and results of the geophysical investigation.

2.0 METHOD

2.1 Downhole Seismic

In a downhole seismic survey, a seismic source is placed on the ground surface near a borehole, and two geophone assemblies are placed at selected depths in the borehole. The test boring is prepared by installing three-inch I.D. flush-joint PVC casing. The PVC casing is grouted in place to provide the required seismic coupling between the casing and the surrounding formation. The raw data obtained from a downhole survey are the travel times for compressional (P-Wave) and shear (S-Wave) waves from the source to the geophones and the distance between the source and geophones. All depths are measured from the top of the casing.

Seismic waves with a large compressional wave component are produced by striking a steel plate on the ground with a sledge hammer. The plate is located five feet from the boring. Shear-waves travel slower than compressional waves. Therefore, shear wave signals are often obscured by the compressional wave signal. This interference sometimes makes identification of the first shear-wave arrival difficult. To improve the resolution of the shear-wave arrival, the seismic source is designed to produce a signal that contains a large shear-wave component while minimizing the compressional wave signal. A signal enhancement seismograph is used to process the signals received from the geophones. The shear-wave source for this study consisted of sledge hammer impacts on alternate ends of an 8"x10"x8' wooden beam with aluminum end plates. The beam was coupled to the ground by weighing it down with the front tires of the field recording truck. The beam was offset a distance of five feet from the borehole to minimize direct coupling of the seismic energy to the casing.

The downhole sensors consisted of two Bison Instruments Type 1462 triaxial geophone assemblies. Each assembly contains three sensing elements: one vertical and two orthogonal horizontal geophones. A distance of ten feet separated the two geophone assemblies within the borehole. Two geophone assemblies at a fixed separation are used so that interval velocities can be determined from the same set of impulses. The use of two geophone assemblies provides at least two compressional and two shear-wave travel times at each measurement level. This method reduces timing errors caused by differences in seismic triggering and variations in source impulse characteristics. In this study, travel time values are accurate to ±1 millisecond (ms).

The recording procedure consists of placing the geophone assemblies at the desired depths in the borehole. The geophone assemblies are locked to the inside of the casing wall by inflating a pneumatic rubber packer. A Seismic Source, Inc. DAQ Link IV signal enhancement seismograph and laptop computer are used to record signals from the geophones. The travel times are determined in the field and the data are checked for consistency before proceeding to the next measurement depth.

The data are analyzed by plotting the overall travel time versus distance. These plots are commonly referred to as travel-time plots. Linear regression analysis is used to compute line segments joining data points of similar slope. The slope of the line segment is proportional to the average velocity of the material within the depth corresponding to the line segment.

Downhole shear wave data were acquired in three borings. The borings are identified as MMB-1, NMB-1 and NMB-3. According to the drilling records, NMB-1 and NMB-3 were cased to an approximate depth of 100 feet below the ground surface (bgs) and MMB-1 was cased to an approximate depth of 80 feet bgs. Velocity measurements were obtained every five feet from the ground surface to bottom of the casing in each boring. The bottom of MMB-1 was encountered at a depth of 78 feet bgs, the bottom of NMB-1 was encountered at a depth of 98 feet bgs, and the bottom of NMB-3 was encountered at a depth of 100 feet bgs.

3.0 RESULTS

The results of the downhole seismic study are contained in Figures 3-1 through 3-3. P-wave and S-wave travel-times from the downhole test at Borings MMB-1, NMB-1, and NMB-3 are plotted in Figures 3-1, 3-2, and 3-3 respectively. Linear regression of the travel-time data in the figures are used to determine the average P and S-wave velocities of the subsurface material. The velocities are corrected to compensate for the offset of the seismic source from the borehole. Table 3-1 summarizes seismic velocity as a function of depth for the downhole measurements.

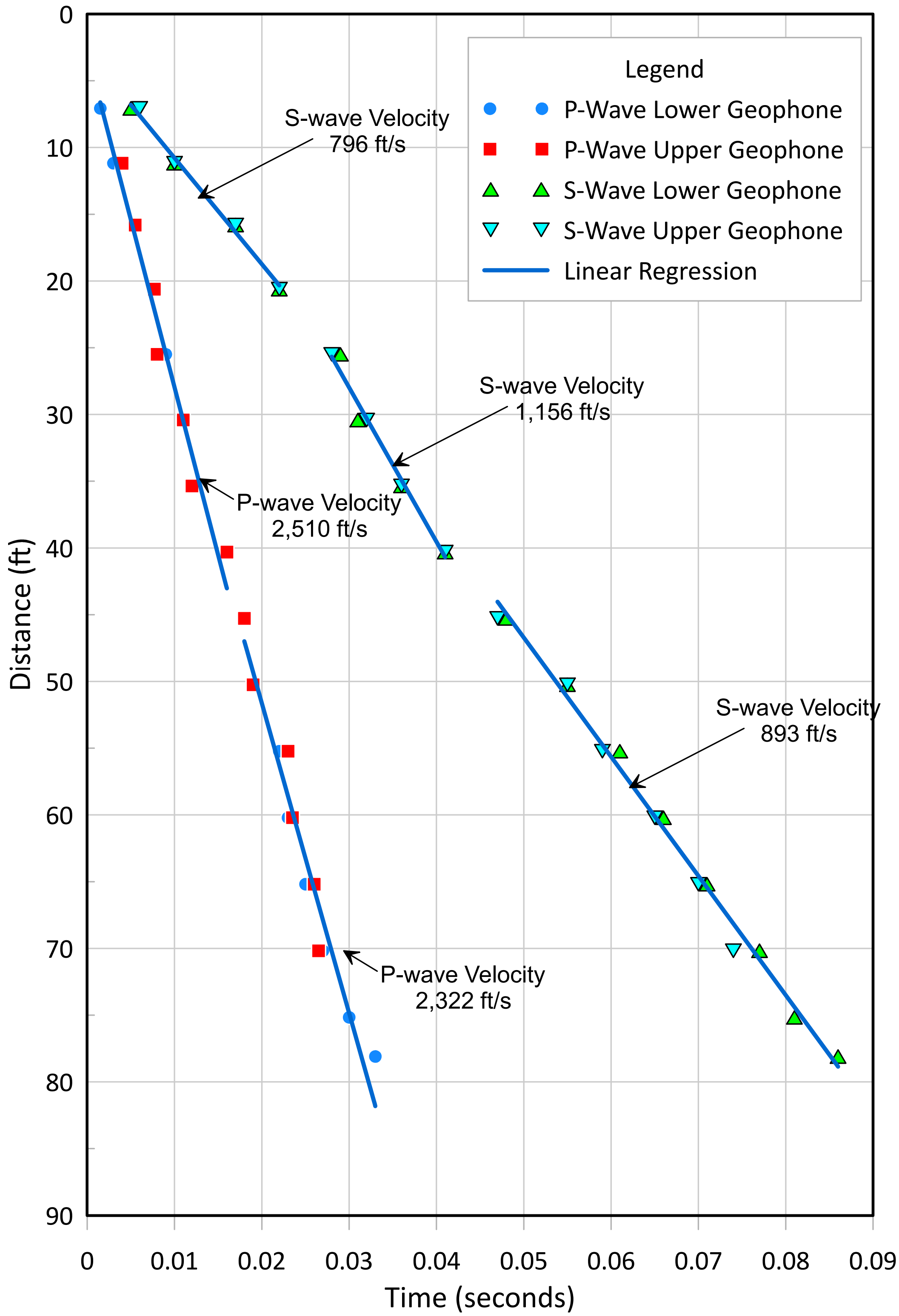

Figure 3-1. Seismic Velocity Travel Time Plot for Boring MMB-1.

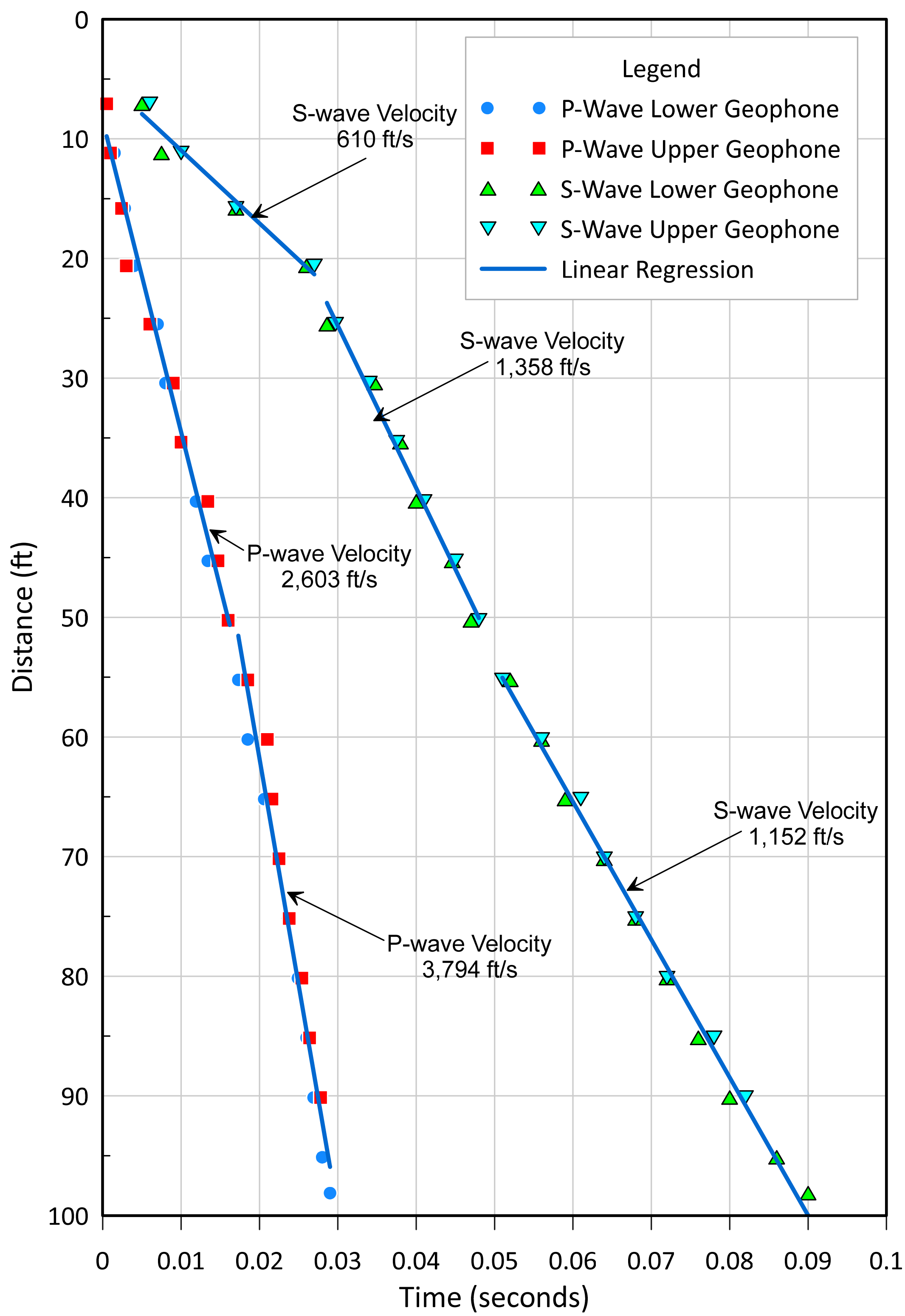

Figure 3-2. Seismic Velocity Travel Time Plot for Boring NMB-1.

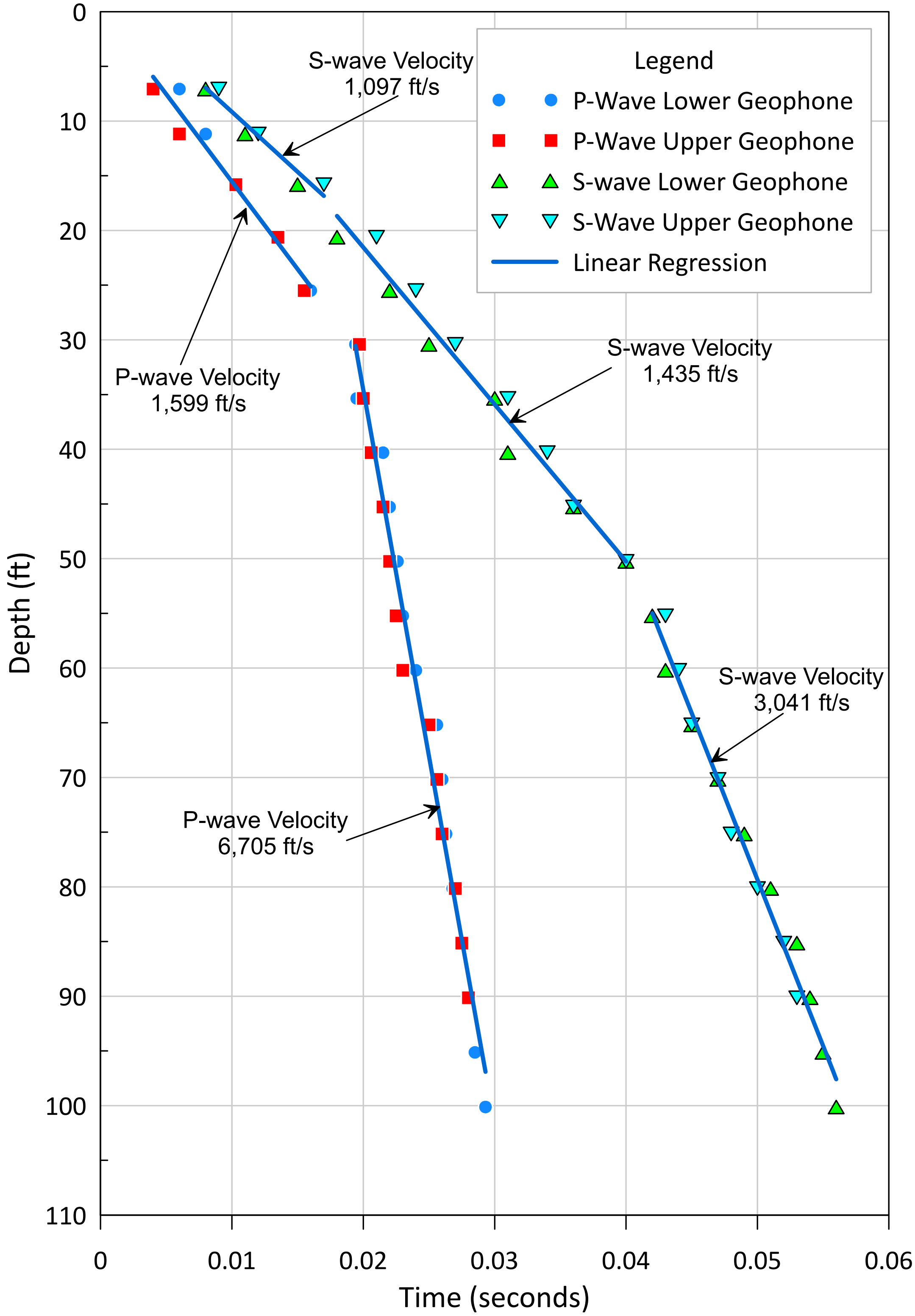

Figure 3-3. Seismic Velocity Travel Time Plot for Boring NMB-3.

Table 3-1 Summary of Seismic Velocity from Downhole Measurements.

Boring	Depth Range (ft)	P-Wave Velocity (ft/s)	Depth Range (ft)	S-Wave Velocity (ft/s)
MMB-1	0 - 45 45 - 78	2,510 2,322	0 - 25 25 - 45 45 - 78	796 1,156 893
NMB-1	0 - 50 50 - 98	2,603 3,794	0 - 20 20 - 50 50 – 98	610 1,358 1,152
NMB-3	0 – 30 30 – 100	1,599 6,705	0 – 15 15 – 50 50 – 100	1,097 1,435 3,041

4.0 DISCUSSION

4.1 Boring MMB-1

Boring MMB-1 is located in the middle of an active compressor station (Miller Station). Vibrations from nearby equipment such as motors and fans as well as active construction work made picking of the downhole first arrivals for both the P-waves and S-waves difficult.

Data from MMB-1 draft boring log indicate that the boring encountered the following subsurface material:

- 0 – 5 feet bgs: Poorly graded Fill
- 5 – 52 feet bgs: Fat clay with increasing stiffness with depth
- 52 – 68 feet bgs: Medium dense Silty Sand.
- 68 – 80 feet bgs: Very stiff Sandy Silt.

It appears that there is moderately good correlation between the downhole seismic velocity data and the boring logs for Boring MMB-1.

4.2 Boring NMB-1

Boring MMB-1 is located approximately 200 feet north of an active compressor station (North Mist Station). Vibrations from nearby equipment such as motors and fans are much lower than that of the Miller Station boring. Picking of the downhole first arrivals for both the P-waves and S-waves is moderately difficult.

Data from NMB-1 draft boring log indicate that the boring encountered the following subsurface material:

- 0 – 5 feet bgs: Organic Fill
- 5 – 19 feet bgs: Stiff Silt with Sand
- 19 – 33 feet bgs: Stiff Silt
- 33 – 47 feet bgs: Weathered Columbia River Basalt
- 47 – 53 feet bgs: Stiff Silt
- 53 – 100 feet bgs: Medium Dense Sand with Silt

It appears that there is moderately good correlation between the downhole seismic velocity data and the boring logs for Boring NMB-1.

4.2 Boring NMB-3

Boring MMB-1 is located approximately 200 feetnorth of an active compressor station (North Mist Station). Vibrations from nearby equipment such as motors and fans are much lower than that of the Miller Station boring. Picking of the downhole first arrivals for both the P-waves and S-waves is moderately difficult.

Data from NMB-3 draft boring log indicate that the boring encountered the following subsurface material:

- 0 – 9 feet bgs: Organic Fill
- 9 – 24 feet bgs: Medium Dense to Dense Silty Sand
- 24 – 43 feet bgs: Weathered Columbia River Basalt
- 43 – 100 feet bgs: Columbia River Basalt

It appears that there is moderately good correlation between the downhole seismic velocity data and the boring logs for Boring NMB-3.

4.0 <u>LIMITATIONS</u>

We have presented models and interpretations which we believe to be the best fit given the geology and known conditions at the site. However, no warranty is made or intended by this report or by oral or written presentation of this work. Earth Dynamics accepts no responsibility for damages because of decisions made or actions taken based upon this report.

RESPECTFULLY SUBMITTED
EARTH DYNAMICS LLC

Daniel Lauer, M.S.
Partner

Report on

Electrical Resistivity Testing
Northwest Natural Gas North Mist Compressor Station
Mist, Oregon

Test Date: June 21, 2023
Report Date: July 24, 2023

Prepared for:

GeoEngineers, Inc
4000 Kruse Way Place
Lake Oswego, OR 97035

Prepared by:

EARTH DYNAMICS LLC
2284 N.W. Thurman St.
Portland, OR 97210
(503) 227-7659

Project No. 23215

1.0 INTRODUCTION

This report presents the results of an electrical resistivity exploration at the Northwest Natural Gas North Mist Compressor station near Mist, Oregon. This work was requested and authorized by Jerad Hoffman of GeoEngineers. The exploration consisted of Wenner 4-pin electrical resistivity measurements at one location.

The geophysical data were acquired on June 21, 2023 under the supervision of Mr. Daniel Lauer of Earth Dynamics LLC. This report describes the methodology and results of the geophysical investigation.

2.0 METHOD and INSTRUMENTATION

2.1 Electrical Resistivity

Electrical resistivity sounding measurements were obtained in accordance with *ASTM G 57 –95a, Standard Test Method for Field Measurement of Soil Resistivity Using the Wenner Four-Electrode Method*. An Advanced Geosciences, Inc. MiniSting Earth Resistivity Meter equipped with a four-pin Wenner electrode array was used to acquire the resistivity data. For each measurement, the instrument applies a current (I), reverses polarity and applies the current again and then reverses polarity back to the original and applies current again. The reversed polarity technique is used to reduce electrode polarization. The voltage (V) at the potential electrodes is measured for each current injection, and the values are averaged. The average resistance (V/I), resistivity and standard deviation are displayed on a screen and stored in the internal memory. The memory also stores the date and time of the measurement, and the electrode configuration. The system does not require scale multipliers that are common on older analog resistivity meters. Voltage divided by current (resistance) rather than voltage is recorded by the instrument because resistance is the term that is commonly used in apparent resistivity equations. The MiniSting calibration was verified prior to data acquisition with a test resistor.

Data were acquired using two perpendicular linear Wenner arrays with a common midpoint to evaluate the lateral homogeneity of the subsurface material. Data were acquired using A spacings of 1, 2, 3, 4, 5, 6, 8, 10, 20, 30, 40, and 60 feet in the north-south direction and 1, 2, 3, 4, 5, 6, 8, 10, 20, 30, 40, 60, 80 and 100 feet in the east/west direction.

The ground surface, at the time of the test, consisted of damp fill comprised of soil and wood chips. Stainless steel electrodes (3/8’ diameter) were pounded into the ground surface with a hammer. The electrodes are connected to the MiniSting with 16-gauge stranded wire. During data acquisition the weather was overcast and calm with an air temperature of approximately 70 °F and a soil temperature (at 4” deep) of approximately 65 °F.

3.0 RESULTS

The center point of the Wenner array testing is located at approximately Latitude: 46º 2.870’N, Longitude: 123º 17.911W. Data were acquired in the largest possible A-spacing in each direction. The resistivity data acquired along the north/south and east/west arrays are listed in Tables 3-1 and 3-2 respectively. The data are plotted in Figure 3-1. The correlation between data from the two arrays indicates that the electrical properties of the subsurface materials are laterally homogeneous over the test area.

The two current electrodes for the North-South A-spacing of 100’ were within 3’ feet of the metal perimeter fence at the site. The Apparent resistivity for this measurement is lower than the other measurements. It is possible that the proximity to the fence influenced the reading for this A-spacing.

Table 3-1. North/South array Wenner Data.

A Spacing (ft)	Current (mA)	Resistance Ω	Apparent Resistivity Ω-ft	Standard Deviation (%)
1	50	60.40	379.5	0.1
2	20	34.43	432.7	0.1
3	100	20.06	378.2	0.1
4	50	16.18	406.6	0.1
5	50	13.34	419.0	0.1
6	50	11.34	427.5	0.1
8	50	8.048	404.6	0.1
10	50	5.754	361.5	0.1
20	50	2.765	347.5	0.1
30	100	1.915	361.0	0.1
40	100	1.419	356.5	0.1
60	50	0.738	278.3	0.2

Table 3-2. East/West array Wenner Data.

A Spacing (ft)	Current (mA)	Resistance Ω	Apparent Resistivity Ω-ft	Standard Deviation (%)
1	50	58.70	368.8	0.1
2	50	32.17	404.2	0.1
3	100	23.59	444.7	0.1
4	50	18.94	476.1	0.1
5	100	14.68	461.3	0.1
6	50	12.18	459.1	0.1
8	50	8.536	429.1	0.1
10	50	6.208	390.1	0.1
20	50	2.775	348.7	0.1
30	100	1.854	349.5	0.1
40	100	13.93	250.2	0.1
60	50	0.784	295.4	0.1
80	50	0.418	210.3	0.2
100*	100	0.057	35.53	0.5

* The two current electrodes were very close to the metal perimeter fence for A spacing of 100'

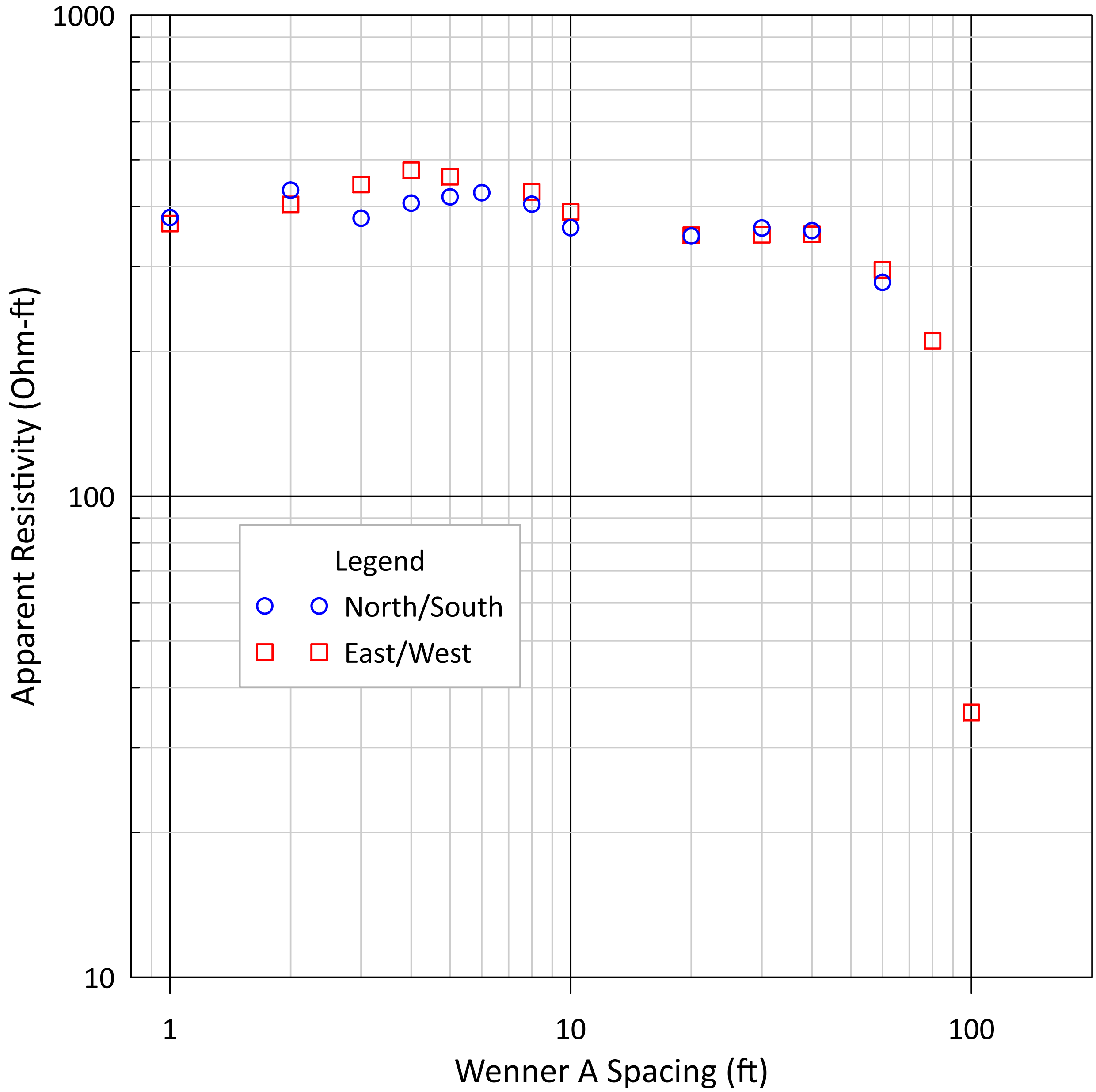

Figure 3-2. Wenner Data.

4.0 LIMITATIONS

We have presented data and interpretations that we believe to be accurate within the limitations of the test methods. However, no warranty is made or intended by this report or by oral or written presentation of this work. Earth Dynamics accepts no responsibility for damages as a result of decisions made or actions taken based upon this report.

RESPECTFULLY SUBMITTED:
EARTH DYNAMICS

Daniel Lauer
Senior Geophysicist

APPENDIX C
Report Limitations and Guidelines for Use

APPENDIX C
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This attachment provides information to help you manage your risks with respect to the use of this report.

Geotechnical Services Are Performed For Specific Purposes, Persons and Projects

This report has been prepared for the exclusive use by NW Natural, and their authorized agents. This report is not intended for use by others, and the information contained herein is not applicable to other sites.

GeoEngineers structures our services to meet the specific needs of our clients. For example, a geotechnical or geologic study conducted for a civil engineer or architect may not fulfill the needs of a construction contractor or even another civil engineer or architect that are involved in the same project. Because each geotechnical or geologic study is unique, each geotechnical engineering or geologic report is unique, prepared solely for the specific client and project site. Our report is prepared for the exclusive use of our Client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing. This is to provide our firm with reasonable protection against open-ended liability claims by third parties with whom there would otherwise be no contractual limits to their actions. Within the limitations of scope, schedule and budget, our services have been executed in accordance with our Agreement with the Client and generally accepted geotechnical practices in this area at the time this report was prepared. This report should not be applied for any purpose or project except the one originally contemplated.

A Geotechnical Engineering or Geologic Report Is Based on a Unique Set of Project-Specific Factors

This report has been prepared for NW Natural for the proposed North Mist Compressor Station Resiliency project as part of NW Natural's Mist Resiliency project. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, do not rely on this report if it was:

- Not prepared for you,
- Not prepared for your project,
- Not prepared for the specific site explored, or
- Completed before important project changes were made.

For example, changes that can affect the applicability of this report include those that affect:

- The function of the proposed structure;
- Elevation, configuration, location, orientation or weight of the proposed structure;

[1] Developed based on material provided by ASFE, Professional Firms Practicing in the Geosciences; www.asfe.org.

- Composition of the design team; or
- Project ownership.

If important changes are made after the date of this report, GeoEngineers should be given the opportunity to review our interpretations and recommendations and provide written modifications or confirmation, as appropriate.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by manmade events such as construction on or adjacent to the site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. Always contact GeoEngineers before applying a report to determine if it remains applicable.

Most Geotechnical and Geologic Findings Are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied our professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

Geotechnical Engineering Report Recommendations Are Not Final

Do not over-rely on the preliminary construction recommendations included in this report. These recommendations are not final, because they were developed principally from GeoEngineers' professional judgment and opinion. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for this report's recommendations if we do not perform construction observation.

Sufficient monitoring, testing and consultation by GeoEngineers should be provided during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions.

A Geotechnical Engineering or Geologic Report Could Be Subject To Misinterpretation

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having GeoEngineers confer with appropriate members of the design team after submitting the report. Also retain GeoEngineers to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having GeoEngineers participate in pre-bid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

Give Contractors a Complete Report and Guidance

Some owners and design professionals believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering or geologic report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer. A pre-bid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might an owner be in a position to give contractors the best information available, while requiring them to at least share the financial responsibilities stemming from unanticipated conditions. Further, a contingency for unanticipated conditions should be included in your project budget and schedule.

Contractors Are Responsible For Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and to adjacent properties.

Read These Provisions Closely

Some clients, design professionals and contractors may not recognize that the geoscience practices (geotechnical engineering or geology) are far less exact than other engineering and natural science disciplines. This lack of understanding can create unrealistic expectations that could lead to disappointments, claims and disputes. GeoEngineers includes these explanatory "limitations" provisions in our reports to help reduce such risks. Please confer with GeoEngineers if you are unclear how these "Report Limitations and Guidelines for Use" apply to your project or site.

Geotechnical, Geologic and Environmental Reports Should Not Be Interchanged

The equipment, techniques and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical or geologic study and vice versa. For that reason, a geotechnical engineering or geologic report does not usually relate any environmental findings, conclusions or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding a specific project.

Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings, or conclusions regarding the detecting, assessing, preventing or abating of

Biological Pollutants and no conclusions or inferences should be drawn regarding Biological Pollutants, as they may relate to this project. The term “Biological Pollutants” includes, but is not limited to, molds, fungi, spores, bacteria, and viruses, and/or any of their byproducts.

If Client desires these specialized services, they should be obtained from a consultant who offers services in this specialized field.

Attachment H-4.
Miller Station Resiliency Geotechnical Report

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Geotechnical Engineering Report

Mist Resiliency Project
Miller Station Resiliency Area
Columbia County, Oregon

for
NW Natural

October 20, 2023

GeoEngineers
Earth Science + Technology

Geotechnical Engineering Report

Mist Resiliency Project
Miller Station Resiliency Area
Columbia County, Oregon

for
NW Natural

October 20, 2023

GeoEngineers

4000 Kruse Way Place
Building 3, Suite 200
Lake Oswego, Oregon
503.624.9274

Geotechnical Engineering Report

Mist Resiliency Project
Miller Station Resiliency Area
Columbia County, Oregon

File No. 6024-308-00

October 20, 2023

Prepared for:

NW Natural
250 SW Taylor Street
Portland, Oregon 97204

Attention: Andrew Bauer, RG

Prepared by:

GeoEngineers, Inc.
4000 Kruse Way Place
Building 3, Suite 200
Lake Oswego, Oregon
503.624.9274

Jerad A. Hoffman, PE
Project Engineer

Brian C. Ranney, CEG
Associate Engineering Geologist

Lyle J. Stone, PE
Associate Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
100288PE
OREGON
MAY 9, 2023
LYLE JOSEPH STONE
EXPIRES: 06/30/2024

JAH:BCR:LJS:leh:cdb

GeoEngineers

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APPENDICES

1.0 INTRODUCTION

This report presents the results of our geotechnical engineering services for the proposed Miller Station Resiliency Area which is part of the Mist Resiliency Project in Columbia County, Oregon. The site is shown relative to surrounding area in the Vicinity Map, Figure 1.

The existing Miller Station facility is a roughly 8-acre complex that includes a variety of single-story metal-frame buildings, natural gas handling and compression equipment, and associated under- and above-ground utilities including gas and water pipelines and electrical conduits. The ground surface is mantled with crushed rock fill. NW Natural is planning to replace two existing reciprocating compressors; the replacement compressors are centrifugal compressors driven by gas turbines estimated to weigh 120 kips each. We understand that the new compressors will be placed on or near our site-specific borings as shown in the Site Plan, Figure 2.

GeoEngineers, Inc. (GeoEngineers) has completed numerous studies in the project vicinity. In completing this report, we considered data included in GeoEngineers' Geotechnical Engineering Report, Miller Station – Proposed Mist Control Building Addition; Columbia County, Oregon (GeoEngineers 2017) in addition to the explorations completed as part of this project.

2.0 SCOPE OF SERVICES

Our services were conducted in general accordance with our proposal dated May 5, 2023 and Change Order No. 1 dated May 26, 2023 authorized by NW Natural with Purchase Order No. 4510006216. The purpose of our geotechnical engineering services was to explore the subsurface soil and groundwater conditions and provide geotechnical engineering recommendations for designing and constructing the compressor stations. Our specific scope of services for this task included the following:

1. Visited the site to mark proposed boring locations.
2. Notified the public "one-call" utility center to mark existing utilities near our proposed boring locations.
3. Subcontracted a private utility locator to locate utilities near our proposed boring locations.
4. Explored subsurface conditions by drilling two borings to depths of 40 feet and 80 feet below ground surface (bgs) on the selected locations. The borings were drilled using mud rotary equipment on a track-mounted drilling rig. A 3-inch diameter polyvinyl chloride (PVC) casing was installed and grouted in place in the 80-foot boring for subsequent downhole seismic testing. While observing the borings we:
    a. Utilized a vacuum trailer to excavate to depths of 5 to 8 feet in the proposed boring locations to clear potential unmarked shallow utilities.
    b. Completed in-situ sampling during standard penetration testing (SPT) using standard 1.5-inch samplers (SPT samplers) 2.5-foot intervals to 15 feet and 5-foot intervals thereafter. We obtained relatively undisturbed Shelby tube samples on cohesive soils at representative intervals.
    c. Classified the materials encountered in the borings in general accordance with ASTM International (ASTM) Standard Practices Test Method D 2488.

   d. Maintained a detailed log of each exploration.

5. Performed laboratory tests on selected samples to determine index, strength or compressibility properties as necessary. The testing included:
   a. Six moisture contents in accordance with ASTM Test Method D 2937 for site soil characterization and interpretation, and to evaluate the fill suitability of the existing soils.
   b. Three Atterberg limits tests in general accordance with ASTM D4318.
   c. Six percent fines determinations in general accordance with ASTM D1140.
6. Subcontracted laboratory testing for corrosion potential including: pH (U.S. Environmental Protection Agency [EPA] 9045D), Soluble Sulfates (EPA 300.0), Chloride Ion (EPA 300.0), Electrical Resistivity (American Association of State Highway and Transportation Officials [AASHTO] T 288), Redox Potential (Standard Method [SM] 2580B) and Sulfide (EPA 6010D).
7. Subcontracted a geophysical subconsultant to complete downhole seismic testing on a separate mobilization from drilling. GeoEngineers provided field staff to observe the downhole seismic testing.
8. Evaluated the collected data to determine the site's suitability for the proposed construction, including foundation support requirements.
9. Provided grading recommendations, including stripping depths, unsuitable soil removal, fill type for imported materials, maximum lift thicknesses compaction criteria, cut and fill slope criteria, procedures for use of on-site soils, and wet weather earthwork procedures.
10. Provided excavation recommendations, including temporary and final slope inclinations and trench excavation and backfill compaction.
11. Provided foundation recommendations for proposed structures and appurtenant facilities, including the proposed compressors and associated lightly loaded building as required. Design recommendations address preferred foundation type (mat foundations, deep foundations, ground improvement, etc.), allowable bearing pressure, overturning resistance, minimum footing dimensions and embedment, and settlement behavior.
12. Provided lateral resistance recommendations, including friction coefficient and passive earth pressures.
13. Evaluated site seismic hazards and recommended the appropriate zone factor and site coefficients for seismic design using conventional equivalent static lateral force methods, as well as recommendations to address seismic hazards identified at the site.
14. Provided this report summarizing our findings and providing geotechnical design recommendations stamped by a licensed professional engineer in Oregon.

3.0 SITE CONDITIONS

3.1. Surface Conditions

The subject site consists of the existing Miller Station facility and associated compressor equipment. The ground surface generally slopes upward to the northeast. Existing buildings and equipment are positioned on flat benches oriented northwest to southeast along the slope. Based on the observed topography the benches appear to be constructed of fill. Gravel driveways lead up the slope and along the benches between the existing buildings and equipment. Elevations at the site range between approximately 1,050 feet above mean sea level (MSL) and 1,030 feet MSL. The approximate locations of the existing site features are shown in Figure 2.

3.2. Subsurface Conditions

3.2.1. Site Geology

The Oregon Department of Geology and Mineral Industries (DOGAMI 2015) shows the site underlain by Oligocene-aged siltstone bedrock of the Pittsburg Bluff Formation. The Pittsburg Bluff rocks are typically tuffaceous and arkosic sandstones, locally glauconitic and fossiliferous, with lesser tuffaceous siltstone, claystone, and coal. They were deposited in marine to deltaic waters that appear to have been becoming shallower with time; ultimately, the area rose above sea level, and there is an erosional unconformity between the top of the Pittsburg Bluff Formation and overlying strata.

Although not described in the published mapping, our experience in the Coast Range indicates that the sedimentary bedrock is typically deeply weathered to decomposed to depths extending up to tens of feet. The residual soils formed by decomposition range from silt and clay to sand and occasional clayey or silty gravel.

3.2.2. Subsurface Explorations

We completed two borings (MM B-1 and MM B-2) between June 6 and 8, 2023 to depths of 80 and 40 feet bgs, respectively. A 3-inch diameter PVC casing was installed and grouted in place in the 80-foot boring for subsequent in-situ downhole seismic testing. The approximate locations of the borings are shown in Figure 2. The borings were completed with a track-mounted drill rig using mud rotary drilling methods. Details of the subsurface exploration program and the logs of the explorations are presented in Appendix A, Field Explorations and Laboratory Testing.

3.2.3. Laboratory Testing

Soil samples obtained from the borings were transported to our Portland, Oregon geotechnical laboratory and tested to confirm or modify field classifications, as well as to evaluate engineering properties of the soils we encountered. Representative samples were selected for the following geotechnical index tests:

- Moisture content,
- Atterberg limits, and
- Percent passing the U.S. No. 200 sieve

Soil samples from the borings obtained in native soils at relatively shallow depths below the existing gravel fill were sent to a subcontracted laboratory for corrosion potential testing, including:

- Sulfate Ion Content
- Sulfite Ion Content
- Chloride Ion Content
- Redox Potential

- pH
- Carbonates and Bicarbonates
- Saturated Soil Box Resistivity

Appendix A includes a brief discussion of laboratory test methods and results of the laboratory testing.

3.2.4. Subsurface Soil Conditions

In general, subsurface conditions encountered by our borings consist of dense poorly graded gravel fill soils overlying highly weathered sedimentary rock of the Pittsburg Bluff Formation. The weathered Pittsburg Bluff Formation is represented as loose to very dense silty sand or soft to very stiff clay and elastic silt. As such, we characterized the soil at our boring locations into two general units, including: (1) fill, and (2) Pittsburg Bluff Formation. The following paragraphs provide a description of the soil units encountered in our explorations.

Boring B-1 encountered approximately 5 feet of dense poorly graded gravel fill overlying weathered Pittsburg Bluff Formation materials consisting of soft fat clay and elastic silt that became stiff to very stiff with depth. However, B-1 encountered medium dense to very dense silty sand between 53 feet and 68 feet bgs.

Boring B-2 encountered approximately 8 feet of dense poorly graded gravel fill overlying weathered Pittsburg Bluff Formation materials consisting of loose silty sand that became medium dense with depth. A more detailed description of subsurface conditions is provided in boring logs in Appendix A.

3.2.5. Groundwater Conditions

A well log obtained from the Oregon Water Resources Department for a water well drilled at Miller Station compressor station indicated a static groundwater level of 188 feet beneath the surface (ORWD 2023). Groundwater could not be observed in our borings because of the presence of drilling fluid during drilling. However, free water was observed in a clayey sand sample obtained at 25 feet bgs (1,032 feet MSL) in a boring previously completed by GeoEngineers in January 2017 for the Mist Control Building Addition (GeoEngineers 2017) suggesting that perched groundwater or seepage paths may be present near that elevation or within similar subsurface materials. Groundwater conditions at the site are expected to vary seasonally due to rainfall events and other factors not observed in our explorations.

4.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our subsurface explorations and analyses, it is our opinion that the site can provide suitable support for the proposed replacement centrifugal compressors, provided the recommendations in this report are incorporated into the project design and are implemented during construction. We offer the following conclusions regarding geotechnical engineering design at the site.

- Near surface soils consist of gravel fill with variable silt and gravel content. On-site fills soils may be considered for use as structural fill provided they do not contain a significant amount of fines and can be moisture conditioned and compacted as recommended.

- Native subgrade soils will likely be wet and have a high risk of becoming disturbed from earthwork. Equipment should be confined to gravel pads and haul roads and not be allowed to traverse wet subgrade soils.
- Buried foundations, structural features, and conduits/pipes should be removed from building areas and backfilled with structural fill.
- Overexcavation of soft/loose subgrade soils and placement of approximately 30 inches of structural fill will be required to support shallow foundations. Lightly loaded structures can be supported on spread footings bearing on structural fill or aggregate base material placed over soft clay soils and loose silty sand. Existing granular fill can be included in the minimum fill thickness, provided the depth and consistency of the existing fill is confirmed in the field by GeoEngineers during construction.
- Shallow foundations supported by subgrades prepared as recommended in this report should be adequate to support the proposed equipment at the site. GeoEngineers can provide specific recommendations for pier foundations, if required, and after the preliminary configuration of pier foundations is determined by the structural engineer.

5.0 EARTHWORK RECOMMENDATIONS

5.1. Site Preparation

Initial site preparation and earthwork operations will include grading the site and excavating for utilities and foundations. We recommend that existing buried structures, foundations, and concrete slabs or pads within the planned compressor footprints be completely removed during site preparation. Demolished materials generated from these operations should be transported off site for disposal.

All existing utilities in the construction area should be identified prior to excavation. Live utility lines identified beneath proposed structures should be relocated. Abandoned utility lines beneath structures should be completely removed or filled with grout in order to reduce potential settlement of new structures. Soft or loose soil encountered in utility line excavations should be removed and replaced with structural fill where loose or soft soil is located within structural areas.

Excavations resulting from removing foundations, utilities, or other subsurface elements should be replaced with structural fill. The bottoms of the excavations should be excavated to expose firm subgrade, as approved in the field by a qualified geotechnical engineer. All structural fill used during site preparations should meet the criteria in Sections 5.6 and 5.7 of this report.

5.2. Subgrade Preparation and Evaluation

Following stripping, the site should be cut (rough grading) to establish planned subgrade elevations. Rough grading and stripping will expose the on-site materials at final subgrade elevations. Fill subgrades with slopes in excess of 5H:1V (horizontal to vertical) should be properly benched.

The lateral limits of subgrade preparation should extend at least 5 feet beyond the compressor station equipment area and other areas to receive fill. Soft soil encountered at subgrade elevation should be removed to medium stiff/medium dense or better conditions, or as recommended by the geotechnical engineer during construction.

We recommend that soil exposed at planned subgrade elevations be observed by GeoEngineers prior to placing fill. We recommend that the subgrade be evaluated by proof-rolling. During wet weather, the subgrade should be probed instead of proof-rolled to minimize the potential for subgrade disturbance. Soft or unstable areas identified during proof-rolling and probing activities should be overexcavated to firm bearing conditions and replaced with suitable subbase material. The contractor should use construction equipment that can travel on the subgrade areas without causing damage to the subgrade until the subgrade can be stabilized or covered.

Subgrade stabilization consisting of Imported Select Granular Fill over geotextile may be required for some areas of the site. Geotextile fabric should have a minimum Mullen burst strength of 500 pounds per square inch (psi) and an apparent opening size (AOS) between a U.S. Standard No. 70 and U.S. Standard No. 100. Mirafi 500x is a fabric that meets these specifications.

5.3. Excavation

Based on the materials encountered in our borings, it is our opinion that conventional earthmoving equipment in proper working condition should be capable of making necessary general excavations. We recommend that excavators with smooth edge buckets (i.e., no teeth) be used wherever practical to limit disturbance to the subgrade. The earthwork contractor should be responsible for reviewing this report, including the boring logs, providing their own assessments and providing equipment and methods needed to excavate the site soils while protecting subgrades.

5.4. Dewatering

As discussed in the "Groundwater" section of this report, the regional groundwater was not observed in our explorations and is not likely to be encountered. If perched groundwater is encountered, saturated/wet soils should be dewatered. Sump pumps are expected to adequately address groundwater encountered in shallow excavations. In addition to groundwater seepage, surface water inflow to the excavations during the wet season can be problematic. Provisions for surface water control during earthwork and excavations should be included in the project plans and should be installed prior to commencing earthwork.

5.5. Shoring and Sloping

All excavations should be made in accordance with applicable Occupational Safety and Health Administration (OSHA) and state regulations. The OSHA soil type and associated slopes must be determined by the contractor's designated competent person. We recommend the contractor plan on OSHA Type C soils. Excavations deeper than 4 feet should be shored or laid back at an inclination of 1.5H:1V or flatter if workers are required to enter. Excavations should be laid back or shored at the surface as necessary to prevent soil from falling into excavations.

In our opinion, the contractor is in the best position to observe subsurface conditions continuously throughout the construction process and to respond to the soil and groundwater conditions. Construction site safety is the sole responsibility of the contractor, who also is solely responsible for the means, methods, and sequencing of the construction operations and choices regarding excavations and shoring. Under no circumstances should the information provided by GeoEngineers be interpreted to mean that GeoEngineers is assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

5.6. Structural Fill

Structural fill must be placed directly beneath foundations and beyond the edge of the foundations to a horizontal distance equal to the depth of the fill.

All structural fill soils should be free of debris, clay balls, roots, organic matter, frozen soil, man-made contaminants, particles with greatest dimension exceeding 4 inches and other deleterious materials. The suitability of soil for use as structural fill will depend on the gradation and moisture content of the soil. As the amount of fines in the soil matrix increases, the soil becomes increasingly more sensitive to small changes in moisture content and achieving the required degree of compaction becomes more difficult or impossible. Recommendations for suitable fill material are provided in the following sections.

5.6.1. On-Site Soils

5.6.1.1. Fill Soils

The on-site fill soils appear to consist of poorly graded gravel with varying amounts of silt and sand within anticipated excavation depths. On-site soils can be used as structural fill, provided the material meets the above requirements for structural fill. However, the gravel fill soils will need to be segregated from native subgrade soils to prevent fines contents from becoming too high prior to replacement.

5.6.1.2. Native Soils

The moisture content of the native on-site soil currently is significantly greater than the optimum moisture required for proper compaction. The native clay and silty sand soils are also sensitive to small changes in moisture and will be suitable for use as structural fill only if the soil can be properly moisture-conditioned. Based on our experience working on the Miller Station site and surrounding area it will be extremely difficult if not impossible to moisture condition the native soil and reuse it because of the wet weather in the Mist area and the lack of a suitable area for drying the material. If the soil cannot be properly moisture conditioned, we recommend using imported granular material for structural fill.

5.6.2. Imported Select Granular Fill

Select imported granular material may be used as structural fill. The imported material should consist of pit or quarry run rock, crushed rock, or crushed gravel and sand that is fairly well graded between coarse and fine sizes (approximately 25 to 65 percent passing the U.S. No. 4 sieve). It should have less than 5 percent passing the U.S. No. 200 sieve.

5.6.3. Aggregate Base

Aggregate base material under foundations should consist of imported clean, durable, crushed angular rock. Such rock should be well graded, have a maximum particle size of ¾ inch and have less than 5 percent passing the U.S. No. 200 sieve. In addition, aggregate base shall have a minimum of 75 percent fractured particles according to AASHTO TP-61 and a sand equivalent of not less than 30 percent based on AASHTO T-176.

5.6.4. Aggregate Wearing Surface

Aggregate Wearing Surface material should consist of material such as Oregon Department of Transportation (ODOT) approved shoulder aggregate meeting the requirements of Section 02640 of the ODOT Standard Specifications for Construction. This material generally consists of 1" – 0 or ¾" – 0 crushed rock, including sand, that is uniformly graded from coarse to fine.

5.6.5. Trench Backfill

The on-site soils are not suitable for use as trench backfill within the existing facility or where traffic loads are expected. Unless different requirements are specified by the pipe manufacturer, trench backfill for the utility pipe base and pipe zone should consist of well-graded granular material having a maximum particle size of ¾ inch and less than 8 percent passing the U.S. No. 200 sieve. The material should be free of organic matter and other deleterious materials. Above the pipe zone, crushed aggregate should be used as described above. The pipe bedding and backfill above the pipe zone should be placed and compacted as recommended in the "Fill Placement and Compaction" section of this report.

5.7. Fill Placement and Compaction

Structural fill and aggregate base should be compacted at moisture contents that are within 3 percent of the optimum moisture content as determined by ASTM Standard Practices Test Method D1557 (Modified Proctor). The optimum moisture content varies with gradation and should be evaluated during construction. Material that is not near the optimum moisture content should be moisture conditioned prior to compaction.

Fill and backfill material should be placed in uniform, horizontal lifts and compacted with appropriate equipment. The appropriate lift thickness will vary depending on the material and compaction equipment used. Fill material should be compacted to a minimum of 95 percent of ASTM Test Method D1557. It is the contractor's responsibility to select appropriate compaction equipment and place the material in lifts that are thin enough to meet these criteria. However, in no case should the loose lift thickness exceed 12 inches.

A representative from GeoEngineers should evaluate compaction of each lift of fill. Compaction should be evaluated by compaction testing unless other methods are proposed for oversized materials and are approved by GeoEngineers during construction. These other methods typically involve procedural placement and compaction specifications together with verifying requirements such as probing or proof-rolling.

5.7.1. Area Fills and Bases

Fill placed to raise site grades and aggregate base materials under foundations, slabs, and pavements should be placed on a prepared subgrade that consists of firm, inorganic native soils or compacted fill. Fill should be compacted to at least 95 percent of the maximum dry density (MDD) determined by ASTM Test Method D 1557 (modified Proctor).

Fill placed on slopes should be placed on a keyed and benched surface. Typically, a minimum 4-foot-wide by 2-foot-deep keyway is excavated into competent (medium stiff/medium dense or better) soils at the base of the fill. The slope of the downslope edge of this excavation should not be greater than 1H:1V. After excavation of the keyway, the slope to receive fill should be benched with the benches being excavated into medium stiff/medium dense or better soils. The keyway and benching should be observed by the geotechnical engineer or their representative during construction to verify that the keyway and benches were excavated into competent soils.

5.7.2. Trench Backfill

Pipe bedding and fill in the pipe zone should be compacted to 90 percent of the MDD as determined by ASTM Test Method D 1557, or as recommended by the pipe manufacturer.

In nonstructural areas, trench backfill above the pipe zone should be compacted to at least 85 percent of the MDD as determined by ASTM Test Method D 1557. Suitable native soils that are moisture-conditioned or select granular soils are acceptable in nonstructural areas.

Within structural areas, trench backfill placed above the pipe zone should be compacted to at least 92 percent of the MDD as determined by ASTM Test Method D 1557 at depths greater than 2 feet below the finished subgrade, and to 95 percent within 2 feet of finished subgrade. Trench backfill in structural areas should consist of select granular fill or aggregate base as described in "Structural Fill" section of this report.

5.8. Permanent Slopes

Permanent cut or fill slopes should not exceed a gradient of 2H:1V. Fill slopes should be overbuilt by at least 12 inches and trimmed back to the required slope to maintain a firm face.

Slopes should be planted with appropriate vegetation to provide protection against erosion as soon as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

If seepage is encountered at the face of permanent or temporary slopes, it might be necessary to flatten the slopes or install a subdrain to collect the water to prevent long term instability. GeoEngineers should be contacted to evaluate such conditions on a case-by-case basis.

5.9. Compressor Station Surface and Gravel Surfaced Roads

We recommend a minimum total fill thickness of 30 inches along gravel access roads within the existing facility consisting of a minimum of 24 inches of Imported Select Granular Fill overlain by a minimum 6-inch thickness of Aggregate Wearing Surface material. A representative from GeoEngineers should evaluate the need for overexcavation of fill soils with organic materials at the time of construction. Subgrade stabilization may be required for some areas of the site, particularly to protect heavily traveled areas such as haul roads and construction entrances. The traditional method of subgrade stabilization consists of granular structural fill placed over a geotextile.

Existing granular fill can be included in the minimum fill thickness, provided the depth and consistency of the existing fill is confirmed in the field by GeoEngineers during construction.

Prior to placing the gravel surfacing material, the subgrade should be proof-rolled using a fully-loaded dump truck. One of our representatives should observe the proof-rolling to evaluate the subgrade and determine if additional excavation or soft area mitigation is needed.

We recommend that a separation geotextile, such as Geotex 104F or approved alternate, be placed between the subgrade and aggregate base layers. The Granular Structural Fill and Aggregate Wearing Surface materials should be placed in lifts and compacted to at least 95 percent of the MDD determined in accordance with ASTM D 1557.

6.0 FOUNDATION SUPPORT RECOMMENDATIONS

6.1. General

Based on Burns & McDonnell's (B&M) revised geotechnical scope of work document (titled 157831 NWN North Mist 2 and Mist Miller Geotech SOW-r1), site development generally consists of replacement of two existing compressors; the replacement compressors are estimated to weigh 120 kips each. We understand that the project team intends to consider both shallow and deep foundations to support the compressors. General recommendations for shallow foundations are also provided below.

Due to the variability of site soils and unknown loading conditions, we are providing general foundation support recommendations. Once structural loads have been finalized we should review and modify our recommendations as needed.

6.2. Shallow Foundation Subgrade Preparation

The native soft soils are generally inadequate to support foundations bearing directly on the soft or loose soils. We recommend that shallow foundations be founded below the local frost line of 12 inches (OSSC 2022) and that the soft clay soils and loose silty sand be removed to a depth of 30 inches beneath the footing subgrade and be replaced with 24 inches of Imported Select Granular Fill and 6 inches of Aggregate Base. Existing granular fill can be included in the minimum fill thickness, provided the depth and consistency of the existing fill is confirmed in the field by GeoEngineers during construction. The width of the overexcavation and placement of Structural Fill should extend beyond the edge of the footing a distance equal to the depth of the overexcavation below the base of the footing. Compaction of aggregate base rock should be performed as described above in the "Fill Placement and Compaction" Section 5.7.

Foundation bearing surface elevations are expected to be above the groundwater table. However, if water infiltrates and pools in the foundation excavations, the water along with any disturbed soil, should be removed before placing the aggregate base rock and reinforcing steel.

We recommend GeoEngineers observe all foundation excavations before placing concrete forms and reinforcing steel or precast foundations to determine that bearing surfaces have been adequately prepared and the soil conditions are consistent with those observed during our explorations.

6.3. Spread Footings

We recommend footings have a minimum width of 24 inches and the bottom of the exterior footings be founded at least 12 inches below the lowest adjacent grade. The recommended minimum footing depth is equal to the anticipated frost depth.

We recommend conventional spread footings be proportioned using a maximum allowable bearing pressure of 2,500 pounds per square foot (psf) if supported on subgrades prepared in accordance with Section 6.2. This bearing pressure applies to the total of dead and long-term live loads and may be increased by one-third when considering earthquake or wind loads. This is a net bearing pressure. The weight of the footing and overlying backfill can be ignored in calculating footing sizes.

6.4. Spread Footing Settlement

We anticipate that the soft clays and loose silty sands underlying the site will have a moderate settlement potential relative to the proposed structural loads. Strip footings and column footings designed and constructed as recommended are expected to experience settlements of approximately 1 inch or less. Differential settlements of up to one half of the total settlement magnitude can be expected between adjacent foundation elements supporting comparable loads.

6.5. Lateral Resistance

Lateral loads on shallow and mat foundations can be resisted by passive earth pressures on the sides of footings and by friction on the bearing surface. We recommend that passive earth pressures be calculated using an equivalent fluid unit weight of 200 pounds per cubic foot (pcf) for foundations confined by existing fill consisting of medium dense or denser silty gravel, or 350 pcf if confined by imported granular fill extending two times the depth beyond the edge of the footing.

We recommend using a friction coefficient 0.45 for foundations bearing on a minimum 2-foot thickness of Structural Fill. The passive earth pressure and friction components may be combined, provided the passive component does not exceed two-thirds of the total.

The passive earth pressure value is based on the assumptions that the adjacent grade is level and groundwater remains below the base of the footing throughout the year. The top 1 foot of soil should be neglected when calculating passive lateral earth pressures, unless the adjacent area is covered with pavement. The lateral resistance values do not include safety factors. We recommend a minimum factor of safety of 1.5 or as determined by the project structural engineer.

6.6. Floor Slabs and Mat Foundations

Concrete mat foundations may be supported on subgrades prepared in accordance with Section 6.2. We recommend that floor slabs be underlain by at least 6 inches of Aggregate Base to aid as a capillary break. Mat foundations and floor slabs supported on subgrades prepared in Accordance with Section 6.2 can be designed using a coefficient of subgrade reaction modulus (k1) of 150 pounds per cubic inch (pci). Slab and mat foundation subgrades should be prepared as described in Section 5.2 of this report.

Based on a total dead load of 350 psf, we estimate that mat foundations constructed as recommended will settle approximately 1 inch or less. Differential settlements less than ½ inch across no less than 25 feet can be expected.

We recommend mat foundations be proportioned using a maximum allowable bearing pressure of 750 psf. This bearing pressure applies to the total of dead and long-term live loads and may be increased by one-third when considering wind loads. The allowable bearing pressure should not be increased by one-third when considering seismic loads due to the high liquefaction susceptibility of the site. This is a net bearing pressure. The weight of the footing and overlying backfill can be ignored in calculating footing sizes.

Vapor barriers are not required under floor slabs. However, vapor barriers are often required by flooring manufacturers to protect flooring and flooring adhesives. A typical vapor barrier consists of plastic sheeting covered with 2 inches of sand. Many flooring manufacturers will warrant their product only if a vapor barrier is installed according to their recommendations. Selection and design of an appropriate vapor barrier, if needed, should be based on discussions among members of the design team. We can provide additional information to assist you with your decision.

6.7. Dynamic Soil Properties

We recommend using a unit weight of 125 pcf, a Poisson's Ratio of 0.3, a dynamic shear modulus of 940 ksf and a damping ratio of 0.05 for the existing on-site fill soils, or granular structural fill placed in accordance with the recommendations given in the Section 5.0 of this report.

6.8. Design Spectrum Parameters

Parameters provided in Table 1 are based on the conditions encountered during our subsurface exploration program and the procedure outlined in the 2021 International Building Code (IBC), which references the 2016 Minimum Design Loads for Buildings and Other Structures (American Society of Civil Engineers [ASCE] 7-16). Parameters listed in Table 1 below are code level parameters and may be used to determine design ground motions for structural design.

TABLE 1. SEISMIC DESIGN PARAMETERS

Parameter	Recommended Value[1]
Site Class	D
Mapped Spectral Response Acceleration at Short Period (S_S)	0.969 g
Mapped Spectral Response Acceleration at 1 Second Period (S_1)	0.501 g
Site Modified Peak Ground Acceleration (PGA_M)	0.523 g
Site Amplification Factor at 0.2 second period (F_a)	1.112
Site Amplification Factor at 1.0 second period (F_v)	1.8
Design Spectral Acceleration at 0.2 second period (S_{DS})	0.719 g
Design Spectral Acceleration at 1.0 second period (S_{D1})	0.60 g

Notes:

[1] Parameters developed based on Latitude 46.020509° and Longitude -123.269707° using the ATC Hazards online tool.

6.9. Liquefaction

Liquefaction refers to the condition when vibration or ground shaking, usually from earthquake forces, results in the development of excess pore pressures in saturated soils with subsequent loss of strength in the soil deposit affected. In general, soils that are susceptible to liquefaction include very loose to medium dense clean to silty sands and low plasticity silts. For liquefaction to occur, soils must be saturated.

As discussed in the Groundwater Conditions section of this report, groundwater may perch on layers of cohesive soils across portions of the site. Loose to medium dense sand to silty sand encountered on site is not expected to be impacted by perched groundwater. Therefore, liquefaction of the on-site soils is not anticipated following a design level earthquake.

6.10. Construction Observation

Satisfactory earthwork performance depends to a large degree on quality of construction. Sufficient monitoring of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during the subsurface exploration. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

We recommend that GeoEngineers be retained to observe construction at the site to confirm that subsurface conditions are consistent with the site explorations, and to confirm that the intent of project plans and specifications relating to earthwork construction are being met.

7.0 LIMITATIONS

This report has been prepared for the exclusive use of NW Natural and their authorized agents and/or regulatory agencies for the Mist Resiliency project in Columbia County, Oregon. This report is not intended for use by others, and the information contained herein is not applicable to other sites. No other party may rely on the product of our services unless we agree in advance and in writing to such reliance.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with generally accepted geotechnical engineering practices in the area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

Please refer to Appendix C, Report Limitations and Guidelines for Use, for additional information pertaining to use of this report.

8.0 REFERENCES

GeoEngineers, Inc. 2017. Geotechnical Engineering Report, Miller Station – Proposed Mist Control Building Addition, Columbia County, Oregon. File No. 6024-200-00, February 6, 2017.

International Code Council. 2021. 2021 International Building Code.

Oregon Department of Geology and Mineral Industries (DOGAMI). 2015. Oregon Geologic Data Compilation Release 6. Compiled by Smith, R.L and Roe, W.P.

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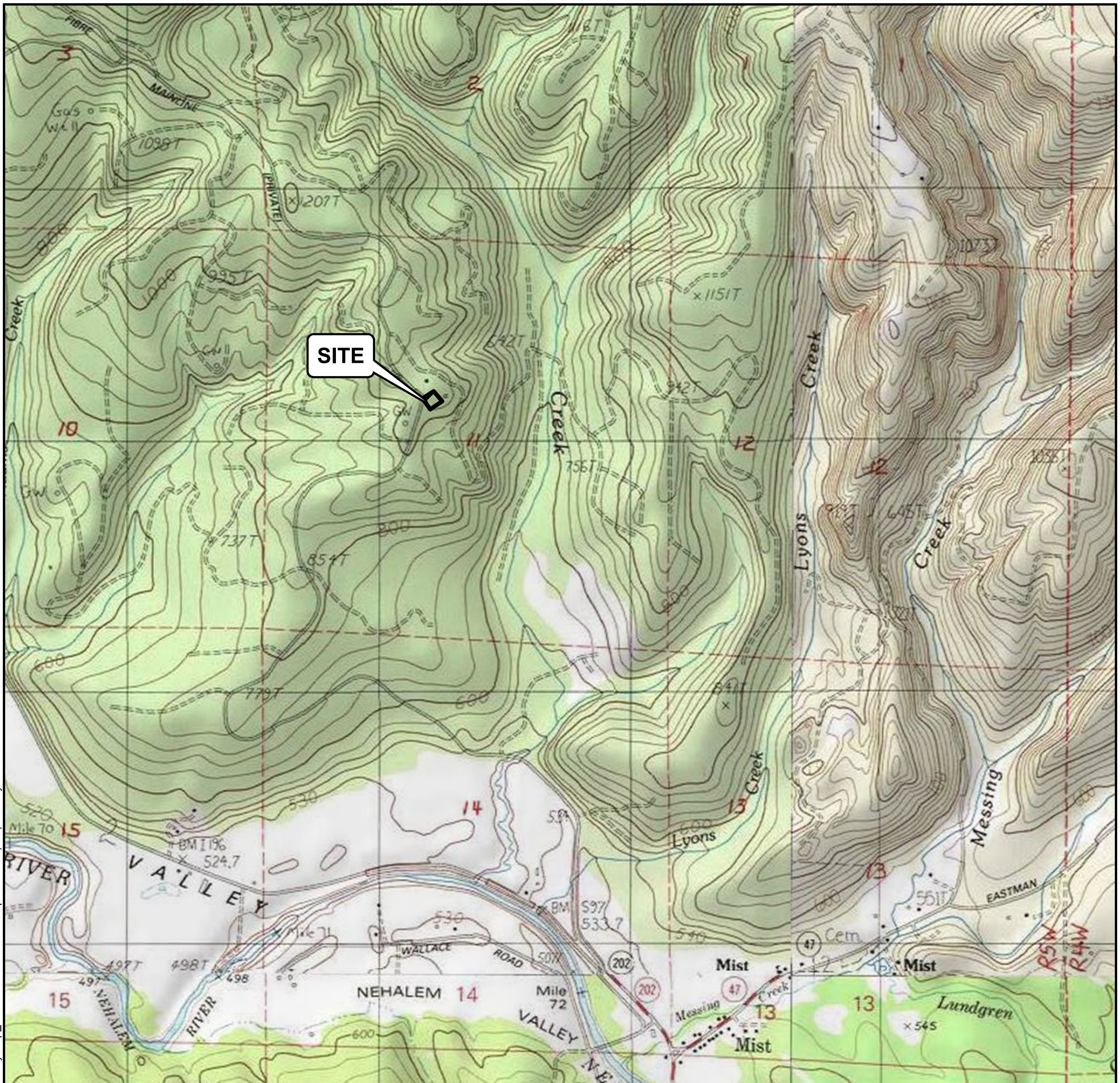
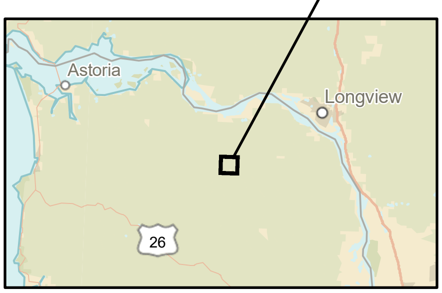
Yves Lacroix and Harry Horn. 1973. Direct Determination and Indirect Evaluation of Relative Density and Its Use on Earthwork Construction Projects: in Evaluation of Relative Density and Its Role in Geotechnical Projects Involving Cohesionless Soils: ASTM Special Technical Publication 523, pp. 251-280.

FIGURES





N

0 2,000
Feet

Source(s):
- ESRI

Coordinate System: NAD 1983 UTM Zone 10N

Disclaimer: This figure was created for a specific purpose and project. Any use of this figure for any other project or purpose shall be at the user's sole risk and without liability to GeoEngineers. The locations of features shown may be approximate. GeoEngineers makes no warranty or representation as to the accuracy, completeness, or suitability of the figure, or data contained therein. The file containing this figure is a copy of a master document, the original of which is retained by GeoEngineers and is the official document of record.

Vicinity Map

Mist Resiliency Project
Miller Station Resiliency Area
Columbia County, Oregon

GEOENGINEERS

Figure 1

P:\6\6024308\GIS\6024308_Project\6024308_Project.aprx\602430800_F01_VicinityMap_MillerStation Date Exported: 07/31/23 by ccabrera

Legend

Boring Location

N

0 200
Feet

Note(s):
1. GeoEngineers, Inc. has not verified the field location of the existing utilities.

Source(s):
- Aerial from Google Earth Pro, dated 10/12/18.

Coordinate System: Oregon State Plane, North Zone, NAD27, US Foot

Disclaimer: This figure was created for a specific purpose and project. Any use of this figure for any other project or purpose shall be at the user's sole risk and without liability to GeoEngineers. The locations of features shown may be approximate. GeoEngineers makes no warranty or representation as to the accuracy, completeness, or suitability of the figure, or data contained therein. The file containing this figure is a copy of a master document, the original of which is retained by GeoEngineers and is the official document of record.

Boring Location Map

Mist Resiliency Project
Miller Station Resiliency Area
Columbia County, Oregon

GEOENGINEERS

Figure 2

JAH : BTL

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APPENDICES

GeoEngineers

APPENDIX A
Field Explorations and Laboratory Testing

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

We completed two borings (MM B-1 and MM B-2) between June 5 and 8, 2023 at the approximate locations shown in Figure 2. Borings MM B-1 and MM B-2 were completed with a track mounted drill rig using mud rotary drilling methods to depths of 80 feet below ground surface (bgs) and 40 feet bgs, respectively. Western States Soil Conservation, Inc. of Hubbard, Oregon drilled the borings. A representative from our Portland, Oregon office observed field activities, classified soil encountered, obtained representative samples, observed groundwater conditions, and prepared a log of each exploration. The borings were backfilled with ⅜-inch bentonite chips at the conclusion of each exploration.

Vacuum excavation was utilized to excavate fill soils to depths of 5 to 8 feet bgs in order to confirm there were no utilities present at the boring locations. Soil samples were obtained from the borings at representative intervals using split spoon samplers and Shelby tubes. Soils encountered in the borings were classified in the field by a GeoEngineers representative in general accordance with ASTM International (ASTM) Standard Practices Test Method D2488, the Standard Practice for the Classification of Soils (Visual-Manual Procedure) which is described in Figure A-1. The boring logs are presented in Figures A-2 and A-3. Soil/rock classifications and sampling intervals are shown in the boring logs. Inclined lines at the material contacts shown on the log indicate uncertainty as to the exact contact elevation, rather than the inclination of the contact itself.

Standard penetration tests (SPTs) were performed during soil sampling in general accordance with ASTM Test Method D1586. The sampler was driven with a 140-pound hammer falling 30 inches. The number of blows required to drive the sampler 1 foot, or as otherwise indicated, into the soils is shown adjacent to the sample symbols on the boring log. Disturbed samples were obtained from the split spoon sampler for subsequent classification and index testing. Bedrock core samples were obtained using a 2.5-inch I.D. HQ core barrel.

The relative density of the SPT samples recovered at each interval was evaluated based on correlations with lab and field observations in general accordance with the values outlined in Table A-1 below.

TABLE A-1. CORRELATION BETWEEN BLOW COUNTS AND RELATIVE DENSITY *

Cohesive Soils (Clay/Silt)						
Parameter	Very Soft	Soft	Medium Stiff	Stiff	Very Stiff	Hard
Blows, N	< 2	2 – 4	4 - 8	8 – 16	16 - 32	>32

Cohesionless Soils (Gravel/Sand/Silty Sand) **					
Parameter	Very Loose	Loose	Medium Dense	Dense	Very Dense
Blows, N	0 – 4	4 – 10	10 – 30	30 - 50	> 50

Notes:

* After Terzaghi, K and Peck, R.B., "Soil Mechanics in Engineering Practice," John Wiley & Sons, Inc., 1962.

** Classification applies to soils containing additional constituents; that is, organic clay, silty or clayey sand, etc.

Geotechnical Laboratory Testing

General

All soil samples obtained from the borings were visually classified in the field and/or in our laboratory using a system based on the Unified Soil Classification System (USCS) and ASTM classification methods. ASTM Test Method D 2488 was used to visually classify the soil samples, while ASTM D 2487 was used to classify the soils based on laboratory tests results. These classification procedures are incorporated in the boring logs shown in Figures A-2 and A-3.

Moisture Content Determinations

Moisture contents determinations were determined in general accordance with ASTM D 2216 for selected samples obtained from the boring. The results of these tests are presented in the boring logs at the sample depths.

Percent Passing U.S. No. 200 Sieve

Selected samples were “washed” through the U.S. No. 200 mesh sieve to determine the relative percentage of coarse- and fine-grained particles in the soil. The percent passing value represents the percentage by weight of the sample finer than the U.S. No. 200 sieve. These tests were conducted in general accordance with ASTM D 1140, and the results are shown in the boring logs at the sample depths.

Atterberg Limits Determinations

Selected samples were tested to determine their Atterberg limits in general accordance with ASTM D 4318. The results of these tests are shown in the boring logs at the sample depths and presented in this Appendix in Figure A-4

Corrosion Potential Testing

General

Native soil samples obtained from the borings between depths of 5 feet bgs and 14 feet bgs were selected and sent to Gerhart Cole, Inc.’s laboratory in Midvale, Utah to conduct corrosion potential testing. The results of the testing are presented as an attachment to Appendix A. The corrosion potential testing included the following tests.

Electrical Resistivity Testing

Electrical resistivity tests were performed on the selected soil samples in accordance with American Association of State Highway and Transportation Officials (AASHTO) T 288.

Chloride Ion

Selected soil samples were tested for Chloride Ion in accordance with U.S. Environmental Protection Agency (EPA) 300.0.

Soluble Sulfates

Selected soil samples were tested for soluble sulfate in accordance with EPA 300.0.

Sulfite Ion Content

Selected soil samples were tested for total sulfur content in accordance with EPA 6010D. Sulfite ion content can subsequently be determined with the following equation:

TOTAL SULFUR – SOLUBLE SULFATE = SULFITE ION CONTENT

pH Testing
Selected soil samples were tested for pH in general accordance with EPA 9045D.

Carbonate and Bicarbonate
Selected soil samples were tested for carbonates and bicarbonates in general accordance with Standard Method (SM) 2320B.

Redox Potential
Selected soil samples were tested for oxidation/reduction (redox) potential in general accordance with SM 2580B.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% RETAINED ON NO. 200 SIEVE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS
				SP	POORLY-GRADED SANDS, GRAVELLY SAND
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% PASSING NO. 200 SIEVE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS, ROCK FLOUR, CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS AND SILTS OF MEDIUM TO HIGH PLASTICITY
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Multiple symbols are used to indicate borderline or dual soil classifications

ADDITIONAL MATERIAL SYMBOLS

SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
	AC	Asphalt Concrete
	CC	Cement Concrete
	CR	Crushed Rock/ Quarry Spalls
	SOD	Sod/Forest Duff
	TS	Topsoil

Groundwater Contact

Measured groundwater level in exploration, well, or piezometer

Measured free product in well or piezometer

Graphic Log Contact

Distinct contact between soil strata

Approximate contact between soil strata

Material Description Contact

Contact between geologic units

Contact between soil of the same geologic unit

Sampler Symbol Descriptions

- 2.4-inch I.D. split barrel / Dames & Moore (D&M)
- Standard Penetration Test (SPT)
- Shelby tube
- Piston
- Direct-Push
- Bulk or grab
- Continuous Coring

Blowcount is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted). See exploration log for hammer weight and drop.

"P" indicates sampler pushed using the weight of the drill rig.

"WOH" indicates sampler pushed using the weight of the hammer.

Laboratory / Field Tests

%F	Percent fines
%G	Percent gravel
AL	Atterberg limits
CA	Chemical analysis
CP	Laboratory compaction test
CS	Consolidation test
DD	Dry density
DS	Direct shear
HA	Hydrometer analysis
MC	Moisture content
MD	Moisture content and dry density
Mohs	Mohs hardness scale
OC	Organic content
PM	Permeability or hydraulic conductivity
PI	Plasticity index
PL	Point load test
PP	Pocket penetrometer
SA	Sieve analysis
TX	Triaxial compression
UC	Unconfined compression
UU	Unconsolidated undrained triaxial compression
VS	Vane shear

Sheen Classification

NS	No Visible Sheen
SS	Slight Sheen
MS	Moderate Sheen
HS	Heavy Sheen

NOTE: The reader must refer to the discussion in the report text and the logs of explorations for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the specific exploration locations and at the time the explorations were made; they are not warranted to be representative of subsurface conditions at other locations or times.

Key to Exploration Logs

GeoEngineers

Figure A-1

Drilled	Start 6/6/2023	End 6/7/2023	Total Depth (ft)	81.5	Logged By WCS / Checked By AB	Driller	Western States Soil Conservation, Inc.	Drilling Method	Mud Rotary

Surface Elevation (ft) / Vertical Datum	1051 / NAVD88	Hammer Data	Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment	CME 55 Track Rig
Latitude / Longitude	46.02058142 / -123.269556	System Datum	NAD83 (feet)	Groundwater not observed at time of exploration	

Notes: 3-inch-diameter PVC casing installed to full depth of boring upon completion.

Elevation (feet)	Depth (feet)	Recovered (in)	Blows/foot	Sample Name / Testing	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
1050	0				GP-GM	Gray poorly-graded gravel with silt and sand (dense, moist) (fill)			Vacuum excavation to approximately 5 feet
1045	5	12	3	1	CH	Brown to dark brown fat clay (soft, moist) (weathered Pittsburg Bluff formation)			
		24		2 MC			46		
1040	10	12	8	3		Becomes orange-gray and stiff			
		24		4					
		16	11	5 MC		Becomes wet	84		
1035	15	12	5	6		Becomes gray with trace sand and medium stiff			
1030	20		10	7		Becomes stiff			
1025	25	24		8					
1020	30	18	12	9 %F; AL			44	60	AL (LL = 60, PI = 32)
	35								

Note: See Figure A-1 for explanation of symbols; Figure A-2 for explanation of bedrock terms.
Coordinates Data Source: Horizontal approximated based on GPS. Vertical approximated based on GPS.

Date:10/19/23 Path:P:\6\6024308\GINT\602430800.GPJ DBLibrary/Library:GEOENGINEERS_DF_STD_US_JUNE_2017.GLB/GEI8_GEOTECH_STANDARD_%F_NO_GW

Log of Boring MM B-1

GEOENGINEERS

Project: Miller Station Resiliency Area
Project Location: Columbia County, Oregon
Project Number: 6024-308-00

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
1015	35		17	9		10						
1010	40		18	14		11			Becomes very stiff			
1005	45		18	19		12						
1000	50		18	14		13 MC				44		
			12			14						
								SM	Brown silty sand (medium dense, moist)			
			18	18		15						
995	55		18	18		16						
990	60		18	19		17						
985	65		18	63		18 %F			Becomes very dense	43	45	
								MH	Brown-gray sandy elastic silt (very stiff, moist)			
980	70		18	21		19 %F				46	70	
975	75		18	14		20 %F; AL			Becomes grayish black and stiff	43	52	AL (LL = 53, PI = 21)

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
970	80	X	18	26		21 / %F			Becomes with sand and very stiff	42	84	

Log of Boring MM B-1 (continued)

GeoEngineers

Project: Miller Station Resiliency Area
Project Location: Columbia County, Oregon
Project Number: 6024-308-00

Drilled	Start 6/8/2023	End 6/8/2023	Total Depth (ft) 41.5	Logged By WCS / Checked By AB	Driller: Western States Soil Conservation, Inc.	Drilling Method: Mud Rotary

Surface Elevation (ft) 1044 / Vertical Datum NAVD88	Hammer Data: Autohammer 140 (lbs) / 30 (in) Drop	Drilling Equipment: CME 55 Track Rig
Latitude 46.020461 / Longitude -123.269973	System Datum: NAD83 (feet)	Groundwater not observed at time of exploration

Notes:

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	0							GP	Gray poorly-graded gravel (dense, moist) (fill)			Vacuum excavation to approximately 8 feet
1040	5											
									Becomes coarse gravel			
1035								SM	Brown silty sand (loose, moist) (weathered Pittsburg Bluff formation)			
	10		18	8		1 / %F; AL				51	48	AL (non-plastic)
			20			2						
1030												
	15		18	2		3 / MC			Becomes very loose and wet	60		
1025			20			4						
	20		18	6		5 / MC			Becomes gray and loose	46		
1020												
	25		18	13		6			Becomes medium dense			
1015												
	30		18	13		7 / MC				41		
1010	35											

Note: See Figure A-1 for explanation of symbols; Figure A-2 for explanation of bedrock terms.
Coordinates Data Source: Horizontal approximated based on GPS. Vertical approximated based on GPS.

Log of Boring MM B-2

GEOENGINEERS

Project: Miller Station Resiliency Area
Project Location: Columbia County, Oregon
Project Number: 6024-308-00

Figure A-3
Sheet 1 of 2

Elevation (feet)	Depth (feet)	Interval	Recovered (in)	Blows/foot	Collected Sample	Sample Name / Testing	Graphic Log	Group Classification	MATERIAL DESCRIPTION	Moisture Content (%)	Fines Content (%)	REMARKS
	35		18	14		8						
1005												
	40		18	18		9						

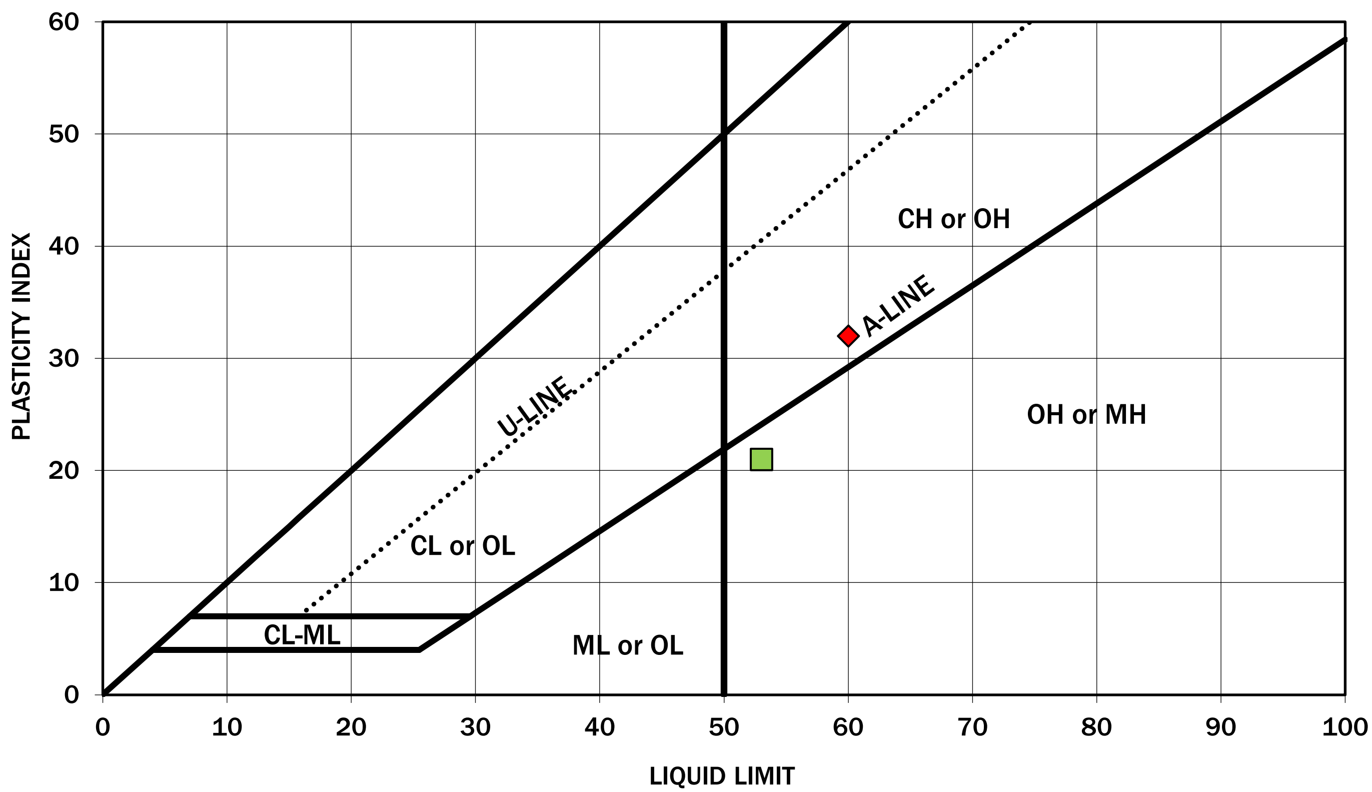
Log of Boring MM B-2 (continued)

GeoEngineers

Project: Miller Station Resiliency Area
Project Location: Columbia County, Oregon
Project Number: 6024-308-00

Figure A-3
Sheet 2 of 2

PLASTICITY CHART



Symbol	Boring Number	Depth (feet)	Moisture Content (%)	Liquid Limit (%)	Plasticity Index (%)	Soil Description
◆	MMB-1	30	44	60	32	Fat Clay (CH)
■	MMB-1	75	43	53	21	Elastic Silt (MH)

The liquid limit and plasticity index were obtained in general accordance with ASTM D 4318.

Atterberg Limits Test Results

Mist Resiliency Project
Miller Station Resiliency Area
Columbia County, Oregon

GEOENGINEERS

Figure A-4

PO Box 880
Midvale, Utah 84047
801.849.0055
gerhartcole.com

To:	Mr. Jarad Hoffman GeoEngineers, Inc 4000 Kruse Way Place Bldg. 3 Suite 200 Lake Oswego, Oregon 97035
Project:	Mist Resiliency
Gerhart Cole Project Number:	23-1681
Client Project Number	6024-308-00
Report Date:	August 24, 2023

Attached are the requested laboratory test results. These laboratory results are for the addressee and must be presented in their entirety without alteration, except with permission.

Gerhart Cole's Laboratory is accredited through the AASHTO Accreditation Program. The results provided were tested in general accordance with the referenced standards. Any test methods reported in this document outside the scope of this accreditation are marked with an asterisk in the header of the individual test report.

Gerhart Cole will dispose of samples two (2) weeks after the date above. Please contact us for storage pricing.

Approved By:____________________________________

Zach Gibbs, PE (UT, MI) Laboratory Director

AASHTO ACCREDITED

Moisture Content Determination

ASTM D2937 / D2216

Project: **Mist Resiliency**

No: **6024-308-00 (23-1681)**

Location: **-**

X:\PROJECTS\23-1681 Mist Resiliency (6024-308-00)\[2023-08-07_MC.xlsx]1

Sample Info.	Sample:	**MMB-1**	**MMB-2**	**NMB-1**	**NMB-2**	**NMB-3**	**NMB-4**
	Depth:	**6.5-8.5 ft**	**11.5-13.5 ft**	**Composite 5-20 ft**	**Composite 5-14 ft**	**Composite 5-14 ft**	**Composite 5-12.5 ft**
	Date Sampled:	**-**	**-**	**-**	**-**	**-**	**-**
	Date tested:	**09-Aug-23**	**09-Aug-23**	**09-Aug-23**	**09-Aug-23**	**09-Aug-23**	**09-Aug-23**
	Laboratory sample description:	**tn - gy clay**	**lt bn - lt ol bn clay**	**bn - dk bn clay**	**lt ol bn - lt bn silt**	**bn - dk bn clay**	**dk yl bn - lt bn clay**
Moisture	Wet soil + tare (g)	445.21	422.31	1176.68	497.10	688.50	805.90
	Dry soil + tare (g)	336.92	300.69	893.25	353.91	483.31	618.31
	Tare (g)	117.33	118.10	119.26	150.66	117.02	197.82
	Moisture content, w (%)	49.3	66.6	36.6	70.5	56.0	44.6
Comments							
QC/QA	Tested By:	JC	JC	JC	JC	JC	JC
	Reduced By:	JC	JC	JC	JC	JC	JC
	Reviewed By:	DBW	DBW	DBW	DBW	DBW	DBW

Minimum Laboratory Soil Resistivity*

After AASHTO T 288

Project: Mist Resiliency
No: 6024-308-00 (23-1681)
Location: -

Section									
Sample Info.	Test Hole/Pit:	MMB-1				MMB-2			
	Depth (ft):	6.5-8.5 ft				11.5-13.5 ft			
	Date Sampled:	-				-			
	Date Tested:	8/9/2023				8/9/2023			
	Laboratory sample description:	tn - gy clay				lt bn - lt ol bn clay			
	Meter Serial Number	4385				4777			
Soil Data	Wet soil + tare (g)	497.13				513.5			
	Air dry soil + tare (g)	383.06				365.44			
	Tare (g)	117.19				117.22			
	Moisture Content, w (%)	42.9				59.6			
	Dry Soil after #10 sieve (g)	246.64				212.01			
Moisture	Water for 10% initial MC (mL)	24.66				21.20			
	% / Step (%)	5				5			
	Water for % / Step (mL)	12.3				10.6			
Resistivity data	Trial	Soil condition (%)	Resistivity (Ω-cm)	Soil condition (%)	Resistivity (Ω-cm)	Soil condition (%)	Resistivity (Ω-cm)	Soil condition (%)	Resistivity (Ω-cm)
	1	Initial	58,000			Initial	1,080,000		
	2	Initial +5	17,000			Initial +5	170,000		
	3	Initial +10	8,000			Initial +10	104,000		
	4	Initial +15	5,700			Initial +15	40,500		
	5	Initial +20	4,500			Initial +20	14,000		
	6	Initial +25	4,300			Initial +25	15,500		
	7	Initial +30	4,300			Initial +30	5,700		
	8	Initial +35	4,800			Initial +35	5,700		
	9	Initial +40	4,500			Initial +40	6,300		
	10					Initial +45	6,350		
	Minimum resistivity (Ω-cm)	4,300				5,700			
QC/QA	Tested By:	JWS				CJS			
	Reduced By:	JC				JC			
	Reviewed By:	DBW				DBW			

Comments:

X:\PROJECTS\23-1681 Mist Resiliency (6024-308-00)\[2023-08-07_Resistivity.xlsx]1

Minimum Laboratory Soil Resistivity*

After AASHTO T 288

Project: Mist Resiliency

No: 6024-308-00 (23-1681)

Location: -

Section	Field	NMB-1				NMB-2			
Sample Info.	Test Hole/Pit:	NMB-1				NMB-2			
	Depth (ft):	Composite 5-20 ft				Composite 5-14 ft			
	Date Sampled:	-				-			
	Date Tested:	8/9/2023				8/9/2023			
	Laboratory sample description:	bn - dk bn clay				lt ol bn - lt bn silt			
	Meter Serial Number	4777				4777			
Soil Data	Wet soil + tare (g)	573.6				677.11			
	Air dry soil + tare (g)	488.07				520.89			
	Tare (g)	196.72				198.02			
	Moisture Content, w (%)	29.4				48.4			
	Dry Soil after #10 sieve (g)	282.17				260.76			
Moisture	Water for 10% initial MC (mL)	28.22				26.08			
	% / Step (%)	5				5			
	Water for % / Step (mL)	14.1				13.0			
Resistivity data	Trial	Soil condition (%)	Resistivity (Ω-cm)	Soil condition (%)	Resistivity (Ω-cm)	Soil condition (%)	Resistivity (Ω-cm)	Soil condition (%)	Resistivity (Ω-cm)
	1	Initial	990,000			Initial	500,000		
	2	Initial +5	270,000			Initial +5	74,000		
	3	Initial +10	110,000			Initial +10	47,000		
	4	Initial +15	49,000			Initial +15	29,000		
	5	Initial +20	22,000			Initial +20	21,000		
	6	Initial +25	23,000			Initial +25	14,000		
	7	Initial +30	24,000			Initial +30	7,500		
	8					Initial +35	7,700		
	9					Initial +40	8,000		
	10								
	Minimum resistivity (Ω-cm)	22,000				7,500			
QC/QA	Tested By:	TK				TK			
	Reduced By:	JC				JC			
	Reviewed By:	DBW				DBW			

Comments:

Minimum Laboratory Soil Resistivity*

After AASHTO T 288

Project: Mist Resiliency

No: 6024-308-00 (23-1681)

Location: -

Section	Field	NMB-3				NMB-4			
Sample Info.	Test Hole/Pit:	NMB-3				NMB-4			
	Depth (ft):	Composite 5-14 ft				Composite 5-12.5 ft			
	Date Sampled:	-				-			
	Date Tested:	8/9/2023				8/9/2023			
	Laboratory sample description:	bn - dk bn clay				dk yl bn - lt bn clay			
	Meter Serial Number	4777				4777			
Soil Data	Wet soil + tare (g)	699.22				636.28			
	Air dry soil + tare (g)	553.47				519.09			
	Tare (g)	196.58				196.16			
	Moisture Content, w (%)	40.8				36.3			
	Dry Soil after #10 sieve (g)	251.56				241.99			
Moisture	Water for 10% initial MC (mL)	25.16				24.20			
	% / Step (%)	5				5			
	Water for % / Step (mL)	12.6				12.1			
Resistivity data	Trial	Soil condition (%)	Resistivity (Ω-cm)	Soil condition (%)	Resistivity (Ω-cm)	Soil condition (%)	Resistivity (Ω-cm)	Soil condition (%)	Resistivity (Ω-cm)
	1	Initial	510,000			Initial	875,000		
	2	Initial +5	89,000			Initial +5	235,000		
	3	Initial +10	36,000			Initial +10	85,500		
	4	Initial +15	24,000			Initial +15	39,000		
	5	Initial +20	15,000			Initial +20	23,000		
	6	Initial +25	9,200			Initial +25	19,000		
	7	Initial +30	8,900			Initial +30	19,000		
	8	Initial +35	9,300			Initial +35	19,500		
	9	Initial +40	9,900			Initial +40	20,000		
	10								
	Minimum resistivity (Ω-cm)	8,900				19,000			
QC/QA	Tested By:	TK				TK			
	Reduced By:	JC				JC			
	Reviewed By:	DBW				DBW			

Comments:

X:\PROJECTS\23-1681 Mist Resiliency (6024-308-00)\[2023-08-07_Resistivity.xlsx]3

8/16/2023

Work Order: 23H0932
Project: 6024-308-00 (23-1681)

Gerhart Cole, Inc.
Attn: Zach Gibbs
7657 South Holden Street
Midvale, UT 84047

Client Service Contact: 801.262.7299

The analyses presented on this report were performed in accordance with the National Environmental Laboratory Accreditation Program (NELAP) unless noted in the comments, flags, or case narrative. If the report is to be used for regulatory compliance, it should be presented in its entirety, and not be altered.

Approved By:

Reed Hendricks, Director of Operations

9632 South 500 West
Sandy, UT 84070
O:(801) 262-7299 F: (866) 792-0093
www.ChemtechFord.com

Certificate of Analysis

Gerhart Cole, Inc.
Zach Gibbs
7657 South Holden Street
Midvale, UT 84047

PO#:
Receipt: **8/9/23 16:45 @ 24.9** °C
Date Reported: 8/16/2023
Project Name: **6024-308-00 (23-1681)**

Sample ID: **MMB-1 / 6.5-8.5 ft**

Matrix: **Solid**
Lab ID: **23H0932-01**
Date Sampled: **8/7/23 0:00**
Sampled By: **client**

	Result	Units	Minimum Reporting Limit	Method	Preparation Date/Time	Analysis Date/Time	Flag(s)
Inorganic							
Chloride, Soluble (IC)	ND	mg/kg dry	15	EPA 300.0	8/10/23	8/10/23	
pH	5.0	pH Units	0.1	EPA 9045D	8/10/23	8/10/23	
Sulfate, Soluble (IC)	ND	mg/kg dry	15	EPA 300.0	8/10/23	8/10/23	
Total Solids	68.0	%	0.1	CTF8000	8/10/23	8/11/23	
Metals							
Sulfur, Total	58.5	mg/kg dry	0.25	EPA 6010D	8/11/23	8/14/23	

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Zach Gibbs
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Midvale, UT 84047

PO#:
Receipt: **8/9/23 16:45 @ 24.9** °C
Date Reported: 8/16/2023
Project Name: **6024-308-00 (23-1681)**

Sample ID: **MMB-2 / 11.5-13.5 ft**

Matrix: **Solid**
Lab ID: **23H0932-02**
Date Sampled: **8/7/23 0:00**
Sampled By: **client**

	Result	Units	Minimum Reporting Limit	Method	Preparation Date/Time	Analysis Date/Time	Flag(s)
Inorganic							
Chloride, Soluble (IC)	ND	mg/kg dry	17	EPA 300.0	8/10/23	8/10/23	
pH	5.0	pH Units	0.1	EPA 9045D	8/10/23	8/10/23	
Sulfate, Soluble (IC)	ND	mg/kg dry	17	EPA 300.0	8/10/23	8/10/23	
Total Solids	59.0	%	0.1	CTF8000	8/10/23	8/11/23	
Metals							
Sulfur, Total	86.3	mg/kg dry	0.22	EPA 6010D	8/11/23	8/14/23	

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PO#:
Receipt: **8/9/23 16:45 @ 24.9** °C
Date Reported: 8/16/2023
Project Name: **6024-308-00 (23-1681)**

Sample ID: **NMB-1 / Composite 5-20 ft**

Matrix: **Solid**
Lab ID: **23H0932-03**
Date Sampled: **8/7/23 0:00**
Sampled By: **client**

	Result	Units	Minimum Reporting Limit	Method	Preparation Date/Time	Analysis Date/Time	Flag(s)
Inorganic							
Chloride, Soluble (IC)	ND	mg/kg dry	14	EPA 300.0	8/10/23	8/10/23	
pH	5.4	pH Units	0.1	EPA 9045D	8/10/23	8/10/23	
Sulfate, Soluble (IC)	ND	mg/kg dry	14	EPA 300.0	8/10/23	8/10/23	
Total Solids	69.6	%	0.1	CTF8000	8/10/23	8/11/23	
Metals							
Sulfur, Total	145	mg/kg dry	0.18	EPA 6010D	8/11/23	8/14/23	

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PO#:
Receipt: **8/9/23 16:45 @ 24.9** °C
Date Reported: 8/16/2023
Project Name: **6024-308-00 (23-1681)**

Sample ID: **NMB-2 / Composite 5-14 ft**

Matrix: **Solid**
Lab ID: **23H0932-04**
Date Sampled: **8/7/23 0:00**
Sampled By: **client**

	Result	Units	Minimum Reporting Limit	Method	Preparation Date/Time	Analysis Date/Time	Flag(s)
Inorganic							
Chloride, Soluble (IC)	ND	mg/kg dry	18	EPA 300.0	8/10/23	8/10/23	
pH	4.8	pH Units	0.1	EPA 9045D	8/10/23	8/10/23	
Sulfate, Soluble (IC)	ND	mg/kg dry	18	EPA 300.0	8/10/23	8/10/23	
Total Solids	56.5	%	0.1	CTF8000	8/10/23	8/11/23	
Metals							
Sulfur, Total	159	mg/kg dry	0.40	EPA 6010D	8/11/23	8/14/23	

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PO#:
Receipt: **8/9/23 16:45 @ 24.9** °C
Date Reported: 8/16/2023
Project Name: **6024-308-00 (23-1681)**

Sample ID: **NMB-3 / Composite 5-14 ft**
Matrix: **Solid**
Lab ID: **23H0932-05**
Date Sampled: **8/7/23 0:00**
Sampled By: **client**

	Result	Units	Minimum Reporting Limit	Method	Preparation Date/Time	Analysis Date/Time	Flag(s)
Inorganic							
Chloride, Soluble (IC)	ND	mg/kg dry	15	EPA 300.0	8/10/23	8/10/23	
pH	5.1	pH Units	0.1	EPA 9045D	8/10/23	8/10/23	
Sulfate, Soluble (IC)	ND	mg/kg dry	15	EPA 300.0	8/10/23	8/10/23	
Total Solids	65.5	%	0.1	CTF8000	8/10/23	8/11/23	
Metals							
Sulfur, Total	51.3	mg/kg dry	0.43	EPA 6010D	8/11/23	8/14/23	

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PO#:
Receipt: **8/9/23 16:45 @ 24.9** °C
Date Reported: 8/16/2023
Project Name: **6024-308-00 (23-1681)**

Sample ID: **NMB-4 / Composite 5-12.5**

Matrix: **Solid**
Lab ID: **23H0932-06**
Date Sampled: **8/7/23 0:00**
Sampled By: **client**

	Result	Units	Minimum Reporting Limit	Method	Preparation Date/Time	Analysis Date/Time	Flag(s)
Inorganic							
Chloride, Soluble (IC)	ND	mg/kg dry	14	EPA 300.0	8/10/23	8/10/23	
pH	5.4	pH Units	0.1	EPA 9045D	8/10/23	8/10/23	
Sulfate, Soluble (IC)	ND	mg/kg dry	14	EPA 300.0	8/10/23	8/10/23	
Total Solids	70.3	%	0.1	CTF8000	8/10/23	8/11/23	
Metals							
Sulfur, Total	172	mg/kg dry	0.34	EPA 6010D	8/11/23	8/14/23	

Certificate of Analysis

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Zach Gibbs
7657 South Holden Street
Midvale, UT 84047

PO#:
Receipt: **8/9/23 16:45 @ 24.9** °C
Date Reported: 8/16/2023
Project Name: **6024-308-00 (23-1681)**

Report Footnotes

Abbreviations

ND = Not detected at the corresponding Minimum Reporting Limit (MRL).
1 mg/L = one milligram per liter or 1 mg/kg = one milligram per kilogram = 1 part per million.
1 ug/L = one microgram per liter or 1 ug/kg = one microgram per kilogram = 1 part per billion.
1 ng/L = one nanogram per liter or 1 ng/kg = one nanogram per kilogram = 1 part per trillion.
On calculated parameters, there may be a slight difference between summing the rounded values shown on the report vs the unrounded values used in the calculation.

CHEMTECH - FORD ANALYTICAL LABORATORY

CHAIN OF CUSTODY

CHEMTECH-FORD LABORATORIES

COMPANY: Gerhart Cole

ADDRESS: 7657 s. Holden St.

CITY/STATE/ZIP: Midvale, Utah 84047

PHONE #: 801.791.3641 FAX:

CONTACT: Zach Gibbs PROJECT: 6024-308-00 (23-1681)

EMAIL: zachg@gerhartcole.com

BILLING ADDRESS:

BILLING CITY/STATE/ZIP:

PURCHASE ORDER #:

TURNAROUND REQUIRED:*

* Expedited turnaround subject to additional charge

W-

23H0932

Sample Receipt Conditions:

- ( ) Custody Seals Present
- (✓) Containers Intact
- (✓) COC and Labels Match
- (✓) Received on Ice
- (✓) Correct Containers
- (✓) COC Included
- (✓) COC Complete
- (✓) Sufficient Sample Volume
- ( ) Headspace Present (VOC)
- ( ) Temperature Blank
- (✓) Received within Hold

Checked by: AW

Lab Use Only	LOCATION / IDENTIFICATION	DATE	TIME	MATRIX	Field: Residual Chlorine	pH	Sulfates and Chlorides	Total Sulphur	Total Coliform + E. coli (Present/Absent)	Total Coliform + E. coli (Enumerated)	HPC (Plate Count)	E. Coli Only
01	1. MMB-1 / ~~5 &~~ 6.5-8.5 ft	8/7/2023				×	×	×				
02	2. MMB-2 / ~~10 &~~ 11.5-13.5 ft	8/7/2023				×	×	×				
03	3. NMB-1 / Composite 5-20 ft	8/7/2023				×	×	×				
04	4. NMB-2 / Composite 5-14 ft	8/7/2023				×	×	×				
05	5. NMB-3 / Composite 5-14 ft	8/7/2023				×	×	×				
06	6. NMB-4 / Composite 5-12.5	8/7/2023				×	×	×				
	7.											
	8.											
	9.					Q						
	10.											

Sampled by: [print] Sampled by: [signature]

Special Instructions:

ON ICE / NOT ON ICE (circled) Temp (C°): 24.9

Samples received outside the EPA recommended temperature range of 0-6 C° may be rejected.

Relinquished by: [signature]	Date/Time	Received by: [signature]	Date/Time
Alicia Walburger	8/9/23 16:45	[signature]	8/9/23 16:45

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Payment Terms are net 30 days OAC. 1.5% interest charge per month (18% per annum). Client agress to pay collection costs and attorney's fees.

Pace Analytical Services, LLC
9608 Loiret Blvd.
Lenexa, KS 66219
(913)599-5665

August 23, 2023

Zach Gibbs
Gerhart Cole
7657 South Holden Street
Midvale, UT 84047

RE: Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

Dear Zach Gibbs:

Enclosed are the analytical results for sample(s) received by the laboratory on August 10, 2023. The results relate only to the samples included in this report. Results reported herein conform to the applicable TNI/NELAC Standards and the laboratory's Quality Manual, where applicable, unless otherwise noted in the body of the report.

The test results provided in this final report were generated by each of the following laboratories within the Pace Network:

- Pace Analytical Services - Kansas City

If you have any questions concerning this report, please feel free to contact me.

Sincerely,

Heather Wilson
heather.wilson@pacelabs.com
1(913)563-1407
Project Manager

Enclosures

Pace Analytical Services, LLC
9608 Loiret Blvd.
Lenexa, KS 66219
(913)599-5665

CERTIFICATIONS

Project:	MIST RESILIENCY 23-1681
Pace Project No.:	60435134

Pace Analytical Services Kansas

9608 Loiret Boulevard, Lenexa, KS 66219
Missouri Inorganic Drinking Water Certification #: 10090
Arkansas Drinking Water
Arkansas Certification #: 88-00679
Illinois Certification #: 2000302023-5
Iowa Certification #: 118
Kansas/NELAP Certification #: E-10116
Louisiana Certification #: 03055
Nevada Certification #: KS000212023-1
Oklahoma Certification #: 2022-057
Florida: Cert E871149 SEKS WET
Texas Certification #: T104704407-22-16
Utah Certification #: KS000212022-12
Illinois Certification #: 004592
Kansas Field Laboratory Accreditation: # E-92587
Missouri SEKS Micro Certification: 10070

Pace Analytical Services, LLC
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Lenexa, KS 66219
(913)599-5665

SAMPLE SUMMARY

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

Lab ID	Sample ID	Matrix	Date Collected	Date Received
60435134001	**MMB-1**	Solid	08/09/23 08:00	08/10/23 08:50
60435134002	**MMB-2**	Solid	08/09/23 08:00	08/10/23 08:50
60435134003	**NMB-1**	Solid	08/09/23 08:00	08/10/23 08:50
60435134004	**NMB-2**	Solid	08/09/23 08:00	08/10/23 08:50
60435134005	**NMB-3**	Solid	08/09/23 08:00	08/10/23 08:50
60435134006	**NMB-4**	Solid	08/09/23 08:00	08/10/23 08:50

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SAMPLE ANALYTE COUNT

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

Lab ID	Sample ID	Method	Analysts	Analytes Reported	Laboratory
60435134001	**MMB-1**	ASTM D2974	DWC	1	PASI-K
		SM 2320B	BMT	3	PASI-K
		SM 2580B	TML	1	PASI-K
60435134002	**MMB-2**	ASTM D2974	DWC	1	PASI-K
		SM 2320B	BMT	3	PASI-K
		SM 2580B	TML	1	PASI-K
60435134003	**NMB-1**	ASTM D2974	DWC	1	PASI-K
		SM 2320B	BMT	3	PASI-K
		SM 2580B	TML	1	PASI-K
60435134004	**NMB-2**	ASTM D2974	DWC	1	PASI-K
		SM 2320B	BMT	3	PASI-K
		SM 2580B	TML	1	PASI-K
60435134005	**NMB-3**	ASTM D2974	DWC	1	PASI-K
		SM 2320B	BMT	3	PASI-K
		SM 2580B	TML	1	PASI-K
60435134006	**NMB-4**	ASTM D2974	DWC	1	PASI-K
		SM 2320B	BMT	3	PASI-K
		SM 2580B	TML	1	PASI-K

PASI-K = Pace Analytical Services - Kansas City

Pace Analytical Services, LLC
9608 Loiret Blvd.
Lenexa, KS 66219
(913)599-5665

ANALYTICAL RESULTS

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

Sample: MMB-1 **Lab ID: 60435134001** Collected: 08/09/23 08:00 Received: 08/10/23 08:50 Matrix: Solid

Results reported on a "dry weight" basis and are adjusted for percent moisture, sample size and any dilutions.

Parameters	Results	Units	Report Limit	DF	Prepared	Analyzed	CAS No.	Qual
Percent Moisture	Analytical Method: ASTM D2974 Pace Analytical Services - Kansas City							
Percent Moisture	**32.9**	%	0.50	1		08/11/23 10:41		
2320B Alkalinity	Analytical Method: SM 2320B Preparation Method: SM 2320B Pace Analytical Services - Kansas City							
Alkalinity,Bicarbonate (CaCO3)	ND	mg/kg	292	1	08/21/23 07:58	08/21/23 13:40		
Alkalinity,Carbonate (CaCO3)	ND	mg/kg	292	1	08/21/23 07:58	08/21/23 13:40		
Alkalinity, Total as CaCO3	ND	mg/kg	292	1	08/21/23 07:58	08/21/23 13:40		
Oxidation/Reduction Potential	Analytical Method: SM 2580B Pace Analytical Services - Kansas City							
Oxidation/Reduction Potential	**272.2**	mV	1.0	1		08/14/23 15:07		

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Lenexa, KS 66219
(913)599-5665

ANALYTICAL RESULTS

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

Sample: MMB-2 **Lab ID: 60435134002** Collected: 08/09/23 08:00 Received: 08/10/23 08:50 Matrix: Solid

Results reported on a "dry weight" basis and are adjusted for percent moisture, sample size and any dilutions.

Parameters	Results	Units	Report Limit	DF	Prepared	Analyzed	CAS No.	Qual
Percent Moisture	Analytical Method: ASTM D2974 Pace Analytical Services - Kansas City							
Percent Moisture	**37.2**	%	0.50	1		08/11/23 10:41		
2320B Alkalinity	Analytical Method: SM 2320B Preparation Method: SM 2320B Pace Analytical Services - Kansas City							
Alkalinity,Bicarbonate (CaCO3)	ND	mg/kg	304	1	08/21/23 07:58	08/21/23 13:48		
Alkalinity,Carbonate (CaCO3)	ND	mg/kg	304	1	08/21/23 07:58	08/21/23 13:48		
Alkalinity, Total as CaCO3	ND	mg/kg	304	1	08/21/23 07:58	08/21/23 13:48		
Oxidation/Reduction Potential	Analytical Method: SM 2580B Pace Analytical Services - Kansas City							
Oxidation/Reduction Potential	**221.4**	mV	1.0	1		08/14/23 15:23		

REPORT OF LABORATORY ANALYSIS

Pace Analytical Services, LLC
9608 Loiret Blvd.
Lenexa, KS 66219
(913)599-5665

ANALYTICAL RESULTS

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

Sample: NMB-1 **Lab ID: 60435134003** Collected: 08/09/23 08:00 Received: 08/10/23 08:50 Matrix: Solid

Results reported on a "dry weight" basis and are adjusted for percent moisture, sample size and any dilutions.

Parameters	Results	Units	Report Limit	DF	Prepared	Analyzed	CAS No.	Qual
Percent Moisture	Analytical Method: ASTM D2974 Pace Analytical Services - Kansas City							
Percent Moisture	**26.5**	%	0.50	1		08/11/23 10:41		
2320B Alkalinity	Analytical Method: SM 2320B Preparation Method: SM 2320B Pace Analytical Services - Kansas City							
Alkalinity,Bicarbonate (CaCO3)	ND	mg/kg	266	1	08/21/23 07:58	08/21/23 13:51		
Alkalinity,Carbonate (CaCO3)	ND	mg/kg	266	1	08/21/23 07:58	08/21/23 13:51		
Alkalinity, Total as CaCO3	ND	mg/kg	266	1	08/21/23 07:58	08/21/23 13:51		
Oxidation/Reduction Potential	Analytical Method: SM 2580B Pace Analytical Services - Kansas City							
Oxidation/Reduction Potential	**236.8**	mV	1.0	1		08/14/23 15:31		

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ANALYTICAL RESULTS

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

Sample: NMB-2 **Lab ID: 60435134004** Collected: 08/09/23 08:00 Received: 08/10/23 08:50 Matrix: Solid

Results reported on a "dry weight" basis and are adjusted for percent moisture, sample size and any dilutions.

Parameters	Results	Units	Report Limit	DF	Prepared	Analyzed	CAS No.	Qual
Percent Moisture	Analytical Method: ASTM D2974 Pace Analytical Services - Kansas City							
Percent Moisture	**42.7**	%	0.50	1		08/11/23 10:41		
2320B Alkalinity	Analytical Method: SM 2320B Preparation Method: SM 2320B Pace Analytical Services - Kansas City							
Alkalinity,Bicarbonate (CaCO3)	ND	mg/kg	327	1	08/21/23 07:58	08/21/23 13:54		
Alkalinity,Carbonate (CaCO3)	ND	mg/kg	327	1	08/21/23 07:58	08/21/23 13:54		
Alkalinity, Total as CaCO3	ND	mg/kg	327	1	08/21/23 07:58	08/21/23 13:54		
Oxidation/Reduction Potential	Analytical Method: SM 2580B Pace Analytical Services - Kansas City							
Oxidation/Reduction Potential	**269.8**	mV	1.0	1		08/14/23 15:35		

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ANALYTICAL RESULTS

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

Sample: NMB-3 **Lab ID: 60435134005** Collected: 08/09/23 08:00 Received: 08/10/23 08:50 Matrix: Solid

Results reported on a "dry weight" basis and are adjusted for percent moisture, sample size and any dilutions.

Parameters	Results	Units	Report Limit	DF	Prepared	Analyzed	CAS No.	Qual
Percent Moisture	Analytical Method: ASTM D2974 Pace Analytical Services - Kansas City							
Percent Moisture	**39.7**	%	0.50	1		08/11/23 10:42		
2320B Alkalinity	Analytical Method: SM 2320B Preparation Method: SM 2320B Pace Analytical Services - Kansas City							
Alkalinity,Bicarbonate (CaCO3)	ND	mg/kg	333	1	08/21/23 07:58	08/21/23 13:57		
Alkalinity,Carbonate (CaCO3)	ND	mg/kg	333	1	08/21/23 07:58	08/21/23 13:57		
Alkalinity, Total as CaCO3	ND	mg/kg	333	1	08/21/23 07:58	08/21/23 13:57		
Oxidation/Reduction Potential	Analytical Method: SM 2580B Pace Analytical Services - Kansas City							
Oxidation/Reduction Potential	**258.1**	mV	1.0	1		08/14/23 15:36		

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ANALYTICAL RESULTS

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

Sample: NMB-4 **Lab ID: 60435134006** Collected: 08/09/23 08:00 Received: 08/10/23 08:50 Matrix: Solid

Results reported on a "dry weight" basis and are adjusted for percent moisture, sample size and any dilutions.

Parameters	Results	Units	Report Limit	DF	Prepared	Analyzed	CAS No.	Qual
Percent Moisture	Analytical Method: ASTM D2974 Pace Analytical Services - Kansas City							
Percent Moisture	**30.1**	%	0.50	1		08/11/23 10:42		
2320B Alkalinity	Analytical Method: SM 2320B Preparation Method: SM 2320B Pace Analytical Services - Kansas City							
Alkalinity,Bicarbonate (CaCO3)	**295**	mg/kg	273	1	08/21/23 07:58	08/21/23 14:00		
Alkalinity,Carbonate (CaCO3)	ND	mg/kg	273	1	08/21/23 07:58	08/21/23 14:00		
Alkalinity, Total as CaCO3	**295**	mg/kg	273	1	08/21/23 07:58	08/21/23 14:00		
Oxidation/Reduction Potential	Analytical Method: SM 2580B Pace Analytical Services - Kansas City							
Oxidation/Reduction Potential	**256.1**	mV	1.0	1		08/14/23 15:39		

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QUALITY CONTROL DATA

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

QC Batch: 860144 | Analysis Method: ASTM D2974
QC Batch Method: ASTM D2974 | Analysis Description: Dry Weight/Percent Moisture
Laboratory: Pace Analytical Services - Kansas City

Associated Lab Samples: 60435134001, 60435134002, 60435134003, 60435134004, 60435134005, 60435134006

METHOD BLANK: 3405992 | Matrix: Solid

Associated Lab Samples: 60435134001, 60435134002, 60435134003, 60435134004, 60435134005, 60435134006

Parameter	Units	Blank Result	Reporting Limit	Analyzed	Qualifiers
Percent Moisture	%	ND	0.50	08/11/23 10:41	

SAMPLE DUPLICATE: 3405993

Parameter	Units	60435134001 Result	Dup Result	RPD	Max RPD	Qualifiers
Percent Moisture	%	32.9	33.1	1	20	

Results presented on this page are in the units indicated by the "Units" column except where an alternate unit is presented to the right of the result.

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QUALITY CONTROL DATA

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

QC Batch: 861257
QC Batch Method: SM 2320B
Analysis Method: SM 2320B
Analysis Description: 2320BS Analysis
Laboratory: Pace Analytical Services - Kansas City
Associated Lab Samples: 60435134001, 60435134002, 60435134003, 60435134004, 60435134005, 60435134006

METHOD BLANK: 3410620 Matrix: Solid
Associated Lab Samples: 60435134001, 60435134002, 60435134003, 60435134004, 60435134005, 60435134006

Parameter	Units	Blank Result	Reporting Limit	Analyzed	Qualifiers
Alkalinity, Total as CaCO3	mg/kg	ND	200	08/21/23 13:31	
Alkalinity,Bicarbonate (CaCO3)	mg/kg	ND	200	08/21/23 13:31	
Alkalinity,Carbonate (CaCO3)	mg/kg	ND	200	08/21/23 13:31	

LABORATORY CONTROL SAMPLE: 3410621

Parameter	Units	Spike Conc.	LCS Result	LCS % Rec	% Rec Limits	Qualifiers
Alkalinity, Total as CaCO3	mg/kg	5000	5030	101	90-110	

SAMPLE DUPLICATE: 3410622

Parameter	Units	60435134001 Result	Dup Result	RPD	Max RPD	Qualifiers
Alkalinity, Total as CaCO3	mg/kg	ND	ND		20	
Alkalinity,Bicarbonate (CaCO3)	mg/kg	ND	ND		20	
Alkalinity,Carbonate (CaCO3)	mg/kg	ND	ND		20	

Results presented on this page are in the units indicated by the "Units" column except where an alternate unit is presented to the right of the result.

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QUALITY CONTROL DATA

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

QC Batch: 860452
QC Batch Method: SM 2580B
Analysis Method: SM 2580B
Analysis Description: Oxidation/Reduction Potential
Laboratory: Pace Analytical Services - Kansas City
Associated Lab Samples: 60435134001, 60435134002, 60435134003, 60435134004, 60435134005, 60435134006

SAMPLE DUPLICATE: 3407377

Parameter	Units	60434923001 Result	Dup Result	RPD	Max RPD	Qualifiers
Oxidation/Reduction Potential	mV	201.6	201.6			

Results presented on this page are in the units indicated by the "Units" column except where an alternate unit is presented to the right of the result.

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QUALIFIERS

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

DEFINITIONS

DF - Dilution Factor, if reported, represents the factor applied to the reported data due to dilution of the sample aliquot.

ND - Not Detected at or above adjusted reporting limit.

TNTC - Too Numerous To Count

J - Estimated concentration above the adjusted method detection limit and below the adjusted reporting limit.

MDL - Adjusted Method Detection Limit.

PQL - Practical Quantitation Limit.

RL - Reporting Limit - The lowest concentration value that meets project requirements for quantitative data with known precision and bias for a specific analyte in a specific matrix.

S - Surrogate

1,2-Diphenylhydrazine decomposes to and cannot be separated from Azobenzene using Method 8270. The result for each analyte is a combined concentration.

Consistent with EPA guidelines, unrounded data are displayed and have been used to calculate % recovery and RPD values.

LCS(D) - Laboratory Control Sample (Duplicate)

MS(D) - Matrix Spike (Duplicate)

DUP - Sample Duplicate

RPD - Relative Percent Difference

NC - Not Calculable.

SG - Silica Gel - Clean-Up

U - Indicates the compound was analyzed for, but not detected.

N-Nitrosodiphenylamine decomposes and cannot be separated from Diphenylamine using Method 8270. The result reported for each analyte is a combined concentration.

Reported results are not rounded until the final step prior to reporting. Therefore, calculated parameters that are typically reported as "Total" may vary slightly from the sum of the reported component parameters.

Pace Analytical is TNI accredited. Contact your Pace PM for the current list of accredited analytes.

TNI - The NELAC Institute.

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QUALITY CONTROL DATA CROSS REFERENCE TABLE

Project: MIST RESILIENCY 23-1681
Pace Project No.: 60435134

Lab ID	Sample ID	QC Batch Method	QC Batch	Analytical Method	Analytical Batch
60435134001	**MMB-1**	ASTM D2974	860144		
60435134002	**MMB-2**	ASTM D2974	860144		
60435134003	**NMB-1**	ASTM D2974	860144		
60435134004	**NMB-2**	ASTM D2974	860144		
60435134005	**NMB-3**	ASTM D2974	860144		
60435134006	**NMB-4**	ASTM D2974	860144		
60435134001	**MMB-1**	SM 2320B	861257	SM 2320B	861388
60435134002	**MMB-2**	SM 2320B	861257	SM 2320B	861388
60435134003	**NMB-1**	SM 2320B	861257	SM 2320B	861388
60435134004	**NMB-2**	SM 2320B	861257	SM 2320B	861388
60435134005	**NMB-3**	SM 2320B	861257	SM 2320B	861388
60435134006	**NMB-4**	SM 2320B	861257	SM 2320B	861388
60435134001	**MMB-1**	SM 2580B	860452		
60435134002	**MMB-2**	SM 2580B	860452		
60435134003	**NMB-1**	SM 2580B	860452		
60435134004	**NMB-2**	SM 2580B	860452		
60435134005	**NMB-3**	SM 2580B	860452		
60435134006	**NMB-4**	SM 2580B	860452		

Pace ANALYTICAL SERVICES

DC#_Title: ENV-FRM-LENE-0009_Sample Condition Upon Receipt (SCUR)		
Revision: 2	Effective Date: 01/12/2022	Issued By: Lenexa

Client Name: Gerhart Cole

Courier: FedEx ☑ UPS ☐ VIA ☐ Clay ☐ PEX ☐ ECI ☐ Pace ☐ Xroads ☐ Client ☐ Other ☐

Tracking #: 7822 8933 2096 Pace Shipping Label Used? Yes ☐ No ☑

Custody Seal on Cooler/Box Present: Yes ☑ No ☐ Seals intact: Yes ☑ No ☐

Packing Material: Bubble Wrap ☑ Bubble Bags ☐ Foam ☐ None ☐ Other ☐

Thermometer Used: T249 **Type of Ice:** Wet Blue None

Cooler Temperature (°C): As-read 2.0 **Corr. Factor** +0.2 **Corrected** 2.2

Date and initials of person examining contents: AF 8/10

Temperature should be above freezing to 6°C

Chain of Custody present:	☑Yes ☐No ☐N/A	
Chain of Custody relinquished:	☑Yes ☐No ☐N/A	
Samples arrived within holding time:	☑Yes ☐No ☐N/A	
Short Hold Time analyses (<72hr):	☐Yes ☑No ☐N/A	
Rush Turn Around Time requested:	☐Yes ☑No ☐N/A	
Sufficient volume:	☑Yes ☐No ☐N/A	
Correct containers used:	☑Yes ☐No ☐N/A	
Pace containers used:	☐Yes ☑No ☐N/A	
Containers intact:	☑Yes ☐No ☐N/A	
Unpreserved 5035A / TX1005/1006 soils frozen in 48hrs?	☐Yes ☐No ☑N/A	
Filtered volume received for dissolved tests?	☐Yes ☐No ☑N/A	
Sample labels match COC: Date / time / ID / analyses	☑Yes ☐No ☐N/A	
Samples contain multiple phases? Matrix: SL	☐Yes ☑No ☐N/A	
Containers requiring pH preservation in compliance? (HNO_3, H_2SO_4, HCl<2; NaOH>9 Sulfide, NaOH>10 Cyanide) (Exceptions: VOA, Micro, O&G, KS TPH, OK-DRO) **LOT#:**	☐Yes ☐No ☑N/A	List sample IDs, volumes, lot #'s of preservative and the date/time added.
Cyanide water sample checks: Lead acetate strip turns dark? (Record only) Potassium iodide test strip turns blue/purple? (Preserve)	☐Yes ☐No ☐Yes ☐No	
Trip Blank present:	☐Yes ☐No ☑N/A	
Headspace in VOA vials (>6mm):	☐Yes ☐No ☑N/A	
Samples from USDA Regulated Area: State:	☐Yes ☑No ☐N/A	
Additional labels attached to 5035A / TX1005 vials in the field?	☐Yes ☐No ☑N/A	

Client Notification/ Resolution: Copy COC to Client? Y / N Field Data Required? Y / N

Person Contacted: ______________ Date/Time: ______________

Comments/ Resolution: ______________

Project Manager Review: ______________ Date: ______________

Chain of Custody

GC Set Number 23-1681

Company Name: Gerhart Cole
Project Name: Mist Resiliency
Project Number: 23-1681
Company Address: 7657 South Holden Street, Midvale, Utah 84047
Contact Name: Zach Gibbs
Phone Number: 801-791-3641
Email: zachg@gerhartcole.com

GERHART COLE
7657 South Holden Street
Midvale, Utah 84047
(801) 849-0055

Relinquished By: (Signature)	Date 8/9/23	Time 15:00	Received By: (Signature)	Date 8/9/23	Time 15:00
Relinquished By: (Print) Zach Gibbs PISCCUT	8/9/23	17:00	Received By: (Print)	8/10	850

2.2

60173434

Test Hole / Test Pit #	Depth Range (ft)	Sample Type	Sample Idendificatio	Redox	Carbonates	Bicarbonates				Special instructions / comments
MMB-1	6.5-8.5	SH	-	1	1	1				Need carbonates and bicarbonates results separate on all samples
MMB-2	11.5-13.5	SH	-	1	1	1				
NMB-1	5-20	SPT/D&M	-	1	1	1				
NMB-2	5-14	SPT/D&M	-	1	1	1				
NMB-3	5-14	SPT/D&M	-	1	1	1				
NMB-4	5-12.5	SPT/D&M	-	1	1	1				

Client: Gerhart Cole

Profile # 15655-1

Site: Mist Resiliency

Notes

COC Line Item	Matrix	VG9H	DG9H	DG9Q	VG9U	DG9U	DG9M	DG9B	BG1U	AG1H	AG1U	AG2U	AG3S	AG4U	AG5U	JGFU	WGKU	WGDU	BP1U	BP2U	BP3U	BP1N	BP3N	BP3F	BP3S	BP3C	BP3Z	WPDU	ZPLC	Other		
1	SL																												1			
2																													1			
3																													1			
4																													1			
5																													1			
6																													1			
7																																
8																																
9																																
10																																
11																																
12																																

Container Codes

Glass			
DG9B	40mL bisulfate clear vial	WGKU	8oz clear soil jar
DG9H	40mL HCl amber voa vial	WGFU	4oz clear soil jar
DG9M	40mL MeOH clear vial	WG2U	2oz clear soil jar
DG9Q	40mL TSP amber vial	JGFU	4oz unpreserved amber wide
DG9S	40mL H2SO4 amber vial	AG0U	100mL unores amber glass
DG9T	40mL Na Thio amber vial	AG1H	1L HCl amber glass
DG9U	40mL amber unpreserved	AG1S	1L H2SO4 amber glass
VG9H	40mL HCl clear vial	AG1T	1L Na Thiosulfate clear/amber glass
VG9T	40mL Na Thio. clear vial	AG1U	1liter unpres amber glass
VG9U	40mL unpreserved clear vial	AG2N	500mL HNO3 amber glass
BG1S	1liter H2SO4 clear glass	AG2S	500mL H2SO4 amber glass
BG1U	1liter unpres glass	AG3S	250mL H2SO4 amber glass
BG3H	250mL HCL Clear glass	AG2U	500mL unpres amber glass
BG3U	250mL Unpres Clear glass	AG3U	250mL unpres amber glass
WGDU	16oz clear soil jar	AG4U	125mL unpres amber glass
		AG5U	100mL unpres amber glass

Plastic	
BP1C	1L NAOH plastic
BP1N	1L HNO3 plastic
BP1S	1L H2SO4 plastic
BP1U	1L unpreserved plastic
BP1Z	1L NaOH, Zn Acetate
BP2C	500mL NAOH plastic
BP2N	500mL HNO3 plastic
BP2S	500mL H2SO4 plastic
BP2U	500mL unpreserved plastic
BP2Z	500mL NaOH, Zn Acetate
BP3C	250mL NaOH plastic
BP3F	250mL HNO3 plastic - field filtered
BP3N	250mL HNO3 plastic
BP3U	250mL unpreserved plastic
BP3S	250mL H2SO4 plastic
BP3Z	250mL NaOH, Zn Acetate
BP4U	125mL unpreserved plastic
BP4N	125mL HNO3 plastic
BP4S	125mL H2SO4 plastic
WPDU	16oz unpresserved plstic

Misc.	
I	Wipe/Swab
SP5T	120mL Coliform Na Thiosulfate
ZPLC	Ziploc Bag
AF	Air Filter
C	Air Cassettes
R	Terracore Kit
U	Summa Can

Matrix	
WT	Water
SL	Solid
NAL	Non-aqueous Liquid
OL	OIL
WP	Wipe
DW	Drinking Water

Work Order Number: 60435134

APPENDIX B
Downhole Seismic Testing Results

Report on

Downhole Seismic Explorations Northwest Natural Gas North Mist Compressor Station

Mist, Oregon

Report Date: July 24, 2023

Prepared for:

GeoEngineers, Inc
4000 Kruse Way Place
Lake Oswego, OR 97035

Prepared by:

EARTH DYNAMICS LLC
2284 N.W. Thurman St.
Portland, OR 97210
(503) 227-7659
Project No. 23215

1.0 INTRODUCTION

This report presents the results of downhole seismic explorations at the Northwest Natural Gas North Mist Compressor station near Mist, Oregon. This work was requested and authorized by Jerad Hoffman of GeoEngineers. The exploration consisted of downhole measurements of compressional and shear wave velocity at three locations.

The geophysical data were acquired on June 20 and 21, 2023 under the supervision of Mr. Daniel Lauer of Earth Dynamics LLC. This report describes the methodology and results of the geophysical investigation.

2.0 METHOD

2.1 Downhole Seismic

In a downhole seismic survey, a seismic source is placed on the ground surface near a borehole, and two geophone assemblies are placed at selected depths in the borehole. The test boring is prepared by installing three-inch I.D. flush-joint PVC casing. The PVC casing is grouted in place to provide the required seismic coupling between the casing and the surrounding formation. The raw data obtained from a downhole survey are the travel times for compressional (P-Wave) and shear (S-Wave) waves from the source to the geophones and the distance between the source and geophones. All depths are measured from the top of the casing.

Seismic waves with a large compressional wave component are produced by striking a steel plate on the ground with a sledge hammer. The plate is located five feet from the boring. Shear-waves travel slower than compressional waves. Therefore, shear wave signals are often obscured by the compressional wave signal. This interference sometimes makes identification of the first shear-wave arrival difficult. To improve the resolution of the shear-wave arrival, the seismic source is designed to produce a signal that contains a large shear-wave component while minimizing the compressional wave signal. A signal enhancement seismograph is used to process the signals received from the geophones. The shear-wave source for this study consisted of sledge hammer impacts on alternate ends of an 8"x10"x8' wooden beam with aluminum end plates. The beam was coupled to the ground by weighing it down with the front tires of the field recording truck. The beam was offset a distance of five feet from the borehole to minimize direct coupling of the seismic energy to the casing.

The downhole sensors consisted of two Bison Instruments Type 1462 triaxial geophone assemblies. Each assembly contains three sensing elements: one vertical and two orthogonal horizontal geophones. A distance of ten feet separated the two geophone assemblies within the borehole. Two geophone assemblies at a fixed separation are used so that interval velocities can be determined from the same set of impulses. The use of two geophone assemblies provides at least two compressional and two shear-wave travel times at each measurement level. This method reduces timing errors caused by differences in seismic triggering and variations in source impulse characteristics. In this study, travel time values are accurate to ±1 millisecond (ms).

The recording procedure consists of placing the geophone assemblies at the desired depths in the borehole. The geophone assemblies are locked to the inside of the casing wall by inflating a pneumatic rubber packer. A Seismic Source, Inc. DAQ Link IV signal enhancement seismograph and laptop computer are used to record signals from the geophones. The travel times are determined in the field and the data are checked for consistency before proceeding to the next measurement depth.

The data are analyzed by plotting the overall travel time versus distance. These plots are commonly referred to as travel-time plots. Linear regression analysis is used to compute line segments joining data points of similar slope. The slope of the line segment is proportional to the average velocity of the material within the depth corresponding to the line segment.

Downhole shear wave data were acquired in three borings. The borings are identified as MMB-1, NMB-1 and NMB-3. According to the drilling records, NMB-1 and NMB-3 were cased to an approximate depth of 100 feet below the ground surface (bgs) and MMB-1 was cased to an approximate depth of 80 feet bgs. Velocity measurements were obtained every five feet from the ground surface to bottom of the casing in each boring. The bottom of MMB-1 was encountered at a depth of 78 feet bgs, the bottom of NMB-1 was encountered at a depth of 98 feet bgs, and the bottom of NMB-3 was encountered at a depth of 100 feet bgs.

3.0 RESULTS

The results of the downhole seismic study are contained in Figures 3-1 through 3-3. P-wave and S-wave travel-times from the downhole test at Borings MMB-1, NMB-1, and NMB-3 are plotted in Figures 3-1, 3-2, and 3-3 respectively. Linear regression of the travel-time data in the figures are used to determine the average P and S-wave velocities of the subsurface material. The velocities are corrected to compensate for the offset of the seismic source from the borehole. Table 3-1 summarizes seismic velocity as a function of depth for the downhole measurements.

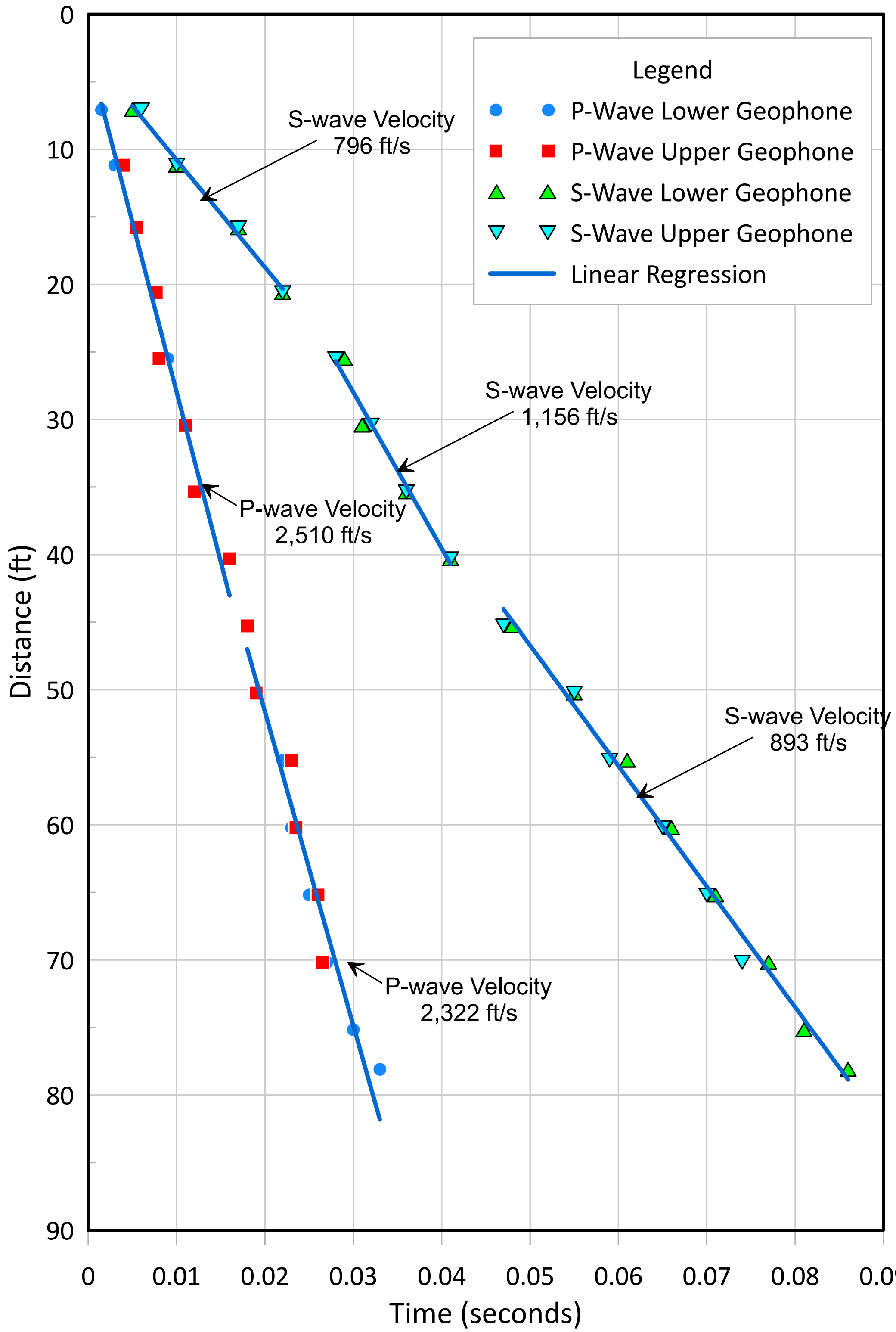

Figure 3-1. Seismic Velocity Travel Time Plot for Boring MMB-1.

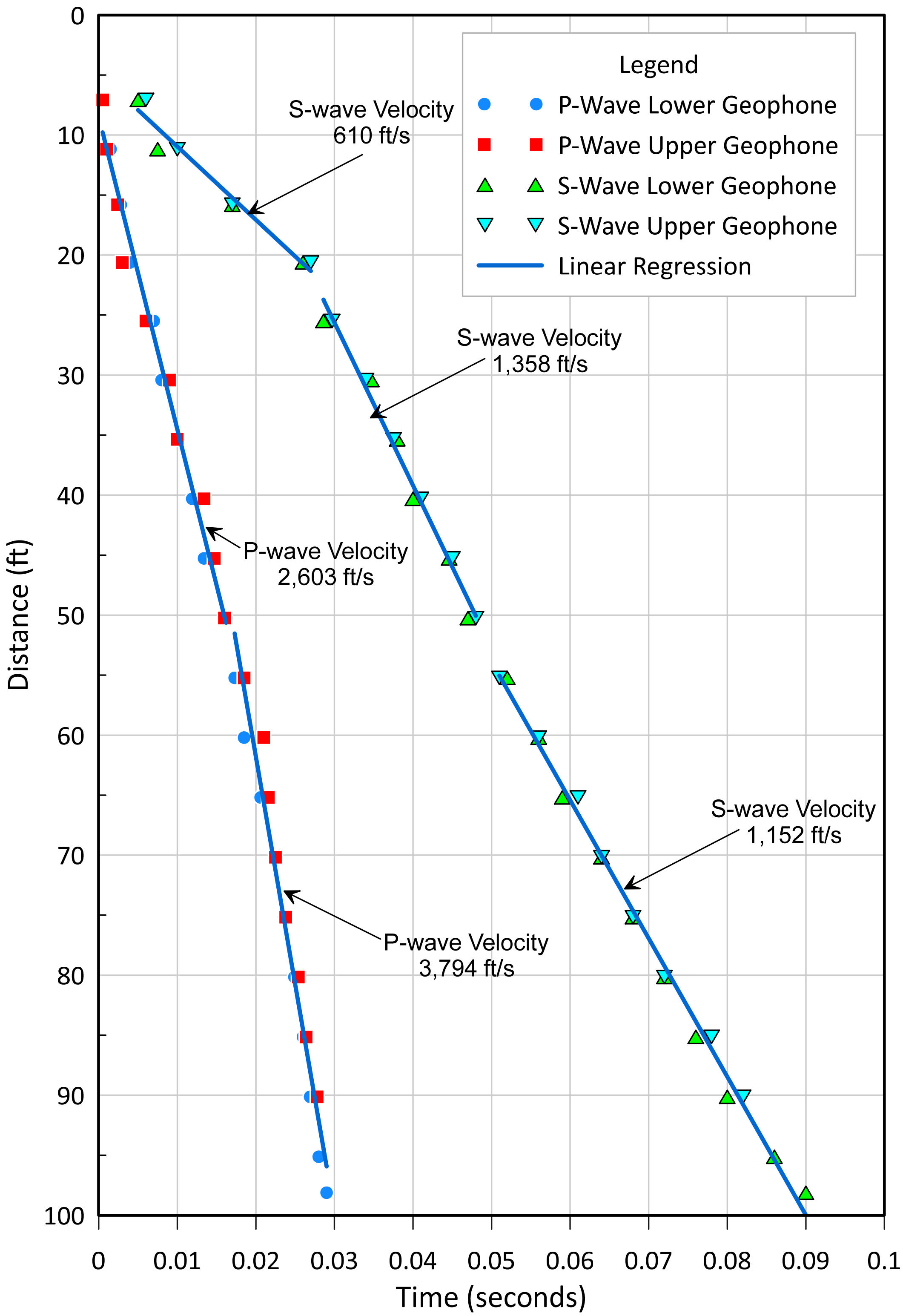

Figure 3-2. Seismic Velocity Travel Time Plot for Boring NMB-1.

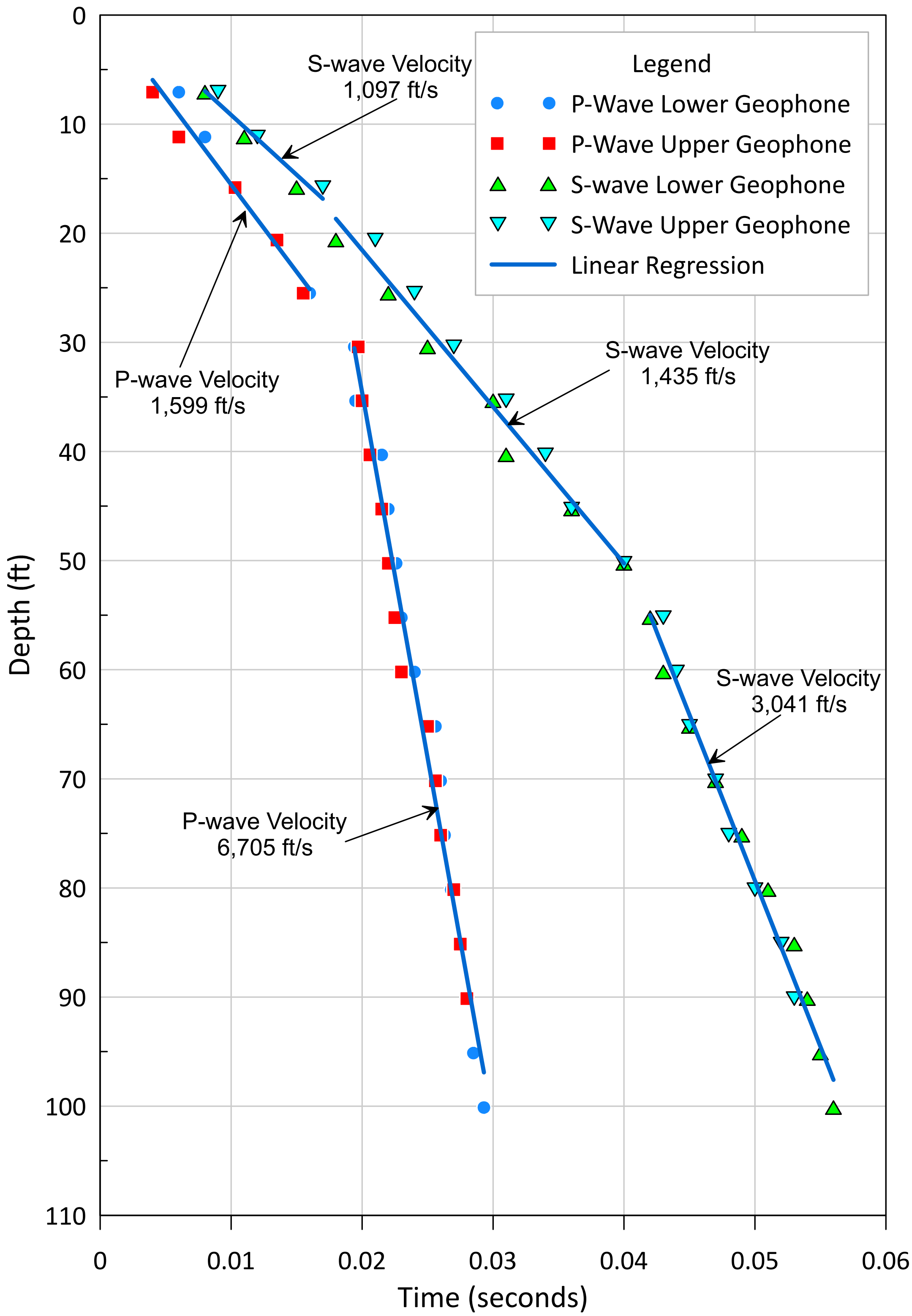

Figure 3-3. Seismic Velocity Travel Time Plot for Boring NMB-3.

Table 3-1 Summary of Seismic Velocity from Downhole Measurements.

Boring	Depth Range (ft)	P-Wave Velocity (ft/s)	Depth Range (ft)	S-Wave Velocity (ft/s)
MMB-1	0 - 45 45 - 78	2,510 2,322	0 - 25 25 - 45 45 - 78	796 1,156 893
NMB-1	0 - 50 50 - 98	2,603 3,794	0 - 20 20 - 50 50 – 98	610 1,358 1,152
NMB-3	0 – 30 30 – 100	1,599 6,705	0 – 15 15 – 50 50 – 100	1,097 1,435 3,041

4.0 DISCUSSION

4.1 Boring MMB-1

Boring MMB-1 is located in the middle of an active compressor station (Miller Station). Vibrations from nearby equipment such as motors and fans as well as active construction work made picking of the downhole first arrivals for both the P-waves and S-waves difficult.

Data from MMB-1 draft boring log indicate that the boring encountered the following subsurface material:

- 0 – 5 feet bgs: Poorly graded Fill
- 5 – 52 feet bgs: Fat clay with increasing stiffness with depth
- 52 – 68 feet bgs: Medium dense Silty Sand.
- 68 – 80 feet bgs: Very stiff Sandy Silt.

It appears that there is moderately good correlation between the downhole seismic velocity data and the boring logs for Boring MMB-1.

4.2 Boring NMB-1

Boring MMB-1 is located approximately 200 feet north of an active compressor station (North Mist Station). Vibrations from nearby equipment such as motors and fans are much lower than that of the Miller Station boring. Picking of the downhole first arrivals for both the P-waves and S-waves is moderately difficult.

Data from NMB-1 draft boring log indicate that the boring encountered the following subsurface material:

- 0 – 5 feet bgs: Organic Fill
- 5 – 19 feet bgs: Stiff Silt with Sand
- 19 – 33 feet bgs: Stiff Silt
- 33 – 47 feet bgs: Weathered Columbia River Basalt
- 47 – 53 feet bgs: Stiff Silt
- 53 – 100 feet bgs: Medium Dense Sand with Silt

It appears that there is moderately good correlation between the downhole seismic velocity data and the boring logs for Boring NMB-1.

4.2 Boring NMB-3

Boring MMB-1 is located approximately 200 feetnorth of an active compressor station (North Mist Station). Vibrations from nearby equipment such as motors and fans are much lower than that of the Miller Station boring. Picking of the downhole first arrivals for both the P-waves and S-waves is moderately difficult.

Data from NMB-3 draft boring log indicate that the boring encountered the following subsurface material:

- 0 – 9 feet bgs: Organic Fill
- 9 – 24 feet bgs: Medium Dense to Dense Silty Sand
- 24 – 43 feet bgs: Weathered Columbia River Basalt
- 43 – 100 feet bgs: Columbia River Basalt

It appears that there is moderately good correlation between the downhole seismic velocity data and the boring logs for Boring NMB-3.

4.0 LIMITATIONS

We have presented models and interpretations which we believe to be the best fit given the geology and known conditions at the site. However, no warranty is made or intended by this report or by oral or written presentation of this work. Earth Dynamics accepts no responsibility for damages because of decisions made or actions taken based upon this report.

RESPECTFULLY SUBMITTED
EARTH DYNAMICS LLC

Daniel Lauer, M.S.
Partner

APPENDIX C
Report Limitations and Guidelines for Use

APPENDIX C
REPORT LIMITATIONS AND GUIDELINES FOR USE[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

Read These Provisions Closely

It is important to recognize that the geoscience practices (geotechnical engineering, geology and environmental science) rely on professional judgment and opinion to a greater extent than other engineering and natural science disciplines, where more precise and/or readily observable data may exist. To help clients better understand how this difference pertains to our services, GeoEngineers includes the following explanatory "limitations" provisions in its reports. Please confer with GeoEngineers if you need to know more how these "Report Limitations and Guidelines for Use" apply to your project or site.

Geotechnical Services are Performed for Specific Purposes, Persons and Projects

This report has been prepared for NW Natural and for the Project(s) specifically identified in the report. The information contained herein is not applicable to other sites or projects.

GeoEngineers structures its services to meet the specific needs of its clients. No party other than the party to whom this report is addressed may rely on the product of our services unless we agree to such reliance in advance and in writing. Within the limitations of the agreed scope of services for the Project, and its schedule and budget, our services have been executed in accordance with our Agreement with NW Natural dated May 10, 2023 and executed May 12, 2023 and generally accepted geotechnical practices in this area at the time this report was prepared. We do not authorize, and will not be responsible for, the use of this report for any purposes or projects other than those identified in the report.

A Geotechnical Engineering or Geologic Report is based on a Unique Set of Project-Specific Factors

This report has been prepared for the proposed Miller Station Resiliency Area as part of NW Natural's Mist Resiliency project. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, it is important not to rely on this report if it was:

- Not prepared for you,
- Not prepared for your project,
- Not prepared for the specific site explored, or
- Completed before important project changes were made.

For example, changes that can affect the applicability of this report include those that affect:

- The function of the proposed structure;

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1 Developed based on material provided by GBA, GeoProfessional Business Association; www.geoprofessional.org.

- Elevation, configuration, location, orientation or weight of the proposed structure;
- Composition of the design team; or
- Project ownership.

If changes occur after the date of this report, GeoEngineers cannot be responsible for any consequences of such changes in relation to this report unless we have been given the opportunity to review our interpretations and recommendations. Based on that review, we can provide written modifications or confirmation, as appropriate.

Environmental Concerns are Not Covered

Unless environmental services were specifically included in our scope of services, this report does not provide any environmental findings, conclusions, or recommendations, including but not limited to, the likelihood of encountering underground storage tanks or regulated contaminants.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by man-made events such as construction on or adjacent to the site, new information or technology that becomes available subsequent to the report date, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. If more than a few months have passed since issuance of our report or work product, or if any of the described events may have occurred, please contact GeoEngineers before applying this report for its intended purpose so that we may evaluate whether changed conditions affect the continued reliability or applicability of our conclusions and recommendations.

Geotechnical and Geologic Findings are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies the specific subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied its professional judgment to render an informed opinion about subsurface conditions at other locations. Actual subsurface conditions may differ, sometimes significantly, from the opinions presented in this report. Our report, conclusions and interpretations are not a warranty of the actual subsurface conditions.

Geotechnical Engineering Report Recommendations are Not Final

We have developed the following recommendations based on data gathered from subsurface investigation(s). These investigations sample just a small percentage of a site to create a snapshot of the subsurface conditions elsewhere on the site. Such sampling on its own cannot provide a complete and accurate view of subsurface conditions for the entire site. Therefore, the recommendations included in this report are preliminary and should not be considered final. GeoEngineers' recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for the recommendations in this report if we do not perform construction observation.

We recommend that you allow sufficient monitoring, testing and consultation during construction by GeoEngineers to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes if the conditions revealed during the work differ from those anticipated, and to evaluate whether earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective means of managing the risks associated with unanticipated conditions. If another party performs field observation and confirms our expectations, the other party must take full responsibility for both the observations and recommendations. Please note, however, that another party would lack our project-specific knowledge and resources.

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by members of the design team or by contractors can result in costly problems. GeoEngineers can help reduce the risks of misinterpretation by conferring with appropriate members of the design team after submitting the report, reviewing pertinent elements of the design team's plans and specifications, participating in pre-bid and preconstruction conferences, and providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. The logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Photographic or electronic reproduction is acceptable, but separating logs from the report can create a risk of misinterpretation.

Give Contractors a Complete Report and Guidance

To help reduce the risk of problems associated with unanticipated subsurface conditions, GeoEngineers recommends giving contractors the complete geotechnical engineering or geologic report, including these "Report Limitations and Guidelines for Use." When providing the report, you should preface it with a clearly written letter of transmittal that:

- Advises contractors that the report was not prepared for purposes of bid development and that its accuracy is limited; and
- Encourages contractors to conduct additional study to obtain the specific types of information they need or prefer.

Contractors are Responsible for Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor's procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and adjacent properties.

Biological Pollutants

GeoEngineers' Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants, and no conclusions or inferences should be drawn regarding Biological Pollutants as they may relate to this project. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria and viruses, and/or any of their byproducts.

A Client that desires these specialized services is advised to obtain them from a consultant who offers services in this specialized field.

Information Provided by Others

GeoEngineers has relied upon certain data or information provided or compiled by others in the performance of our services. Although we use sources that we reasonably believe to be trustworthy, GeoEngineers cannot warrant or guarantee the accuracy or completeness of information provided or compiled by others.

GeoEngineers

Attachment H-5.
OWRD Well Logs

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1-COLU 990

COPY

6N/5W/14

STATE OF OREGON
WATER SUPPLY WELL REPORT
(as required by ORS 537.765)
Instructions for completing this report are on the last page of this form.

(START CARD) # 73652

RECEIVED FEB 14 1995 WATER RESOURCES DEPT. SALEM, OREGON

(1) OWNER: Well Number #2
Name ALITA DAVIS
Address 627 G.P. ERICKSON RD
City CLATSKANIE State OR Zip 97016

(2) TYPE OF WORK
[X] New Well [ ] Deepening [ ] Alteration (repair/recondition) [ ] Abandonment

(3) DRILL METHOD:
[X] Rotary Air [ ] Rotary Mud [ ] Cable [ ] Auger
[ ] Other

(4) PROPOSED USE:
[X] Domestic [ ] Community [ ] Industrial [ ] Irrigation
[ ] Thermal [ ] Injection [ ] Livestock [ ] Other

(5) BORE HOLE CONSTRUCTION:
Special Construction approval [ ] Yes [X] No Depth of Completed Well 36 ft.
Explosives used [ ] Yes [X] No Type ____ Amount ____

HOLE Diameter	From	To	SEAL Material	From	To	Sacks or pounds
10	0	36	Cement/gel	0	18	12 sks.
			Gravel	18	36	
6	36	148	SEE #12			

How was seal placed: Method [ ] A [ ] B [X] C [ ] D [ ] E
[ ] Other
Backfill placed from ____ ft. to ____ ft. Material ____
Gravel placed from 18 ft. to 36 ft. Size of gravel 3/4

(6) CASING/LINER:

	Diameter	From	To	Gauge	Steel	Plastic	Welded	Threaded
Casing:	6"	+1	36	.250	[X]	[ ]	[X]	[ ]
					[ ]	[ ]	[ ]	[ ]
					[ ]	[ ]	[ ]	[ ]
					[ ]	[ ]	[ ]	[ ]
Liner:					[ ]	[ ]	[ ]	[ ]
					[ ]	[ ]	[ ]	[ ]

Final location of shoe(s)

(7) PERFORATIONS/SCREENS:
[X] Perforations Method ROTARY PERFORATOR
[ ] Screens Type ____ Material STEEL

From	To	Slot size	Number	Diameter	Tele/pipe size	Casing	Liner
19	30	¼x3	33		6"	[X]	[ ]
						[ ]	[ ]
						[ ]	[ ]
						[ ]	[ ]
						[ ]	[ ]

(8) WELL TESTS: Minimum testing time is 1 hour
[ ] Pump [X] Bailer [ ] Air [ ] Flowing Artesian

Yield gal/min	Drawdown	Drill stem at	Time
8	26		1 hr.

Temperature of water 52°F Depth Artesian Flow Found ____
Was a water analysis done? [ ] Yes By whom ____
Did any strata contain water not suitable for intended use? [ ] Too little
[ ] Salty [ ] Muddy [ ] Odor [ ] Colored [ ] Other
Depth of strata:

(9) LOCATION OF WELL by legal description:
County COLUMBIA Latitude ____ Longitude ____
Township 6N N or S Range 5W E or W. WM.
Section 14 ____ 1/4 ____ 1/4
Tax Lot 300 Lot ____ Block ____ Subdivision ____
Street Address of Well (or nearest address) HWY 202

(10) STATIC WATER LEVEL:
10 ft. below land surface. Date 11/09/94
Artesian pressure ____ lb. per square inch. Date ____

(11) WATER BEARING ZONES:
Depth at which water was first found 19

From	To	Estimated Flow Rate	SWL
19	24	8 gpm	10

(12) WELL LOG:
Ground Elevation ____

Material	From	To	SWL
Brown silty clay w/sandy seams	0	19	
Coarse gravel	19	25	10
Sticky gray claystone w/soft streaks	25	148	
LOWER HOLE ABANDONMENT:			
Cement 3 sks	148	135	
Drill gel	135	90	
Cement 2 sks	90	80	
Drill gel	80	50	
Cement 3 sks	50	36	

Date started 11/07/94 Completed 11/09/94

(unbonded) Water Well Constructor Certification:
I certify that the work I performed on the construction, alteration, or abandonment of this well is in compliance with Oregon water supply well construction standards. Materials used and information reported above are true to the best of my knowledge and belief.
WWC Number 1492
Signed [signature] Date 11/14/94

(bonded) Water Well Constructor Certification:
I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. All work performed during this time is in compliance with Oregon water supply well construction standards. This report is true to the best of my knowledge and belief.
WWC Number 1266
Signed [signature] Date 11/14/94

ORIGINAL & FIRST COPY-WATER RESOURCES DEPARTMENT SECOND COPY-CONSTRUCTOR THIRD COPY-CUSTOMER

COLU 52972

COLU 52972

STATE OF OREGON
WATER SUPPLY WELL REPORT
(as required by ORS 537.765)
Instructions for completing this report are on the last page of this form.

WELL I.D. # L 76571
START CARD # 173131

(1) LAND OWNER Well Number
Name DAVE Parkin Mr Deck
Address 14154 S. Holcomb
City Oregon City State OR Zip 97045

(2) TYPE OF WORK
☑ New Well ☐ Deepening ☐ Alteration (repair/recondition) ☐ Abandonment

(3) DRILL METHOD:
☑ Rotary Air ☐ Rotary Mud ☐ Cable ☐ Auger
☐ Other

(4) PROPOSED USE:
☑ Domestic ☐ Community ☐ Industrial ☐ Irrigation
☐ Thermal ☐ Injection ☐ Livestock ☐ Other

(5) BORE HOLE CONSTRUCTION:
Special Construction approval ☐ Yes ☐ No Depth of Completed Well 60 ft.
Explosives used ☐ Yes ☑ No Type — Amount —

HOLE Diameter	From	To	SEAL Material	From	To	Sacks or pounds
10	0	42	3/8 Bentonite	0	16	500 lb
10	0	42	Cement	16	19	100 lb
6	42	60				

How was seal placed: Method ☐ A ☐ B ☐ C ☐ D ☐ E
☑ Other Bentonite poured Dry / Cement Tremmie
Backfill placed from ___ ft. to ___ ft. Material
Gravel placed from 19 ft. to 42 ft. Size of gravel 3/8 X 3/8

(6) CASING/LINER:

	Diameter	From	To	Gauge	Steel	Plastic	Welded	Threaded
Casing:	6	+6"	42	.250	☑	☐	☑	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
					☐	☐	☐	☐
Liner:	4	1	60		☐	☑	☑	☐
					☐	☐	☐	☐

Drive Shoe used ☐ Inside ☐ Outside ☑ None
Final location of shoe(s)

(7) PERFORATIONS/SCREENS: Torch / Steel
☐ Perforations Method Cir Saw / PVC
☐ Screens Type Material

From	To	Slot size	Number	Diameter	Tele/pipe size	Casing	Liner
20	42	1/8 X 12	52			☑	☐
						☐	☐
50	60	1/8 X 7	81			☐	☑
						☐	☐

(8) WELL TESTS: Minimum testing time is 1 hour
☐ Pump ☐ Bailer ☑ Air ☐ Flowing Artesian

Yield gal/min	Drawdown	Drill stem at	Time
			1 hr.
8	38	48	4 hr

Temperature of water 53° Depth Artesian Flow Found —
Was a water analysis done? ☐ Yes By whom —
Did any strata contain water not suitable for intended use? ☐ Too little
☐ Salty ☐ Muddy ☐ Odor ☐ Colored ☐ Other
Depth of strata: —

(9) LOCATION OF WELL by legal description:
County Col Latitude Longitude
Township 6 N N or S Range 5 W E or W. WM.
Section 14 SE 1/4 SE 1/4
Tax Lot 503 Lot Block Subdivision
Street Address of Well (or nearest address) Next to 14756 Hwy 202 (Mist Dr)

(10) STATIC WATER LEVEL:
10 ft. below land surface. Date 20 MAR
Artesian pressure ___ lb. per square inch Date

(11) WATER BEARING ZONES:
Depth at which water was first found 20

From	To	Estimated Flow Rate	SWL
20	24	5 - 8	10

(12) WELL LOG:
Ground Elevation Aprox 535

Material	From	To	SWL
Top Soil	0	2	
Brown Clay	2	10	
Brown Sandy Clay	10	17	
Blue Silty Clay	17	20	
Med to Coarse Gravel & Sand	20	24	10
Grey Clay Stone	24	60	10

RECEIVED
MAR 27 2006
WATER RESOURCES DEPT
SALEM, OREGON

Date started 13 MAR 06 Completed 20 MAR 06

(unbonded) Water Well Constructor Certification:
I certify that the work I performed on the construction, alteration, or abandonment of this well is in compliance with Oregon water supply well construction standards. Materials used and information reported above are true to the best of my knowledge and belief.
Signed [signature] WWC Number 1538 Date 21-MARCH-06

(bonded) Water Well Constructor Certification:
I accept responsibility for the construction, alteration, or abandonment work performed on this well during the construction dates reported above. All work performed during this time is in compliance with Oregon water supply well construction standards. This report is true to the best of my knowledge and belief.
WWC Number 602
Signed [signature] Date April 21 06

RECEIVED
SEP 13 2007
WATER RESOURCES DEPT
SALEM OREGON

ORIGINAL – WATER RESOURCES DEPARTMENT FIRST COPY – CONSTRUCTOR SECOND COPY – CUSTOMER

COLU 53559

STATE OF OREGON
WATER SUPPLY WELL REPORT
(as required by ORS 537.765 & OAR 690-205-0210)

WELL LABEL # L 95426
START CARD # 198399

(1) LAND OWNER
Owner Well I.D. COLU. 1874
First Name ______ Last Name ______
Company Northwest Natural Gas Co.
Address 226 NW 2nd., PO Box 4709
City Portland State OR Zip 97208-4709

(2) TYPE OF WORK
☐ New Well ☐ Deepening ☐ Conversion
☒ Alteration (repair/recondition) ☐ Abandonment

(3) DRILL METHOD
☒ Rotary Air ☐ Rotary Mud ☐ Cable ☐ Auger ☐ Cable Mud
☐ Reverse Rotary ☐ Other

(4) PROPOSED USE
☒ Domestic ☐ Irrigation ☐ Community
☐ Industrial/ Commercial ☐ Livestock ☐ Dewatering
☐ Thermal ☐ Injection ☐ Other

(5) BORE HOLE CONSTRUCTION
Special Standard ☐ (Attach copy)
Depth of Completed Well 360 ft.

BORE HOLE Dia	From	To	SEAL Material	From	To	Amt	sacks/ lbs

How was seal placed: Method ☐ A ☐ B ☐ C ☐ D ☐ E
☐ Other
Backfill placed from ______ ft. to ______ ft. Material ______
Filter pack from ______ ft. to ______ ft. Material ______ Size ______
Explosives used: ☐ Yes Type ______ Amount ______

(6) CASING/LINER

Casing	Liner	Dia	+	From	To	Gauge	Stl	Plstc	Wld	Thrd
○	◉	4.5	☐	6	360	160#	○	◉	☒	☐

Shoe ☐ Inside ☐ Outside ☐ Other Location of shoe(s) ______
Temp casing ☐ Yes Dia ______ From ______ To ______

(7) PERFORATIONS/SCREENS
Perforations Method Drilled
Screens Type ______ Material PVC 160

Perf/ Screen	Casing/ Liner	Screen Dia	From	To	Scrn/slot width	Slot length	# of slots	Tele/ pipe size
Perf	Liner	4.5	240	260	.5		30	
Perf	Liner	4.5	280	300	.5		30	
Perf	Liner	4.5	340	360	.5		30	

(8) WELL TESTS: Minimum testing time is 1 hour
○ Pump ○ Bailer ◉ Air ○ Flowing Artesian

Yield gal/min	Drawdown	Drill stem/Pump depth	Duration (hr)
30		360	1
25		300	.2

Temperature 54 °F Lab analysis ☐ Yes By ______
Water quality concerns? ☐ Yes (describe below)

From	To	Description	Amount	Units

RECEIVED
MAY 21 2008
WATER RESOURCES DEPT
SALEM, OREGON

(9) LOCATION OF WELL (legal description)
County COLUMBIA Twp 6 N N/S Range 5 W E/W WM
Sec 11 SW 1/4 of the NW 1/4 Tax Lot 2500
Tax Map Number ______ Lot ______
Lat ° 0 ' " or ______ DMS or DD
Long ° 0 ' " or ______ DMS or DD
◉ Street address of well ○ Nearest address
NWNG Mist Miller Station, Clatskanie, OR

(10) STATIC WATER LEVEL

	Date	SWL(psi)	+	SWL(ft)
Existing Well / Predeepening	05-06-2008		☐	198
Completed Well	05-09-2008		☐	198

Flowing Artesian? ☐ Dry Hole? ☐

WATER BEARING ZONES Depth water was first found 240

SWL Date	From	To	Est Flow	SWL(psi)	+	SWL(ft)
05-09-2008	240	360	30		☐	198

(11) WELL LOG
Ground Elevation ______

Material	From	To
Existing 6" well (COLU 1874) with steel liner broken @ 250 ft. Removed top portion (165 ft.) of steel liner. Remaining (125 ft.) could not be fished out. Milled/side drilled remaining liner and installed 4.5" PVC Liner from 6 ft.-360 ft.		

Date Started 05-06-2008 Completed 05-09-2008

(unbonded) Water Well Constructor Certification
I certify that the work I performed on the construction, deepening, alteration, or abandonment of this well is in compliance with Oregon water supply well construction standards. Materials used and information reported above are true to the best of my knowledge and belief.
License Number 1492 Date 05-12-2008
Password : (if filing electronically)
Signed [signature]

(bonded) Water Well Constructor Certification
I accept responsibility for the construction, deepening, alteration, or abandonment work performed on this well during the construction dates reported above. All work performed during this time is in compliance with Oregon water supply well construction standards. This report is true to the best of my knowledge and belief.
License Number 573 Date 05-12-2008
Password : (if filing electronically)
Signed [signature]
Contact Info (optional)

ORIGINAL - WATER RESOURCES DEPARTMENT
THIS REPORT MUST BE SUBMITTED TO THE WATER RESOURCES DEPARTMENT WITHIN 30 DAYS OF COMPLETION OF WORK
Form Version: 0.88